Job Number: 150607 (St Johns Wood Park)
Date: 17 July 2015

CROFT STRUCTURAL ENGINEERS

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Basement Impact Assessment

Property Details
1B St Johns Wood Park
London
NW8

Client Information
Mike Ofori

Structural Design Reviewed by	Above Ground Drainage Reviewed by
Chris Tomlin MEng CEng MIStructE	Phil Henry BEng MEng MICE

Hydrology Report	Geology Report
Hannah Fraser CGeol Separate report	Jon Smithson CGeol Separate Report

Revision	Date	Comment
-	July 2015	First issue
1	Aug 2015	Drawings revised

Contents

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Executive Summary / Non-technical Summary

The London Borough of Camden requires a Basement Impact Assessment (BIA) to be prepared for developments including basements and light wells within its area of responsibility. CGP4 – Basements and Light wells details the requirements for a BIA undertaken in support of proposed developments; in summary the Council will only allow basement construction to proceed if it does not:

- Cause harm to the built environment and local amenity;
- Result in flooding;
- Lead to ground instability.

In order to comply with the above clauses a BIA must undertake 5 stages detailed in CPG 4. This report has been produced in line with the guidance of CPG4 and the associated documents supporting CGP4 such as DP23, DP26, DP25 & DP27.

Project Summary

Description of Property

The existing site has a series of garages that will be demolisheed to give way for new basement and new two storeys on top of basement.

Proposed Works

The proposed works require the construction of:

- A new basement and a new two storey dwelling above basement.
- Light wells to the front and rear
- Superstructure works above the basement
  - New two storey dwelling above basement.

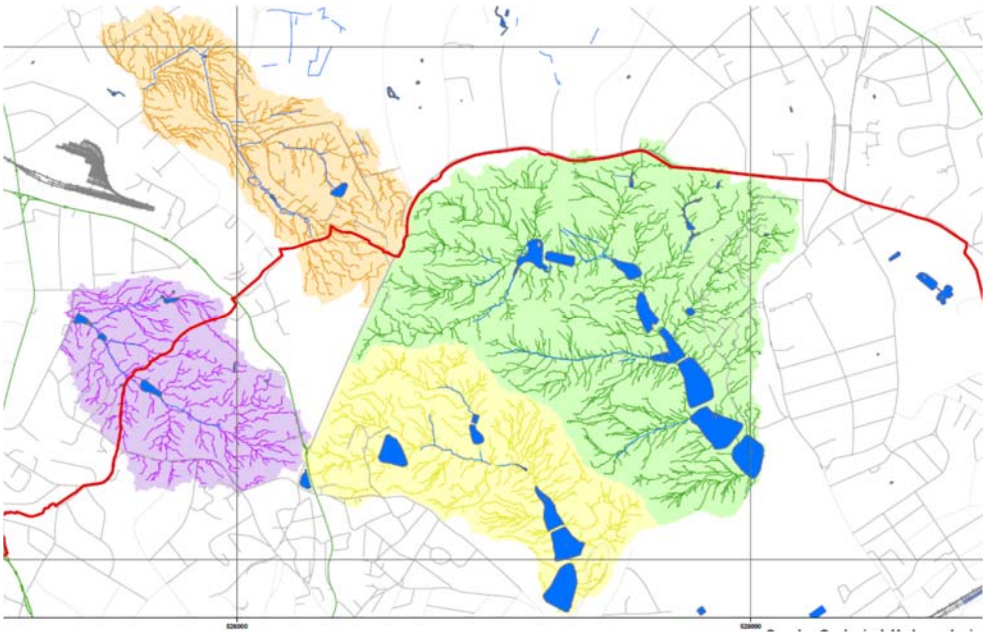
Croft Structural Engineers Ltd has extensive knowledge of constructing new basements. Over the last 10 years Croft Structural Engineers has been involved in the design of over 500 basements in and around London. The method to be utilised at 1B ST JOHNS WOOD PARK is:

1. Excavate front to allow for conveyor to be erected.
2. Safely and securely support the existing building above
3. Form lightwell with cantilevered retaining walls

	4. Slowly work from the front to the rear inserting narrow cantilevered retaining walls sequentially using well developed and understood underpinning methods. 5. Prop retaining walls in temporary condition back to the central soil "dumpling". 6. Prop across the width of the basement, excavate central soil "dumpling" & cast basement slab 7. Waterproof internal space with a drained cavity system.
Stage 1 – Screening	Screening identified areas of concern and concluded a requirement to proceed to a scoping stable for the Land stability, Hydrology, Surface Water and flooding.
Stage 2 – Scoping	The Scoping stage identified the potential impacts and set the parameters required for further study of the areas of concern highlighted in the Screening phase. A walk over desk survey completed by an engineer. The information from this was utilised to formulate the requirement for a ground, Geology and hydrogeology investigation.
Stage 3 – Site investigation and study	A Structural engineer inspected the building to determine the current condition of the property. Visual inspections were completed of the adjacent properties to determine if there were signs of structural movement. The neighbouring land has not been excavated on but an engineer has assessed the age of the adjacent properties and considered the type of foundations used for that period and assumed these in the design. A ground investigation with 12.5m deep boreholes has been completed. • The formation level of the basement will be in London Clay • Initial standpipe readings did not encounter any water Laboratory testing was undertaken on the soil samples. Ground water has been measured over repeat visits to determine water levels and flows. • A repeat observed water at 0.5m below ground level

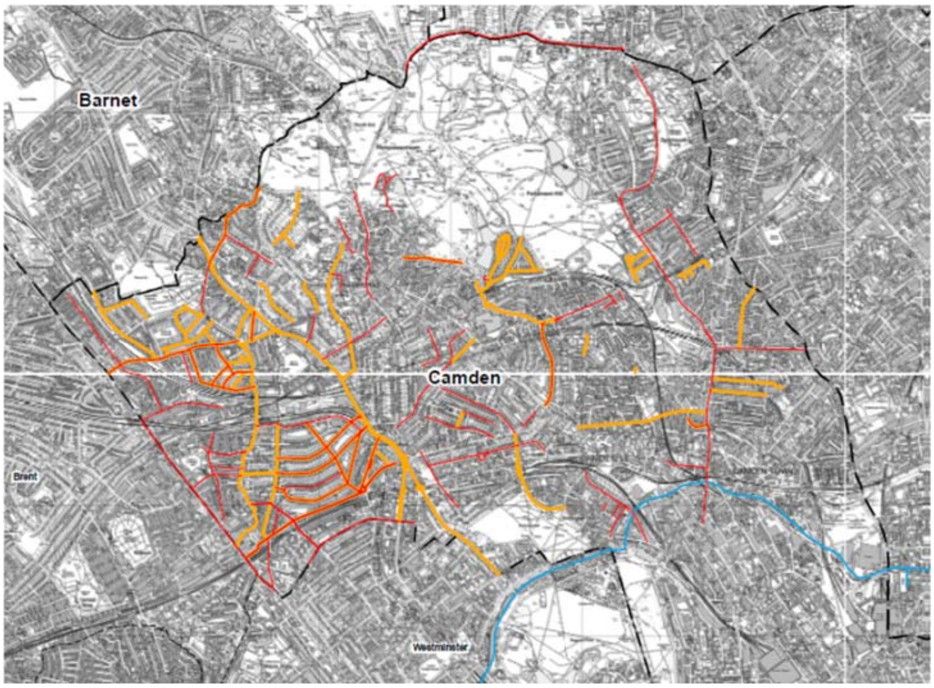
Stage 4 – Impact assessment	**Land stability** The Geologist has concluded that the basement will not make the area unstable. The movement assessment of the basement and its construction are SLIGHT 1-0 on the Burland scale. It has been concluded that The boreholes records have indicated the presence possible perched groundwater to a depth of 0.49 m bgl. However if groundwater is recorded during the construction works it anticipated that any inflow will be very modest, on the basis of the ground conditions encountered. The groundwater would be controlled by pumping to a tank prior to disposal by tanker to an approved facility. Alternatively discharge of the groundwater could be made to the sewer subject to an agreement from the local water company in terms of water quality, flow rate and quantity. Groundwater levels should be continued to be monitored before, during and after construction. Monitoring of adjacent structures and the highway should be carried out before, during and after construction. **Hydrogeology** It is understood that the basement retaining walls will be a contiguous piled wall. Therefore excavation for the basement will be protected from instability by the piled wall. Excavation of the basement area will need to comply with appropriate health and safety criteria in terms of height and width of excavation face. **Drainage & Surface Water Flow** The risk of flooding from excess surface water is not considered to be significant.

1. Screening Stage

	This stage should identify any areas for concern and therefore focus effort for further investigation. The questions below are taken from the Camden CPG 4 – Basements and Lightwells.
Land Stability	Refer to Chartered Geologist Report.
Subterranean Flow	Refer to Chartered Hydrogeologist report completed by A Hydrogeologist with the "CGeol" (Chartered Geologist) qualification from the Geological Society of London.
Surface Flow and Flooding	
	Question 1: **Is the site within the catchment of the pond chains on Hampstead Heath?**  *Figure 1: Extract from figure 14 of the Hydrological Study* **No.** The site lies outside the areas denoted by figure 14 of the Arup report.

	Question 2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route? Due to the construction of the garden basement and the rear lightwell, the flow of water into the ground and the existing surface water drainage system may change. Carry forward to scoping.
	Question 3. Will the proposed basement development result in a change to the hard surfaced /paved external areas? Due to the construction of the garden basement the hard surface/paved external areas may change.
	Question 4. Will the proposed basement result in changes to the inflows (instantaneous and long term of surface water being received by adjacent properties or downstream watercourses? **No**. The proposed development will enter the current drainage system.
	Question 5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses? **No**. The quality of water is unlikely to be altered.
	Question 6 : IS the site in an area identified to have surface water flood risk according to either the Local Flood Risk Management Strategy or the Strategic Flood Risk Assessment or is it at risk from flooding, for example because the proposed basement is below the static water level of nearby surface water feature? The potential sources of flooding are summarised below:

Potential Source	Potential Flood Risk At Site?	Justification
Fluvial flooding	No	EA Flood Mapping shows Flood Zone 1. Distance from nearest surface watercourse >1km
Tidal flooding	No	Site location is 'inland' and topography > 40mAOD.
Flooding from rising / high groundwater	No	Site is located on low permeability London Clay.
Surface water (pluvial) flooding	Yes	The 1B ST JOHNS WOOD PARK is noted on the flood street list and maps from 1975 or 2002 (shown graphically below)



Flooding from infrastructure failure	Yes	Drainage at or near the site could potentially become blocked or cracked and overflow or leak. Drainage of the basement terrace areas may rely on pumping.
Flooding from reservoirs, canals and other artificial sources	No	There are no reservoirs, canals or other artificial sources in the vicinity of the site that could give rise to a flood risk.

Figure 2: Extract from OS map showing contour lines

Carry forward to Scoping Stage

2. Scoping Stage

	Identifies the potential impacts of the areas of concern highlighted in the Screening phase.
Land Stability	Refer to Chartered Geologist Report.
Subterranean Flow	Refer to Chartered Hydrogeologist report. Completed by A Hydrogeologist with the "CGeol" (Chartered Geologist) qualification from the Geological Society of London.
Surface Flow & Flooding	**Conceptual Model** The proposed works at 1B ST JOHNS WOOD PARK require new basement and new two storey dwelling above basement. The basement is under the footing print of the property which will not affect the overall flow. Lightwells increase the hardstanding slightly which may increase flow.
	Question 1: **Is the site within the catchment of the pond chains on Hampstead Heath?** **No** further info required from Scoping stage
	Question 2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route? **No**. Due to the construction of the garden basement and the rear lightwell, the flow of water into the ground and the existing surface water drainage system may change. Carry forward to scoping.
	Question 3. Will the proposed basement development result in a change to the hard surfaced /paved external areas? **Unknown** Due to the construction of the garden basement the hard surface/paved external areas may change.
	Question 4. Will the proposed basement result in changes to the inflows (instantaneous and long term) of surface water being received by adjacent properties or downstream watercourses? **Unknown** – The light wells may reduce the impermeable areas. Carry forward to Site Investigation & desk Study

	~~**Question 5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?**~~ **No**.
	Question 6 : Is the site in an area identified to have surface water flood risk according to either the Local Flood Risk Management Strategy or the Strategic Flood Risk Assessment or is it at risk from flooding, for example because the proposed basement is below the static water level of nearby surface water feature? It is evident from the screening study that the only significant flood risks at 1B ST JOHNS WOOD PARK are due surface water (pluvial) flooding and failure of existing sewers in the vicinity of the site. Carry forward to Site Investigation & Desk Study

3. Site Investigation and Study

	Identifies the relevant features of the site and its immediate surroundings providing further scoping where required.
	Desk Study and Walkover Survey The existing site has a series of garages that will be demolisheed to give way for new basement and new two storeys on top of basement. Noma Manzini, a Structural Engineer from Croft Structural Engineers visited 1B ST JOHNS WOOD PARK. Date of inspection was on the 16th of June 2015
Proposed Development	The existing site has a series of garages that will be demolisheed to give way for new basement and new two storeys on top of basement.  *Figure 3: 1B St Johns Wood Park*
Site History	What was the previous usage of the site?

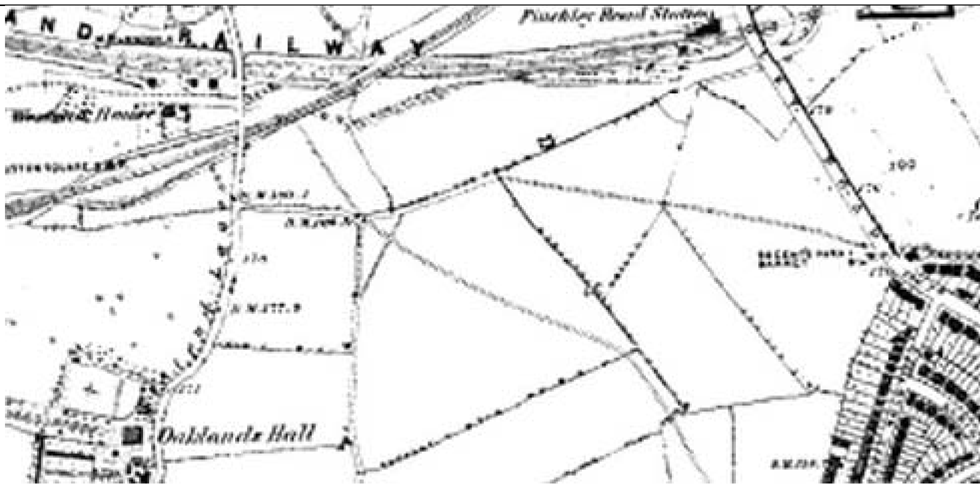
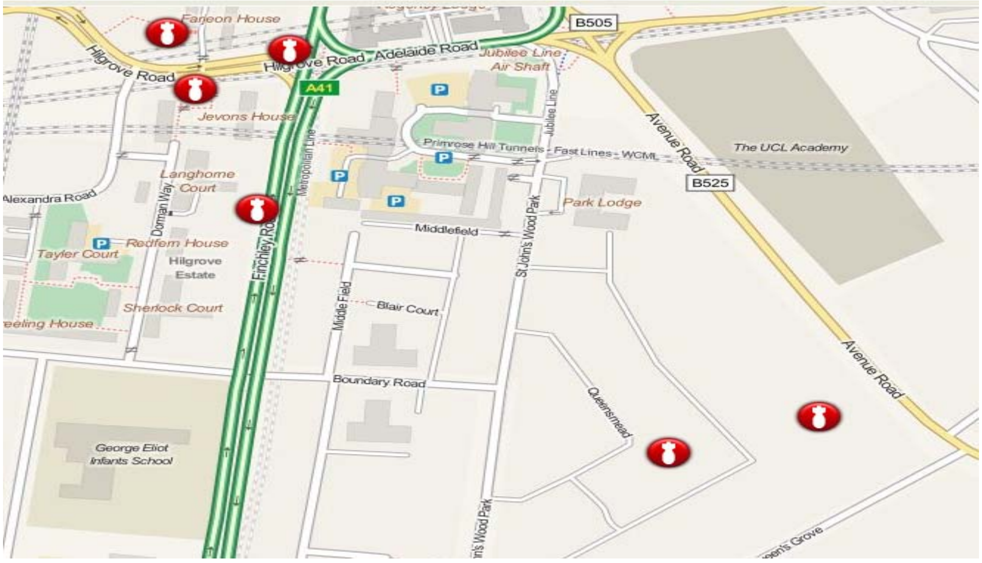
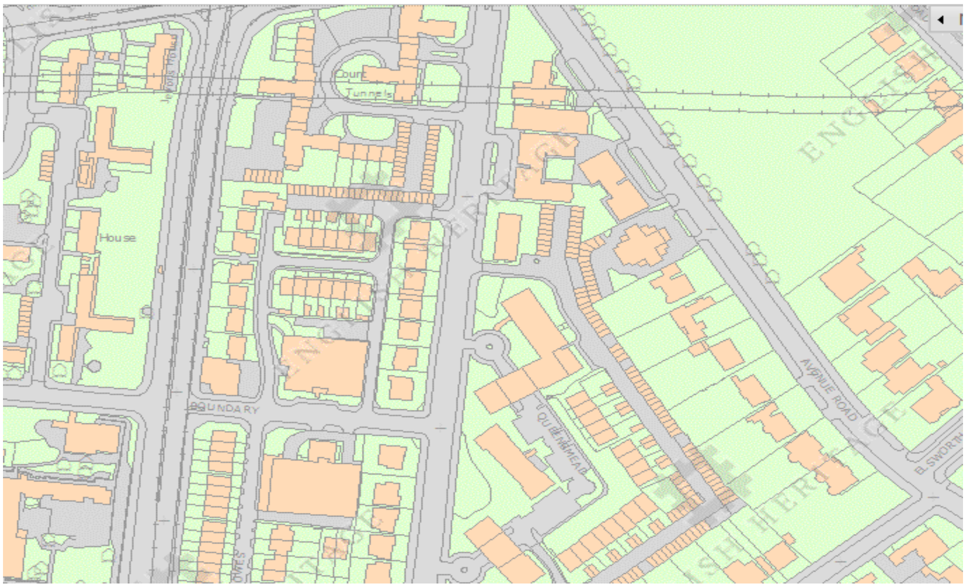


Figure 4: Map from 1850

Figure 5: Map from 1895

The site is noted in Pevsner's Architectural Guide, London 4: North. The area is described as 'an enclave of trim mid-nineteenth century stuccoed streets'

Local Bombing	 *Figure 6: Extract from Bomb Survey Map* A highly explosive bomb is recorded in the Aggregate Night Time bomb census as having been dropped between the 7th of October to 6th of June 1941
Listed Buildings	**Is the building or Adjacent buildings listed**  *Figure 7: Extract showing listed buidlings* **No.** neither the area nor the adjacent buildings are listed.

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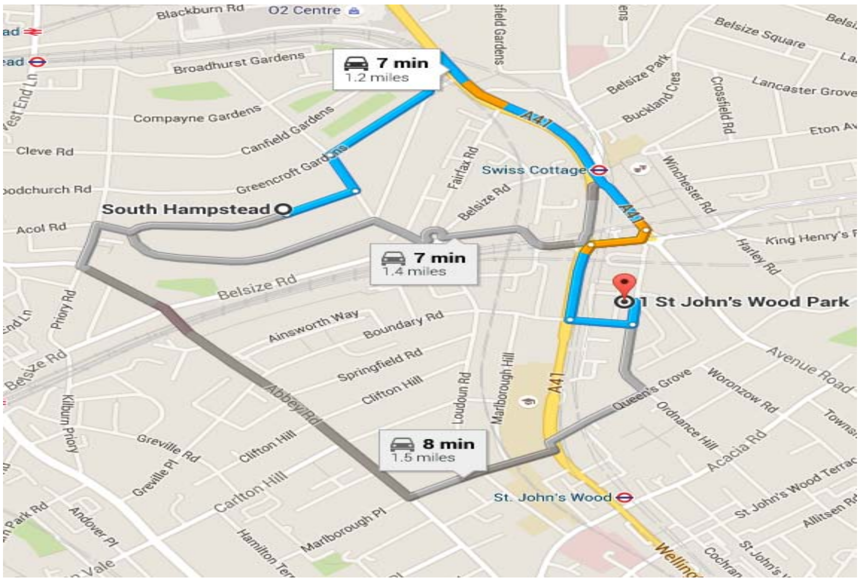
	Highways, Rail and London Underground **Yes.** Site is within 5m of the footpath/alleyway and the road surface is further than 5m from the front lightwell.
London Underground and Network Rail	**Is the site over (or within the exclusion zone) of any tunnels, e.g. railway lines?** **No.** Nearest is the Overground Rail, +/- 65m from site.  *Figure 8: Map showing proximity of rail lines*
UK Power Networks	**Will the basement works affect any UK Power Network Assets?** No, there no significant items of electrical infrastructure (such as pylons or substations) in the immediate vicinity
Vicinity of Trees	Some mature trees and general vegetation in the neighbouring garden; A mature tree is also present in the neighbouring garden. There are trees close by with have tree presentation orders. These are across the road and are not present in the neighbouring gardens.

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Adjacent Properties

The condition of the adjacent buildings have been inspected to consider whether the basement will significantly affect their structure.

Visual inspections of the internal facades have been undertaken of the properties.

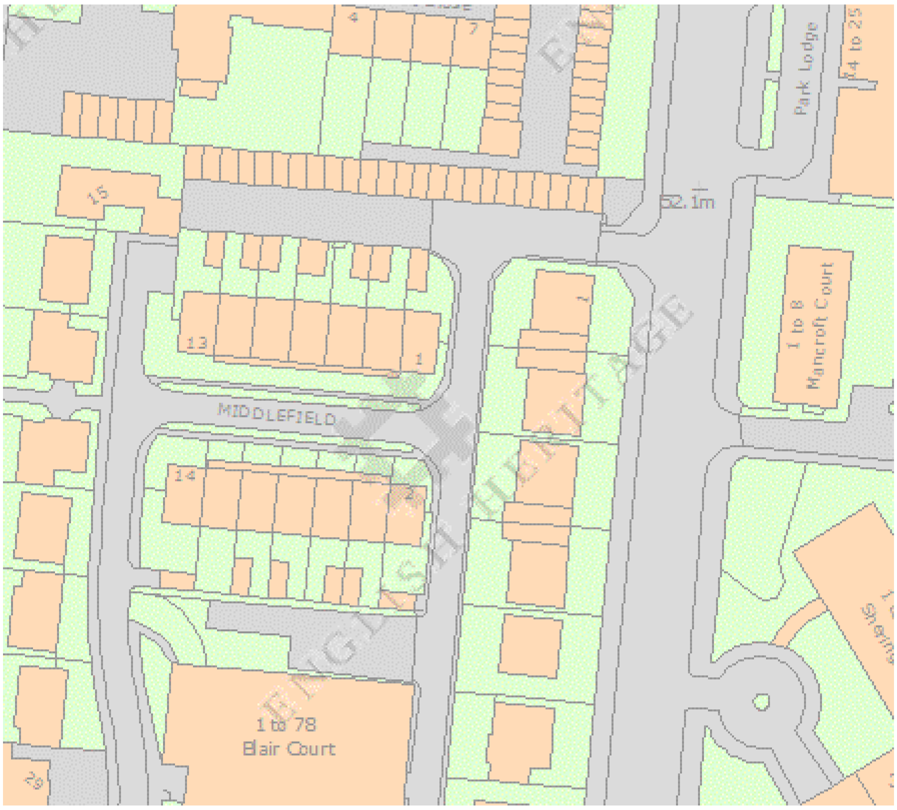


Figure 9: Plan view of neighbouring property

Nos 1 St Johns Wood Park – Property to Left

Property Age : mid Victorian

Property use : Residential

Number of storeys : 2

Is a basement present? : No there is no basement present.

Structural Defects Noted

Structural Assessment of ongoing movement: Note signs of cracking particularly diagonal cracking

<table>
<tr><td></td><td>
Figure 10: 1 St Johns Wood Park</td></tr>
<tr><td>Adjacent apartments Property to Right</td><td>Property Age : Victorian

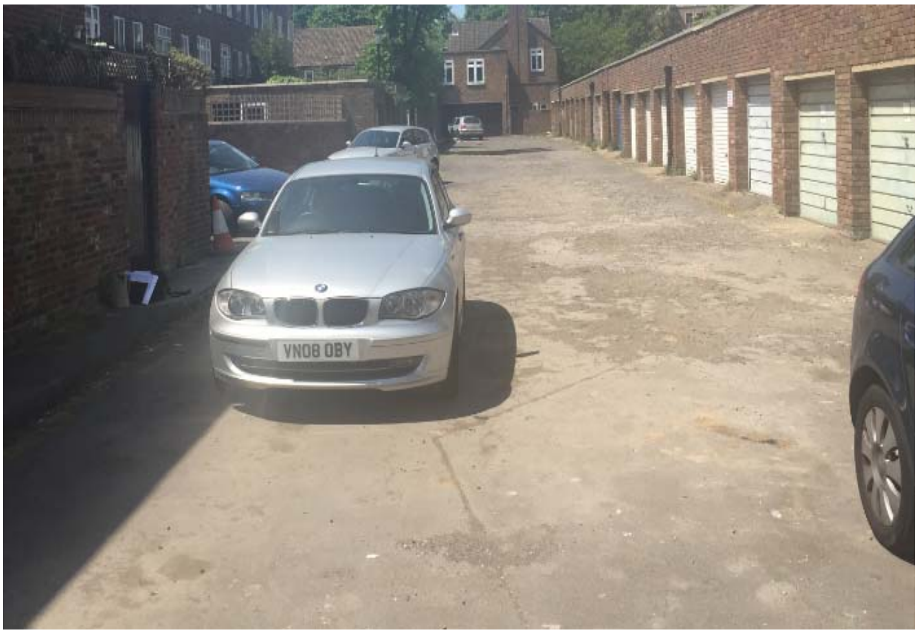
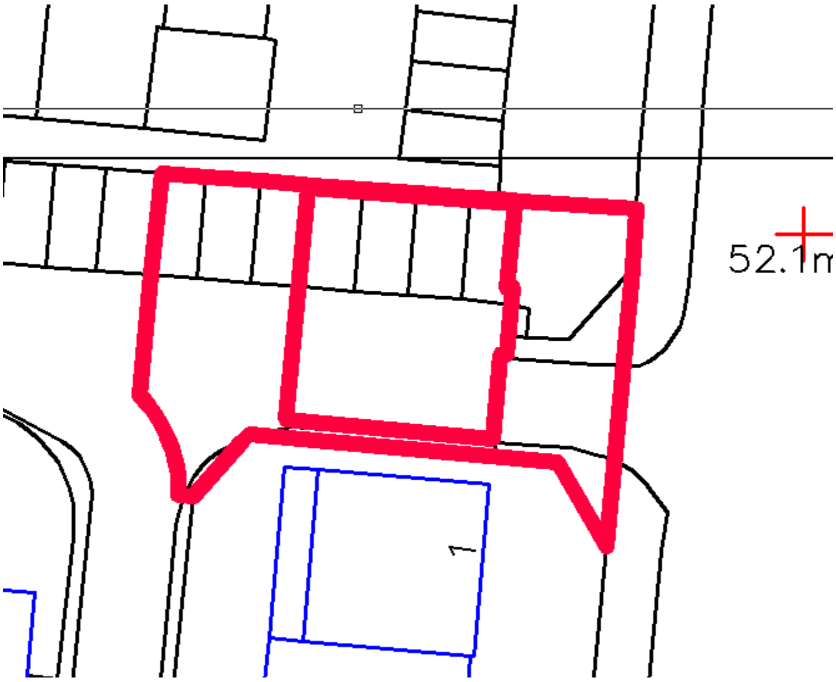
Property use : Residential

Number of storeys : more than 5 storeys

There is apartments more than 5m away. Given the height of apartments piled foundations are assumed, which will be deeper than the formation of the proposed basement.


Figure 11: Adjacent apartments</td></tr>
</table>

Nos 1 Middlefield – Property to Rear	Property Age : Victorian Property use :Residential Number of storeys : 2 Is a basement present? :No Structural Defects Noted: No structural defects noted externally by visual inspection.  *Figure 12: 1 Middlefield*
Local Topography	As mentioned previously, the area surrounding the property has a general slope, downwards from north-west to south-east. The slope is gradual; there are no retaining walls for sudden changes in elevation
Ground Investigation	A ground investigation see separate report.
Geology	See Ground investigation report and Geology report

Surface Flow & Flooding	
Areas of Hard Standing present on site	Existing Area of hardstanding outside is ; Area = 244m^2  *Figure 13: Hard standing area*  52.1m *Figure 14: Areal view*

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Rainwater down pipes, Drains, Manholes and Gulleys	As described previously, there is a surface water drainage gully in the front yard and pea-shingle drainage in the rear yard.
Local Water Sources	**Are there any ponds lakes or water courses on the site or adjacent sites?** No, there are not surface water features (natural or man-made) on the adjacent sites.
	Field Investigation Ground investigation specialists visited the site and subsequently produced are report for the existing ground and groundwater conditions.
	Monitoring, Reporting and Investigation The ground investigation report, which has data from initial site investigations and data from subsequent monitoring, is available as a separate report. Data relevant to land stability and subterranean flow is examined separate documents as described below.
Land Stability	Refer to Chartered Geologist Report for land stability issues addressed to Stage 3. Features and items of concern relating to data from Stage 3 are included in this report.
Subterranean Flow	Refer to Chartered Hydrogeologist report (Basement Impact Assessment: Groundwater). This is completed by a Hydrogeologist with the “CGeol” (Chartered Geologist) qualification from the Geological Society of London. Features and items of concern relating to data from Stage 3 are included in this report.

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Site Investigation

Soil investigation Brief	The Soil investigation was completed by (Ground and Water). From the Scoping stage we considered that their brief should cover: • Two trial pits to the side and rear to confirm the existing foundations of existing garages. The purpose is to consider the effect of the works on the neighbouring properties and the find the ground conditions below the site. • Bore holes to a depth of 12.5m below ground level (i.e. approximately more than twice the depth of the proposed basement). • Stand pipe to be inserted to monitor ground water; record initial strike and the water level after 1 month. • Site testing to determine insitu soil parameter. SPT testing to be undertaken. • Laboratory testing to confirm soil make up and properties. • The Historic maps and walk over survey did not highlight any significant contamination sources, therefore no site test of the ground has been requested. • Factual Report on soil conditions. • Interpretative reports • Calculation of bearing pressures from SPT. • Indication of Ø (angle of friction) from SPT. • Indication of soil type Soil Report is provided under a separate cover.

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4. Basement Impact Assessment

Subterranean Flow	Refer To Hydrogeologist report : Conclusions re stated in the Executive Summary
Land Stability	Refer to Geologist Report: Conclusions re stated in the Executive Summary
Conservation and Listed Buildings	If the property is in a conservation area, or it is listed then management plan for demolition and construction may be needed. This is not included with this BIA document and is not within the Croft Structural Engineers Brief.

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Flood Risk Assessment

In accordance with guidance from CIRIA, PPS25 and the National Planning Policy Framework, the basement will be designed to be sustainable in terms of the risk of flooding. Amongst other considerations, the design will include provisions to minimise the adverse impacts of flooding on the operation of the building, the users, the surroundings and the occupants of nearby properties. These design measures must be preceded by a Flood Risk Assessment (FRA), and is staged as follows:

- A screening study to identify potential sources of flooding and confirm the need for an FRA. This has been carried out in the Section 1.
- A subsequent scoping study to identify sources of flooding and also other features relevant to flooding. This has been done in the previous sections.
- An impact assessment with flood risk management options proposed. This is presented in this section.

Site Location

The site is approximately 400m² in size. It is located in a densely built-up area. These buildings are at the same level. There are gardens to the rear of the site. Immediately to the front, the road is relatively flat. There are no rivers or surface water features within 250km of the site.

From inspection of OS maps, the site appears to lie on ground which slopes down from north-west to south-east, by approximately 1 in 40.

The EA has not identified any flood risks associated with the nearby water courses.

The EA has not identified any flood risks associated with the nearby water courses.

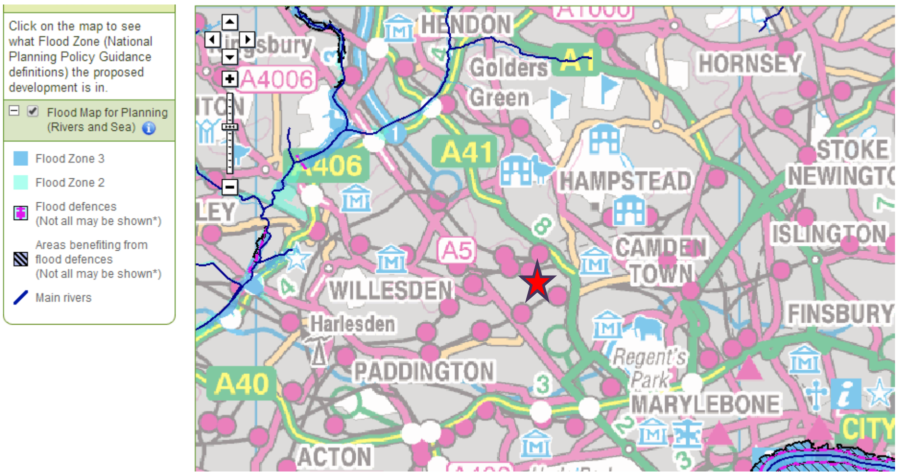


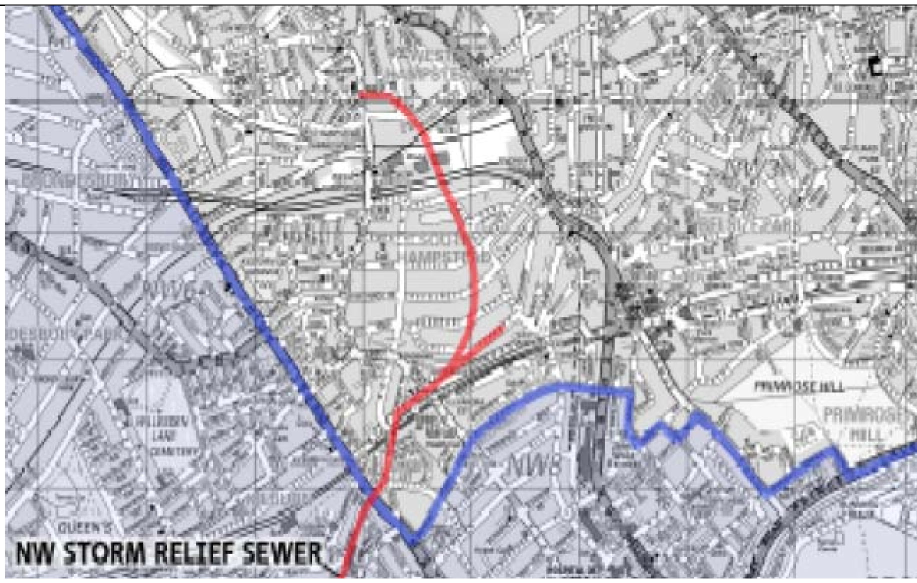
Figure 15: Flood map for planning (Environment Agency)

The site is within Zone 1, a low probability flood risk area.

Potential surface water (pluvial) flooding

1B St Johns Wood Park is reported to have flooded in 2002

It is understood that this flooding was due to the Thames Water relief sewer being overloaded.

	 *Figure 16: Location of Thames Water's North-West storm relief sewer (n red)* It is understood that this flooding was due to the Thames Water relief sewer being overloaded. It is also understood that Thames Water subsequently increased the capacity of this relief system: the likelihood of flooding of this nature is now significantly reduced.
Potential flooding from infrastructure failure	In addition to the storm water relief sewer previously mentioned, there is believed to be a trunk sewer running along the length of the St Johns Wood Park. Blockage or failure of either of these may result in excess flow from the St Johns Wood may accumulate in the front yard. The hard standing in the front yard and the brick wall which separates it from No 1, significantly inhibit the flow of any excess water into the neighbouring property. This will continue to be the case under the proposed development. The added risk of flooding is therefore greater for the owner for 1B St Johns Wood Park than for the adjacent owner. The risk of damage to the property is greatest for the new proposed basement: if the surface water drains become blocked and overflow, then water may enter the front lightwell and damage the basement.

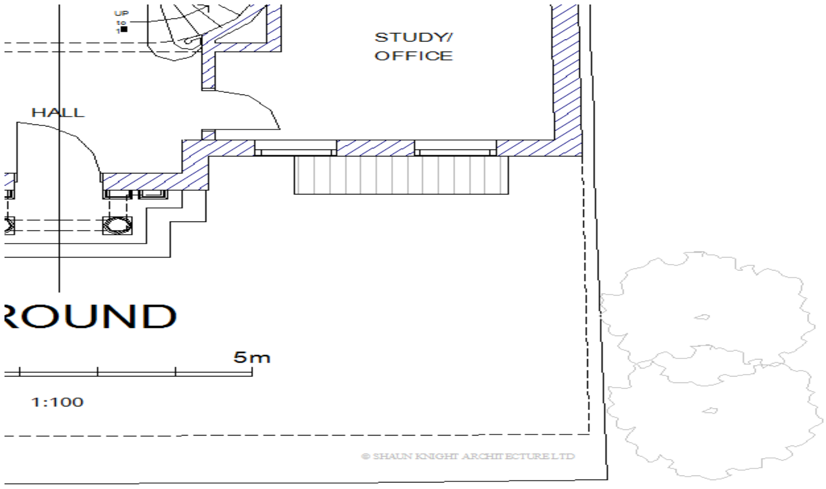
Mitigation measures	This risk, and the extent of the related damage can be reduced as follows: • At ground level, an upstand can be constructed around the front lightwell. • A pumping mechanism will be installed for the proposed basement. There is a likelihood that this may fail and allow excess water to accumulate. If this were to occur, the build-up of water would be gradual and noticeable before it becomes a significant life-threatening hazard. • Install a dual pumping system to maintain operation in the event of a failure. This should include a battery backup and a suitable alarm system for warning purposes. • To reduce the impact of surface water flooding, sustainable drainage systems such as on site attenuation should be considered at detailed design stage.
Summary	The risk of flooding from excess surface water is not considered significant. There is a risk of flooding due to the failure of the pumping system but this can be reduced to acceptable levels with appropriate design and installation measures.

SUDS Assessment

Hard standing	The main design change resulting in the reduction of hardstanding is the removal of the existing garages. The proposed landscaping for the rear yard has not been designed in detail. It is possible that an area similar in size could be incorporated. This would result in the proportion of hard-standing remaining unchanged. These calculations assume that this design feature will not be used and therefore cover the worst case. Existing Hard Standing = 244 m^2 Proposed Hardstanding = 244 m^2 Percentage Increase in Hard standing = 0 %
SUDS Assessment	From review of the existing and proposed hardstanding the increase will be? 0 % Percentage Increase < 5% — No SUDS to be incorporated into scheme Percentage Increase Between 5% to 10% — Where garden basements are present then a soil band of a minimum of 1m should be provided. Where 1m of soil is not present then SUDs is required
Drainage effects on Structure	Not build over agreements known of. Flooding. The site is not in an area of high risk flooding.

Percentage Increase < 5%	No SUDS to be incorporated into scheme
Percentage Increase Between 5% to 10%	

Trees

Root Protection Zone	RPA = 1.5 x Crown diameter.  . *Figure 17: Part ground floor plan* The basement is within the RPA of the trees noted below
Conclusion	The Basement does Cuts into the Root protection Zone The increased depth of foundations necessary for the basement places the new foundations outside the effects of trees. The building will be more stable due to the new basement.

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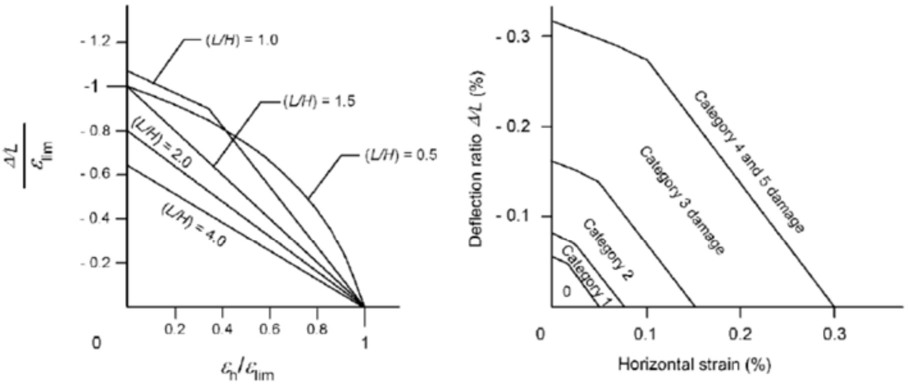
Ground Movement Assessment & Predicted Damage Category

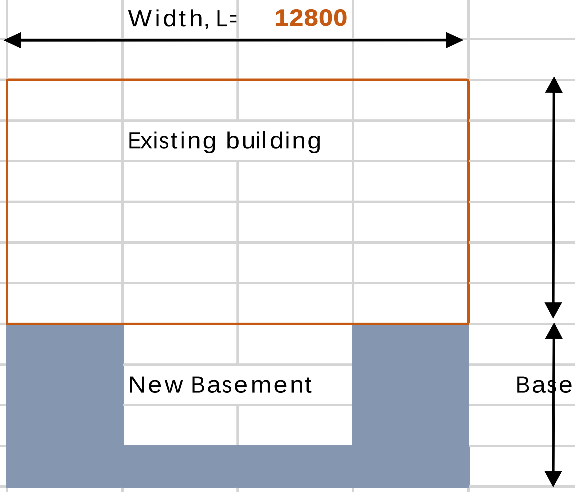
This assessment covers both short term and long term movements relating to the construction and the performance of the permanent works. The design and construction methodology aims to limit damage to the existing building on the site and to all adjoining buildings to Category 1 as set out in Table 2.5 of CIRIA report C 580 .

This assessment has used empirical means as set out in CIRIA2 C 580 Embedded Retaining Walls: Guidance for Economic Design.



(b) Influence of horizontal strain on Δ/L / ε_{lim} (after Burland, 2001)

(c) Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for (L/H) = 1.0 (after Burland, 2001)



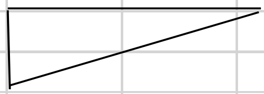
L/H = 1.42222

Horizontal movement Assessment CIRIA C580: Embedded Retaining walls - Guide to Ecomonic Design

Potential Movement Due to wall installation

Horizontal surface movement = 0.05%

DeltaH = 0.05% x 3500 = 1.75 mm



Vertical Surface Movement = 0.05% 1.75

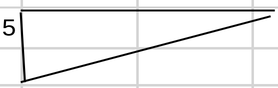
Delta V = 0.05% x 3500 = 1.75 mm = 0.33333 mm/m

Distance behind wall wall to neglibible movement

lh = 3500 x 1.5 = 5250 mm

Potential Movement Due to wall Excavation

Horizontal surface movement = 0.15% 5.25



DeltaH = 0.15% x 3500 = 5.25 mm

= 0.375 mm/m

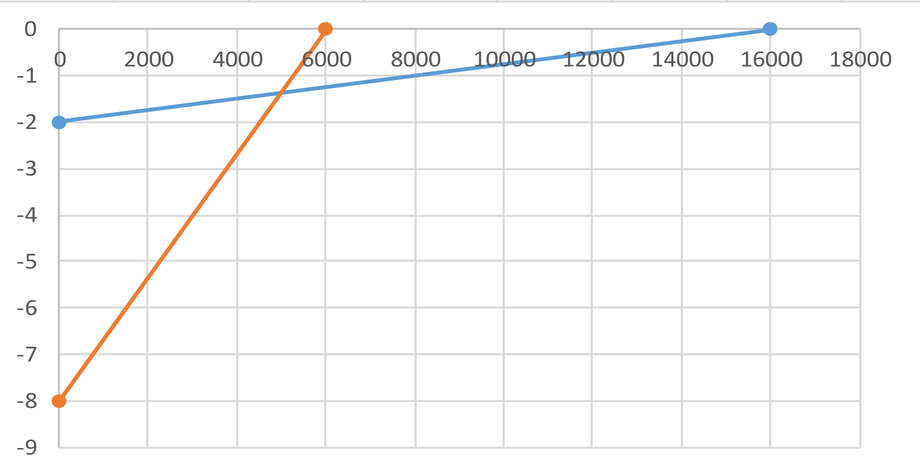
Vertical Surface Movement = 0.10%

Delta V = 0.10% x 3500 = 3.5 mm

Distance behind wall wall to neglibible movement

lh = 3500 x 4 = 14000 mm

		Excavation movement		Installation movement		
		Distance	delta V	Distance	delta V	
Nodes	x	16000	0	6000	0	
	y	0	-2	0	-8	



Determine Horizontal Movement

delta l =	8	mm	=	0.05%	
	16000	mm			

Table 2.4 CIRIA C580

Category of Damage	Normal Degree	Limiting Tensile Strain %		
0	Negligible	0.00%	-	0.05%
1	Very slight	0.05%	-	0.075%
2	Slight	0.075%	-	0.15%
3	Moderate	0.15%	-	0.30%
4 to 5	Severe to Very Server		>	0.30%
5				

Anticipated Damagae May be Categorised as **"Negligible to Slight Category 0-1"**

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	Any ground works pose an elevated risk to adjacent properties. The proposed works undermines the adjacent property along the party wall line: The party wall is to be underpinned. Underpinning the party wall will remove the risk of the movement to the adjacent property. The works must be carried out in accordance with the party wall act and condition surveys will be necessary at the beginning and end of the works. The method statement provided at the end of this report has been formulated with our experience of over 120 basements completed without error. The design of the retaining walls is completed to K_O lateral design stress values. This increase the design stresses on the concrete retaining walls an limits the overall deflection of the retaining wall. It is not expected that any cracking will occurring during the works. However our experience informs us that there is a risk of movement to the neighbours. To reduce the risk the development: • Employ a reputable firm for extensive knowledge of basement works. • Employ suitably qualified consultants. Croft Structural engineer has completed over 120 basements in the last 4 years. • Design the underpins to the stable without the need for elaborate temporary propping or needing the floor slab to be present. • Provide method statements for the contractors to follow • Investigate the ground, now completed. • Record and monitor the external properties. This is completed by a condition survey on under the Party Wall Act before and after the works are completed. See end of method statement. • Allow for unforeseen ground conditions: Loose ground is always a concern. The method statement and drawings show the use of precast lintels to areas of soft ground; this follows the guidance by the underpinning association.

<table>
<tr><td></td><td>With the above the maximum level of cracking anticipated is Hairline cracking which can be repaired with decorative cracking and can be repaired with decorative repairs. Under the party wall Act damage is allowed (although unwanted) to occur to a neighbouring property as long as repairs are suitability undertaken to rectify this. To mitigate this risk The Party Wall Act is to be followed and a Party Wall Surveyor will be appointed.</td></tr>
<tr><td>Burland Scale</td><td>Extract from The Institution of Structural Engineers "Subsidence of Low-Rise Buildings"
Table 6.2 Classification of visible damage to walls with particular reference to type of repair, and rectification consideration
<table>
<tr><th>Category of Damage</th><th>Approximate crack width</th><th>Limiting Tensile strain</th><th>Definitions of cracks and repair types/considerations</th></tr>
<tr><td>0</td><td>Up to 0.1</td><td>0.0-0.05</td><td><u>HAIRLINE</u> – Internally cracks can be filled or covered by wall covering, and redecorated. Externally, cracks rarely visible and remedial works rarely justified.</td></tr>
<tr><td>1</td><td>0.2 to 2</td><td><u>0.05-0.075</u></td><td><u>FINE</u> – Internally cracks can be filled or covered by wall covering, and redecorated. Externally, cracks may be visible, sometimes repairs required for weather tightness or aesthetics. NOTE: Plaster cracks may, in time, become visible again if not covered by a wall covering.</td></tr>
</table>
The anticipated damage Category for the new basement is 0- 1</td></tr>
</table>

Monitoring	
	Monitoring - In order to safeguard the existing structures during underpinning and new basement construction movement monitoring is to be undertaken.

Risk Assessment	Monitoring Level proposed	Type of Works.
	Monitoring 1 Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.	Cross wall removals, insertion of padstones Survey of LUL and Network Rail tunnels. Mass concrete, reinforced and Piled foundations to new build properties
	Monitoring 2 Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate.	Removal of lateral stability and insertion of new stability fames Removal of main masonry load bearing walls. Underpinning works less than 1.2m deep
	Monitoring 3 Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical monitoring movement by standard optical equipment	Underpinning works less than 3.0m deep in clays Basements up to 2.5m deep in clays
	Monitoring 4 Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.	New basements greater than 2.5m and shallower than 4m Deep in gravels Basements up to 4.5m deep in clays

	Inspection of the footing to ensure that the footings are stable and adequate.	Underpinning works to grade I listed building

Monitoring Conclusion

The level of Monitoring Croft recommend on 1B St Johns Wood Park is:

Monitoring 3

Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.
Inspection of the footing to ensure that the footings are stable and adequate. Vertical monitoring movement by standard optical equipment

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

- Risk Assessment to determine level of Monitoring
- Scope of Works
- Applicable standards
- Specification for Instrumentation
- Monitoring of Existing cracks on adjacent properties
- Monitoring of movement on adjacent properties
- Reporting
- Trigger Levels using a RED AMBER GREEN System

Recommend levels are

Movement	CATEGORY	ACTION
0mm-5mm	Green	No action required
5mm-12mm	AMBER	Crack Monitoring: Carry out a local structural review; Preparation for the implementation of remedial measures should be required.
>12mm	RED	Crack Monitoring: Implement structural support as required; Cease works with the exception of necessary works for the safety and stability of the structure and personnel; Review monitoring data and implement revised method of works

Basement Design & Construction Impacts	
Foundation type	Reinforced concrete cantilevered retaining walls The designs for the retaining walls have been calculated using software designed by TEDDS. The software is specifically designed for retaining walls and ensures the design is kept to a limit to prevent damage to the adjacent property. The overall stability of the walls are design using K_a & K_p values, while the design of the wall uses K_o values. This approach minimise the level of movement from the concrete affecting the adjacent properties. The Investigations have highlight that water is a present. The walls are designed to cope with the hydrostatic pressure. The water table was low. The design of the walls however considers the long term items. It is possible that a water main may break causing local high water table. To account for this the wall is designed for water 1m from the top of the wall. The Design also considers floatation as a risk. The design of has considered the weight of the building and the uplift forces from the water. The weight of the building is greater than the uplift resulting in a stable structure.
Roads	The basement must be designed for Yes. Site is within 5m of the footpath/alleyway and the road surface is further than 5m from the front lightwell. Highways loading allow: 10kN/m2 if within 45° of road 100kN point loads if under road or with in 1.5m 5kN/m2 if within 45° of Pavement Garden Surcharge 2.5kN/m2 Surcharge for adjacent property 1.5kN/m2 + 4kN/m2 for concrete ground bearing slab
Intended use of structure and user requirements	Family/domestic use

Loading Requirements (EC1-1)	

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

The basement does not line within a 45º angle of the highway. Therefore Highways HA loading is not required to be applied.

Part A3 Progressive collapse

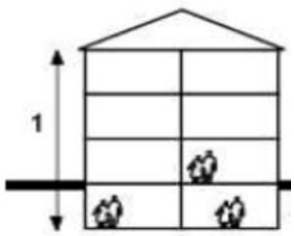
Number of Storeys 4

Is the Building Multi Occupancy? No

Class 1	Single occupancy houses not exceeding 4 storeys

To NHBC guidance compliance is only required to other floors if a material change of use occurs to the property.

Initial Building Class	1
Proposed Building Class	1
If class has changed material change has occurred	No



3 storey over basement

Lateral Stability

Exposure and wind loading conditions

Basic wind speed Vb = 21 m/s to EC1-2
Topography not considered significant.

Stability Design

The cantilevered walls are suitable to carry the lateral loading applied from above

Lateral Actions

The soil loads apply a lateral load on the retaining walls.

	Hydrostatic pressure will be applied to the wall Imposed loading will surcharge the wall.
Retained soil Parameters	Design overall stability to K_a & K_p values. Lateral movement necessary to achieve K_a mobilisation is height/500 (from Tomlinson). This is tighter than the deflection limits of the concrete wall.
Water Table	Has a soil investigation been carried out Yes Known water table from boreholes Design temporary condition for water table level, If deeper than basement ignore Design Permanent condition for water table level: If deeper than existing, design reinforcement for water table at full basement depth to allow for local failure of water mains, drainage and storm water. Global uplift forces can be ignored when water table lower than basement. BS8102 only indicates guidance.
Drainage and Damp Waterproofing	Assumed that drainage and damp proofing is by others: Details are not provided within our brief. It is recommended that a water proofing specialist is employed to ensure all the water proofing requirements are met. Croft structural engineers are not the waterproofing designer nor act as the structural waterproof designer. Croft are not the structural waterproofer. The waterproofing specialist must name who is their structural waterproofer. The Structural waterproofer must inspect the structural details and confirm that are happy with the robustness. Due to the construction nature of the segmental basement it is not possible to water proof the joints. All water proofing must be made by the waterproofing specialist. They should make review of our details and recommend to us if water bars and stops are necessary. The waterproof design must not assume that the structure is watertight. To help reduce water floor through joints in the segmental pins all faces should be; • Cleaned of all debris and detritus • Faces between pins should be needle hammered to improve key • All pipe work and other penetrations should have puddle flanges or hydrophilic strips
Localised Dewatering	Localised dewater to pins may be necessary.

	Some engineers may raise the theoretical questions about pumping of water causing localised settlement. We believe that this argument is a red herring when applied to single storey basements and our reason for stating this is: • The water table in the area is variable, • The water level naturally rises and falls over time and does not lead to subsidence • The water table has naturally been rising and falling for over the last 20,000 years, any fines that will have been removed from the soil would have done so already. • If the water table rises and falls naturally why does this not cause subsidence due to fine removals every year? It does not because the soil has been soil is naturally consolidated by the rise and fall of the water table in the area. • The effect of local pumping for small excavations will not affect the local area. • There is only a risk of subsidence from large scale pumping of soil which lowers the water table below is natural lowest level.
Temporary Works	Walls are designed to be temporarily stable. Temporary propping details will be required for the ground and soil and this must be provided by the contractor. Their details should be forwarded to Croft Structural Engineers. Particular attention should be paid to the point loads from above. Critical areas where point loads are present from above Cross wall Chimney Stack Door openings
Geological Assessment of Land Stability	Has the retaining wall design been assessed by a Chartered Geological Engineer? Yes inspected see supplementary report.

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Retaining Wall Calculation

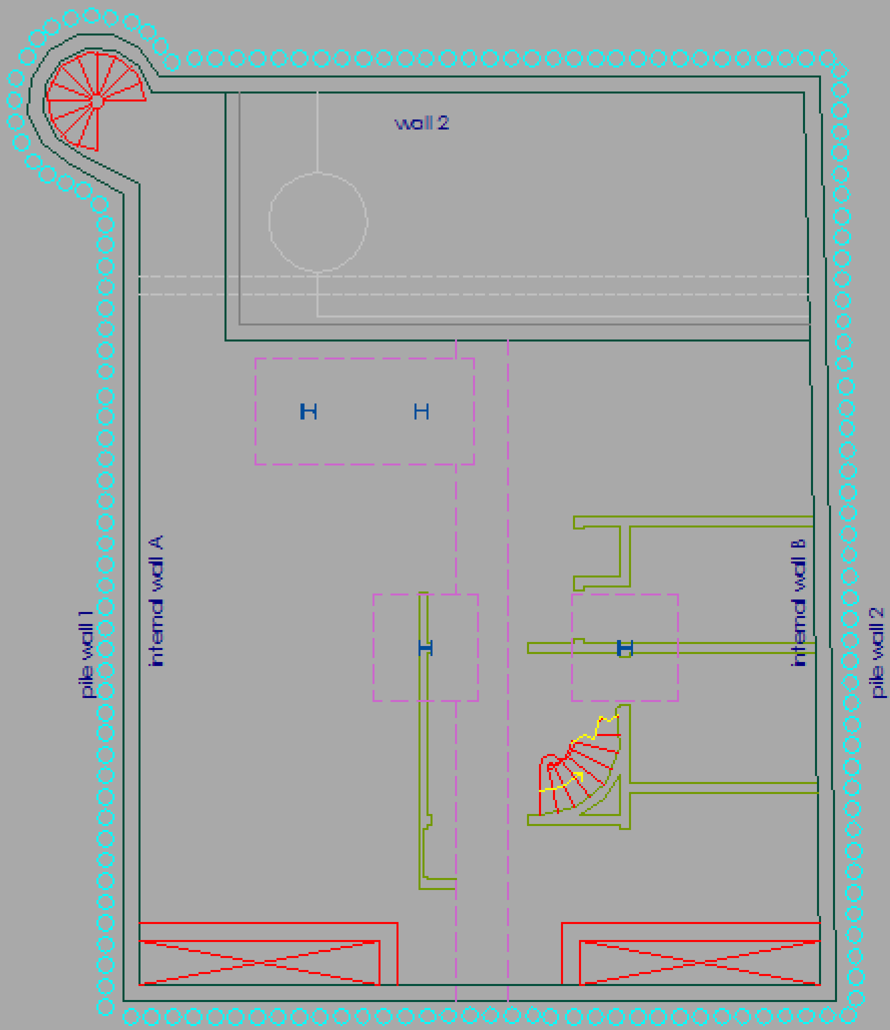
Reference

General Loadings

			Cavity Walls				
Sloped Roof			100 Facing Brick =	2.2		**Timber Partitions**	
Slate =	0.6	kN/m²	100 Block (16kN/m3)=	1.6		50x100 Studs @ 400 =	0.15
Battens =	0.02		Plaster & Skim =	0.18		Insulation =	0.04
Rafers	0.1125		Dead Load =	**3.98**	**kN/m2**	Plaster & Skim =	0.36
Felt =	0.02					**Dead Load =**	**0.55**
Insulation =	0.02		**Internal Walls**				
Plaster=	0.18		100 Block (20kN/m3)=	2			
	0.9525	**kN/m2**	Plaster & Skim =	0.36		**Existing Brick Walls**	
Roof Angle =	25	deg	**Dead Load =**	**2.36**	**kN/m2**	225 Facing Brick =	4.5
Plan Dead load =	**1.051**	**kN/m2**	**Existing Internal Walls**				
Live Load =	**0.6**	**kN/m2**	100 Brick (20kN/m3)=	2.1		Plaster & Lathe =	0.15
			Plaster & Skim =	0.36		**Dead Load =**	**4.65**
Flat Roof			**Dead Load =**	**2.46**	**kN/m2**		
20mm Asphalt =	0.46					**Beam & Block Ground Floors**	
Felt underlay =	0.02		**Timber Floors**			Beam & Block	3.1
insulation =	0.04		18mm Ply	0.15		Screed	1.4
Ply Sheeting =	0.1		Joists 50x225@400 =	0.16875		Insulation	0.07
Firring =	0.1		100 Insulation =	0.05		Finishes	0.05
of joists 50x200@400 =	0.15		Plaster & Skim =	0.18		**Dead Load =**	4.62
Plaster & Skim =	0.18		**Dead Load =**	**0.54875**	**kN/m2**	**Live Load =**	1.5
Plan Dead load =	**1.05**	**kN/m2**	**Live Load =**	**1.5**	**kN/m2**		
Live Load =	**0.75**	**kN/m2**	**Terrace Floor**			**Standing Seam**	
			Promonade Tiles =	0.4		Roof Sheet	0.08
Mansard Roof			20mm Asphalt =	0.46		Insulation	0.07
Slate Tiles =	0.4		Felt underlay =	0.02		Decking	0.2
Battens =	0.02		insulation =	0.04		Steelwork	0.6
Ply Sheeting =	0.125		Ply Sheeting =	0.1		**Dead Load =**	**0.95**
Rafters =	0.125		Firring =	0.1		**Live Load =**	**0.6**
100 Insulation =	0.06		Roof joists 50x200@400 =	0.175			
plaster & Skim =	0.18		Plaster & Skim =	0.18		**Filler joist Floor**	
Felt =	0.02		**Dead Load =**	**1.475**	**kN/m2**	Finishes	1.2
	0.93		**Live Load =**	**1.5**	**kN/m2**	Filler Joist Floor	2.5
			Ceiling			Ceiling	0.18
Roof Angle =	45	deg	50x100 Joists =	0.075		Steel	0.3
Plan Dead load =	**1.316**	**kN/m2**	100 Insulation =	0.06		**Dead Load =**	**4.18**
Live Load =	**0.3**	**kN/m2**	Plaster & Skim =	0.18		**Live Load =**	**3.5**
			Dead Load =	**0.315**	**kN/m2**		
Precast Floor on Steel			**Live Load =**	**0.25**	**kN/m2**		
200PC Floor units =	3.6						
60 Screed =	1.2						
Finishes =	0.1						
Steelwork =	0.6						
Dead Load =	**5.5**	**kN/m2**					
Live Load =	**3**	**kN/m2**					

Table 3 Live Load Reduction					
Area	0	0%	Floors	1	0%
	50	5%		2	10%
	100	10%		3	20%
	150	15%		4	30%
	200	20%		5 to 10	40%

Reference	basement plan									
Location	Area			Type	L	Load	Load kN			
	L	W	m2			kN/m2	Dead	%	Live	Total
internal wall A										
roof DL	3.2	1.0	3.2	g_k		1.05	3.4			
roof LL				q_k		0.75			2.4	
2nd fl DL	3.2	1.0	3.2	g_k		0.63	2.0			
2nd fl LL				q_k		1.50			4.8	
partitions DL	2.7	1.0	2.7	g_k		1.05	2.8			
1st fl DL	3.2	1.0	3.2	g_k		0.63	2.0			
1st fl LL				q_k		1.50			4.8	
partitions DL	3.0	1.0	3.0	g_k		1.05	3.2			
ground fl DL	3.2	1.0	3.2	g_k		4.62	14.8			
ground fl LL				q_k		1.50			4.8	
partitions DL	3.0	1.0	3.0	g_k		1.05	3.2			
							31.3	kN/m	16.8	kN/m
internal wall B										
roof DL	3.2	1.0	3.2	g_k		1.05	3.4			
roof LL				q_k		0.75			2.4	
2nd fl DL	3.2	1.0	3.2	g_k		0.63	2.0			
2nd fl LL				q_k		1.50			4.8	
partitions DL	2.7	1.0	2.7	g_k		1.05	2.8			
1st fl DL	3.2	1.0	3.2	g_k		0.63	2.0			
1st fl LL				q_k		1.50			4.8	
partitions DL	3.0	1.0	3.0	g_k		1.05	3.2			
ground fl DL	3.2	1.0	3.2	g_k		4.62	14.8			
ground fl LL				q_k		1.50			4.8	
partitions DL	3.0	1.0	3.0	g_k		1.05	3.2			
							31.3	kN/m	16.8	kN/m
wall 2										
ground fl DL	3.2	1.0	3.2	g_k		4.62	14.8			
ground fl LL				q_k		1.50			4.8	
partitions DL	3.0	1.0	3.0	g_k		1.05	3.2			
							17.9	kN/m	4.8	kN/m



Basement plan

Scale (1:100)

CROFT STRUCTURAL ENGINEERS

PILE WALL 1 (WITHOUT WATER)

RETAINING WALL ANALYSIS & DESIGN (EN1992/EN1996/EN1997)

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 2.6.04

Retaining wall details

Stem type	Propped cantilever		
Stem height	h_{stem} = **4000** mm		
Prop height	h_{prop} = **3600** mm		
Stem thickness	t_{stem} = **300** mm		
Angle to rear face of stem	α = **90** deg		
Stem density	γ_{stem} = **25** kN/m^3		
Toe length	l_{toe} = **1200** mm		
Base thickness	t_{base} = **400** mm		
Base density	γ_{base} = **25** kN/m^3		
Height of retained soil	h_{ret} = **4000** mm	Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm		
Height of water	h_{water} = **0** mm		
Water density	γ_w = **9.8** kN/m^3		

Retained soil properties

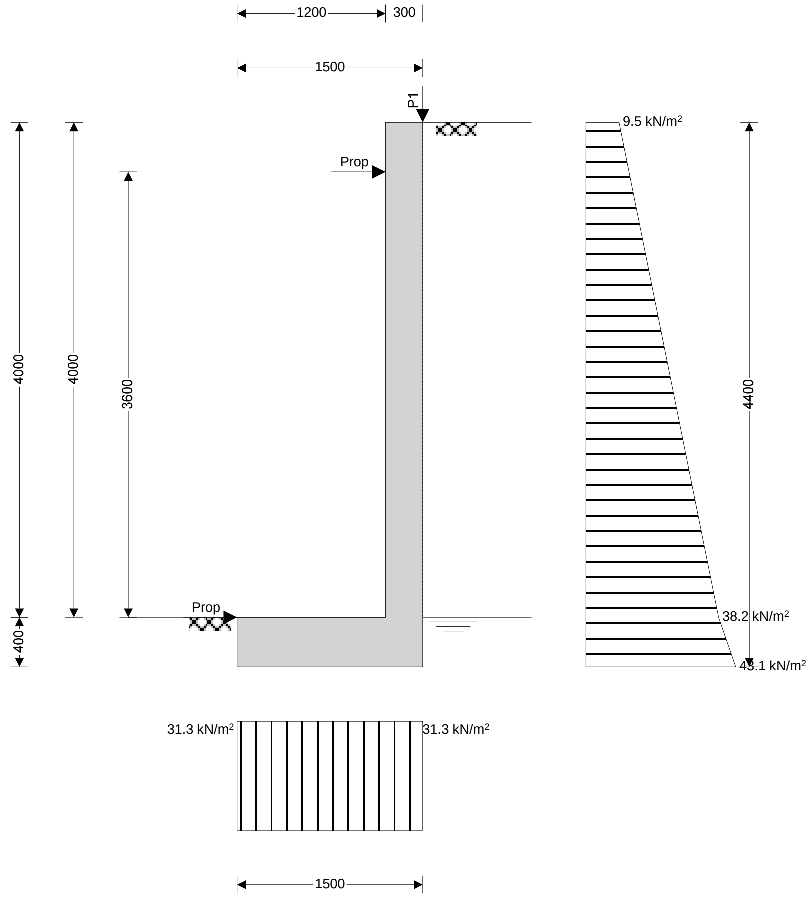
Soil type	Organic clay	
Moist density	γ_{mr} = **15** kN/m^3	
Saturated density	γ_{sr} = **15** kN/m^3	
Characteristic effective shear resistance angle		$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **9** deg	

Base soil properties

Soil type	Medium dense well graded sand	
Moist density	γ_{mb} = **21** kN/m^3	
Characteristic effective shear resistance angle		$\phi'_{b.k}$ = **30** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **15** deg	
Characteristic base friction angle		$\delta_{bb.k}$ = **30** deg
Presumed bearing capacity	$P_{bearing}$ = **150** kN/m^2	

Loading details

Permanent surcharge load	$Surcharge_G$ = **10** kN/m^2
Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 1500 mm	P_{G1} = **1** kN/m
	P_{Q1} = **1** kN/m



Calculate retaining wall geometry

Base length	l_{base} = **1500** mm		
Saturated soil height	h_{sat} = **0** mm		
Moist soil height	h_{moist} = **4000** mm		
Length of surcharge load	l_{sur} = **0** mm		
Vertical distance	x_{sur_v} = **1500** mm		
Effective height of wall	h_{eff} = **4400** mm		
Horizontal distance	x_{sur_h} = **2200** mm		
Area of wall stem	A_{stem} = **1.2** m^2	Vertical distance	x_{stem} = **1350** mm
Area of wall base	A_{base} = **0.6** m^2	Vertical distance	x_{base} = **750** mm

Using Coulomb theory

Active pressure coefficient	K_A = **0.483**	Passive pressure coefficient	K_P = **4.977**

Bearing pressure check

Vertical forces on wall

Total $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} + F_{P\_v}$ = **47** kN/m

Horizontal forces on wall

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **103.6** kN/m

Moments on wall

Total $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P$ = **-139.3** kNm/m

Check bearing pressure

Propping force to stem	F_{prop_stem} = **43.6** kN/m	Propping force to base	F_{prop_base} = **60** kN/m
Bearing pressure at toe	q_{toe} = **31.3** kN/m^2	Bearing pressure at heel	q_{heel} = **31.3** kN/m^2
Factor of safety	FoS_{bp} = **4.787**		

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

CROFT STRUCTURAL ENGINEERS

RETAINING WALL DESIGN

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

Tedds calculation version 2.6.04

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

Concrete strength class C28/35

Char.comp.cylinder strength	f_{ck} = **28** N/mm^2	Mean axial tensile strength	f_{ctm} = **2.8** N/mm^2
Secant modulus of elasticity	E_{cm} = **32308** N/mm^2	Maximum aggregate size	h_{agg} = **20** mm
Design comp.concrete strength	f_{cd} = **15.9** N/mm^2	Partial factor	γ_C = **1.50**

Reinforcement details

Characteristic yield strength	f_{yk} = **500** N/mm^2	Modulus of elasticity	E_s = **200000** N/mm^2
Design yield strength	f_{yd} = **435** N/mm^2	Partial factor	γ_S = **1.15**

Cover to reinforcement

Front face of stem	c_{sf} = **40** mm	Rear face of stem	c_{sr} = **50** mm
Top face of base	c_{bt} = **50** mm	Bottom face of base	c_{bb} = **75** mm

Check stem design at 1915 mm

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **28.6** kNm/m K = **0.018** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{sfM.req}$ = **289** mm^2/m

Tens.reinforcement provided 16 dia.bars @ 200 c/c Tens.reinforcement provided $A_{sfM.prov}$ = **1005** mm^2/m

Min.area of reinforcement $A_{sfM.min}$ = **345** mm^2/m Max.area of reinforcement $A_{sfM.max}$ = **12000** mm^2/m

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

Deflection control - Section 7.4

Limiting span to depth ratio 228 Actual span to depth ratio 15

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.095** mm

PASS - Maximum crack width is less than limiting crack width**Check stem design at base of stem**

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **57.7** kNm/m K = **0.035** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{sr.req}$ = **578** mm^2/m

Tens.reinforcement provided 16 dia.bars @ 100 c/c Tens.reinforcement provided $A_{sr.prov}$ = **2011** mm^2/m

Min.area of reinforcement $A_{sr.min}$ = **348** mm^2/m Max.area of reinforcement $A_{sr.max}$ = **12000** mm^2/m

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

Deflection control - Section 7.4

Limiting span to depth ratio 77 Actual span to depth ratio 14.9

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.074** mm

PASS - Maximum crack width is less than limiting crack width**Rectangular section in shear - Section 6.2**

Design shear force V = **88.9** kN/m Design shear resistance $V_{Rd.c}$ = **118.2** kN/m

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **1.2** kNm/m K = **0.001** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{sr1.req}$ = **12** mm^2/m

Tens.reinforcement provided 16 dia.bars @ 200 c/c Tens.reinforcement provided $A_{sr1.prov}$ = **1005** mm^2/m

Min.area of reinforcement $A_{sr1.min}$ = **348** mm^2/m Max.area of reinforcement $A_{sr1.max}$ = **12000** mm^2/m

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

Deflection control - Section 7.4

Limiting span to depth ratio 11682 Actual span to depth ratio 1.7

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.004** mm

PASS - Maximum crack width is less than limiting crack width**Rectangular section in shear - Section 6.2**

Design shear force V = **36.6** kN/m Design shear resistance $V_{Rd.c}$ = **118.2** kN/m

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Min.area of reinforcement $A_{sx.req}$ = **503** mm^2/m Max.spacing of reinforcement s_{sx_max} = **400** mm

Trans.reinforcement provided 12 dia.bars @ 200 c/c Trans.reinforcement provided $A_{sx.prov}$ = **565** mm^2/m

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

Check base design at toe

Depth of section h = **400** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **20.8** kNm/m K = **0.007** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{bb.req}$ = **159** mm^2/m

Tens.reinforcement provided 16 dia.bars @ 200 c/c Tens.reinforcement provided $A_{bb.prov}$ = **1005** mm^2/m

Min.area of reinforcement $A_{bb.min}$ = **456** mm^2/m Max.area of reinforcement $A_{bb.max}$ = **16000** mm^2/m

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.088** mm

PASS - Maximum crack width is less than limiting crack width**Rectangular section in shear - Section 6.2**

Design shear force V = **34.7** kN/m Design shear resistance $V_{Rd.c}$ = **141.3** kN/m

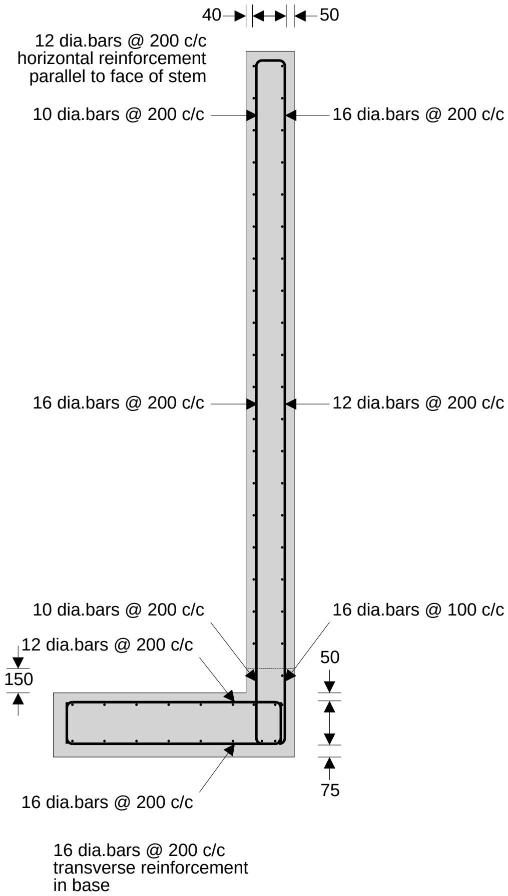
PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Min.area of reinforcement $A_{bx.req}$ = **201** mm^2/m Max.spacing of reinforcement s_{bx_max} = **450** mm

Trans.reinforcement provided 16 dia.bars @ 200 c/c Trans.reinforcement provided $A_{bx.prov}$ = **1005** mm^2/m

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


40
50
12 dia.bars @ 200 c/c
horizontal reinforcement
parallel to face of stem
10 dia.bars @ 200 c/c
16 dia.bars @ 200 c/c
16 dia.bars @ 200 c/c
12 dia.bars @ 200 c/c
10 dia.bars @ 200 c/c
16 dia.bars @ 100 c/c
12 dia.bars @ 200 c/c
150
50
75
16 dia.bars @ 200 c/c
16 dia.bars @ 200 c/c
transverse reinforcement
in base

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PILED WALL 1 (WITH WATER)

RETAINING WALL ANALYSIS & DESIGN (EN1992/EN1996/EN1997)

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 2.6.04

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **4000** mm
Prop height	h_{prop} = **3600** 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} = **1200** mm
Base thickness	t_{base} = **400** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **4000** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **4000** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Soil type	Organic clay
Moist density	γ_{mr} = **15** kN/m^3
Saturated density	γ_{sr} = **15** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **9** deg

Base soil properties

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

Loading details

Permanent surcharge load	$Surcharge_G$ = **10** kN/m^2
Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 1500 mm	P_{G1} = **1** kN/m
	P_{Q1} = **1** kN/m

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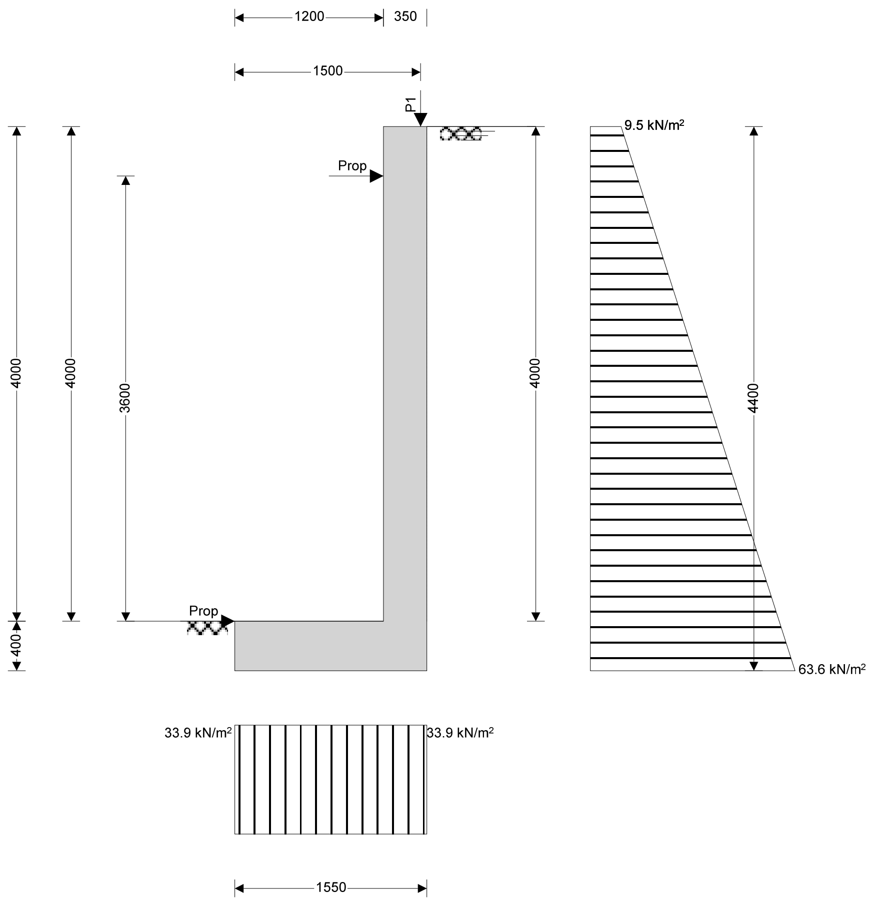

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} =$ **1550** mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} =$ **4000** mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} =$ **0** mm
Length of surcharge load	$l_{sur} = l_{heel} =$ **0** mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **1550** mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **4400** mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 =$ **2200** mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} =$ **1.4** m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 =$ **1375** mm
Area of wall base	$A_{base} = l_{base} \times t_{base} =$ **0.62** m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 =$ **775** mm

Using Coulomb theory

Active pressure coefficient	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) =$ **0.483**
Passive pressure coefficient	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) =$ **4.977**

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **35** kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **15.5** kN/m
Line loads	$F_{P\_v} = P_{G1} + P_{Q1} =$ **2** kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} + F_{P\_v} =$ **52.5** kN/m

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Horizontal forces on wall

Surcharge load — $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times (Surcharge_G + Surcharge_Q) \times h_{eff}$ = **42** kN/m

Saturated retained soil — $F_{sat\_h} = K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2$ = **24** kN/m

Water — $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **95** kN/m

Moist retained soil — $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **0** kN/m

Base soil — $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_{mb} \times (d_{cover} + h_{base})^2 / 2$ = **-6.9** kN/m

Total — $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **154** kN/m

Moments on wall

Wall stem — $M_{stem} = F_{stem} \times x_{stem}$ = **48.1** kNm/m

Wall base — $M_{base} = F_{base} \times x_{base}$ = **12** kNm/m

Surcharge load — $M_{sur} = -F_{sur\_h} \times x_{sur\_h}$ = **-92.4** kNm/m

Line loads — $M_P = (P_{G1} + P_{Q1}) \times p_1$ = **3** kNm/m

Saturated retained soil — $M_{sat} = -F_{sat\_h} \times x_{sat\_h}$ = **-35.2** kNm/m

Water — $M_{water} = -F_{water\_h} \times x_{water\_h}$ = **-139.3** kNm/m

Moist retained soil — $M_{moist} = -F_{moist\_h} \times x_{moist\_h}$ = **0** kNm/m

Total — $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P$ = **-203.7** kNm/m

Check bearing pressure

Propping force to stem — $F_{prop\_stem} = \min((F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}), F_{total\_h})$ = **61.1** kN/m

Propping force to base — $F_{prop\_base} = F_{total\_h} - F_{prop\_stem}$ = **92.9** kN/m

Moment from propping force — $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base})$ = **244.4** kNm/m

Distance to reaction — $\bar{x} = l_{base} / 2$ = **775** mm

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

Loaded length of base — $l_{load} = l_{base}$ = **1550** mm

Bearing pressure at toe — $q_{toe} = F_{total\_v} / l_{base}$ = **33.9** kN/m^2

Bearing pressure at heel — $q_{heel} = F_{total\_v} / l_{base}$ = **33.9** kN/m^2

Factor of safety — $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **4.429**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

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

Tedds calculation version 2.6.04

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

Concrete strength class — C28/35

Characteristic compressive cylinder strength — f_{ck} = **28** N/mm^2

Characteristic compressive cube strength — $f_{ck,cube}$ = **35** N/mm^2

Mean value of compressive cylinder strength — $f_{cm} = f_{ck} + 8$ N/mm^2 = **36** N/mm^2

Mean value of axial tensile strength — $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **2.8** N/mm^2

5% fractile of axial tensile strength — $f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **1.9** N/mm^2

Secant modulus of elasticity of concrete — $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **32308** N/mm^2

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

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

Design compressive concrete strength - exp.3.15 — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **15.9** N/mm^2

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

Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement	E_s = **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Cover to reinforcement

Front face of stem	c_{sf} = **40** mm
Rear face of stem	c_{sr} = **50** mm
Top face of base	c_{bt} = **50** mm
Bottom face of base	c_{bb} = **75** mm

Check stem design at 1893 mm

Depth of section	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **40.7** kNm/m
Depth to tension reinforcement	$d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2$ = **295** mm
	$K = M / (d^2 \times f_{ck})$ = **0.017**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **280** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **37** mm
Area of tension reinforcement required	$A_{sfM.req} = M / (f_{yd} \times z)$ = **334** mm^2/m
Tension reinforcement provided	10 dia.bars @ 100 c/c
Area of tension reinforcement provided	$A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM})$ = **785** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **424** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sfM.max} = 0.04 \times h$ = **14000** mm^2/m
	$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov}$ = **0.54**

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

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000$ = **0.005**
Required tension reinforcement ratio	$\rho = A_{sfM.req} / d$ = **0.001**
Required compression reinforcement ratio	$\rho' = A_{sfM.2.req} / d_2$ = **0.000**
Structural system factor - Table 7.4N	K_b = **1**
Reinforcement factor - exp.7.17	$K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sfM.req} / A_{sfM.prov}), 1.5)$ = **1.5**
Limiting span to depth ratio - exp.7.16.a	$K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **251.4**
Actual span to depth ratio	h_{prop} / d = **12.2**

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

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **28** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sfM.prov} \times z)$ = **127.2** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2)$ = **104375** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.8** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sfM.prov} / A_{c.eff}$ = **0.008**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.19**

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Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff}$ = **362** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.138** mm
	w_k / w_{max} = **0.46**

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **83.7** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2$ = **292** mm
	$K = M / (d^2 \times f_{ck})$ = **0.035**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **277** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **37** mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z)$ = **694** mm^2/m
Tension reinforcement provided	16 dia.bars @ 100 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **2011** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **420** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h$ = **14000** mm^2/m
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.345**

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

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000$ = **0.005**
Required tension reinforcement ratio	$\rho = A_{sr.req} / d$ = **0.002**
Required compression reinforcement ratio	$\rho' = A_{sr.2.req} / d_2$ = **0.000**
Structural system factor - Table 7.4N	K_b = **1**
Reinforcement factor - exp.7.17	$K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**
Limiting span to depth ratio - exp.7.16.a	$K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **77.5**
Actual span to depth ratio	h_{prop} / d = **12.3**

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

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **58.2** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **104.3** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **104500** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.8** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.019**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.19**
Bond property coefficient	k_1 = **0.8**

Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **311** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.097** mm
	w_k / w_{max} = **0.325**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **131** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.828**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sf.prov} / d, 0.02)$ = **0.003**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.458** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **133.6** kN/m
	$V / V_{Rd.c}$ = **0.980**

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **1.3** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr1} / 2$ = **294** mm
	$K = M / (d^2 \times f_{ck})$ = **0.001**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **279** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **37** mm
Area of tension reinforcement required	$A_{sr1.req} = M / (f_{yd} \times z)$ = **10** mm^2/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sr1.prov} = \pi \times \phi_{sr1}^2 / (4 \times s_{sr1})$ = **565** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{sr1.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **423** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr1.max} = 0.04 \times h$ = **14000** mm^2/m
	$\max(A_{sr1.req}, A_{sr1.min}) / A_{sr1.prov}$ = **0.748**

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

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000$ = **0.005**
Required tension reinforcement ratio	$\rho = A_{sr1.req} / d$ = **0.000**
Required compression reinforcement ratio	$\rho' = A_{sr1.2.req} / d_2$ = **0.000**
Structural system factor - Table 7.4N	K_b = **0.4**
Reinforcement factor - exp.7.17	$K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr1.req} / A_{sr1.prov}), 1.5)$ = **1.5**
Limiting span to depth ratio - exp.7.16.a	$K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **19079.5**
Actual span to depth ratio	$(h_{stem} - h_{prop}) / d$ = **1.4**

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

Crack control - Section 7.3

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

Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **0.7** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr1.prov} \times z)$ = **4.7** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2)$ = **104417** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.8** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr1.prov} / A_{c.eff}$ = **0.005**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.19**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr1} / \rho_{p.eff}$ = **547** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.008** mm
	w_k / w_{max} = **0.026**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **49.3** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.825**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sf1.prov} / d, 0.02)$ = **0.001**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.457** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **134.2** kN/m
	$V / V_{Rd.c}$ = **0.368**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **503** mm^2/m
Maximum spacing of reinforcement – cl.9.6.3(2)	s_{sx_max} = **400** mm
Transverse reinforcement provided	10 dia.bars @ 100 c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **785** mm^2/m

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

Check base design at toe

Depth of section	h = **400** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **23.3** kNm/m
Depth to tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **319** mm
	$K = M / (d^2 \times f_{ck})$ = **0.008**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **303** mm
Depth of neutral axis	$x = 2.5 \times (d – z)$ = **40** mm
Area of tension reinforcement required	$A_{bb.req} = M / (f_{yd} \times z)$ = **177** mm^2/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **565** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **459** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bb.max} = 0.04 \times h =$ **16000** mm^2/m
	$max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} =$ **0.811**

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

Crack control - Section 7.3

Limiting crack width	$w_{max} =$ **0.3** mm
Variable load factor - EN1990 – Table A1.1	$\psi_2 =$ **0.6**
Serviceability bending moment	$M_{sls} =$ **17.2** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bb.prov} \times z) =$ **100.3** N/mm^2
Load duration	Long term
Load duration factor	$k_t =$ **0.4**
Effective area of concrete in tension	$A_{c.eff} = min(2.5 \times (h - d), (h – x) / 3, h / 2) =$ **120042** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} =$ **2.8** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff} =$ **0.005**
Modular ratio	$\alpha_e = E_s / E_{cm} =$ **6.19**
Bond property coefficient	$k_1 =$ **0.8**
Strain distribution coefficient	$k_2 =$ **0.5**
	$k_3 =$ **3.4**
	$k_4 =$ **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} =$ **688** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k =$ **0.207** mm
	$w_k / w_{max} =$ **0.69**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V =$ **38.8** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**
	$k = min(1 + \sqrt{(200 mm / d)}, 2) =$ **1.792**
Longitudinal reinforcement ratio	$\rho_l = min(A_{bb.prov} / d, 0.02) =$ **0.002**
	$v_{min} = 0.035 N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} =$ **0.444** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = max(C_{Rd.c} \times k \times (100 N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} =$ **141.7** kN/m
	$V / V_{Rd.c} =$ **0.274**

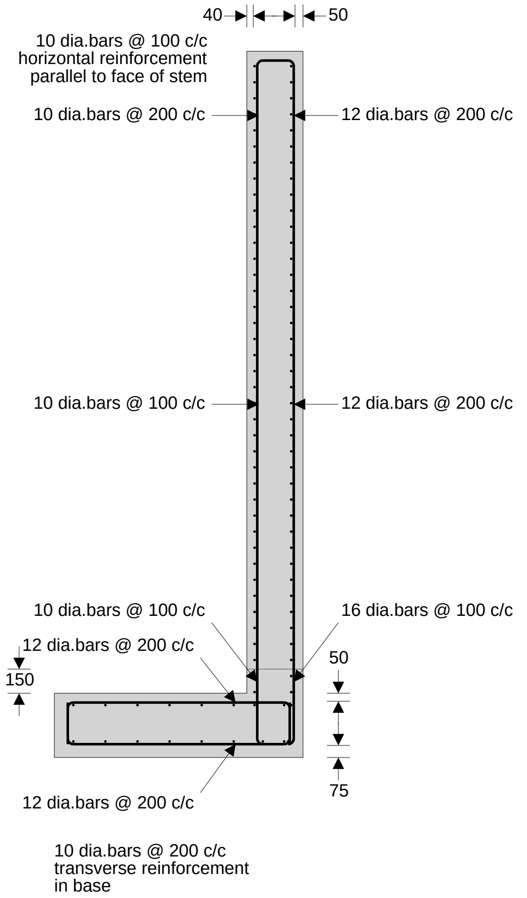
PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

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

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

CROFT STRUCTURAL ENGINEERS



CAPPING BEAM

Propping force = 61.1kN/m

Try 450x450Dp RC beam

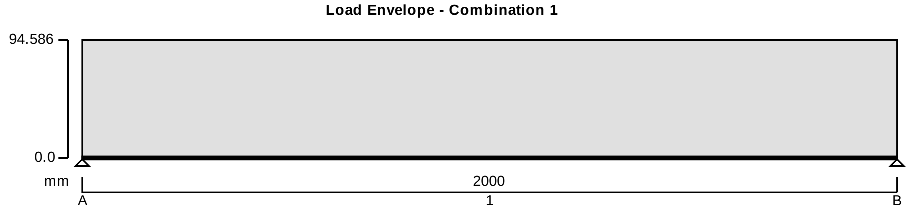
RC BEAM ANALYSIS & DESIGN (EN1992)

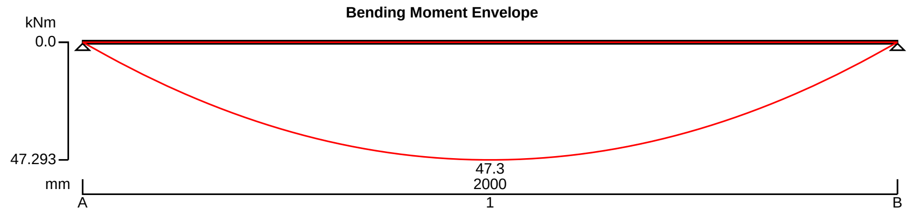
RC BEAM ANALYSIS & DESIGN (EN1992-1)

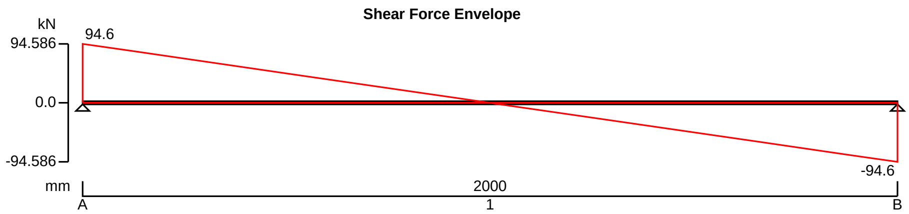
In accordance with UK national annex

TEDDS calculation version 2.1.15

CROFT STRUCTURAL ENGINEERS







Support conditions

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

Applied loading

	Permanent self weight of beam × 1	
	Permanent full UDL 65 kN/m	

Load combinations

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

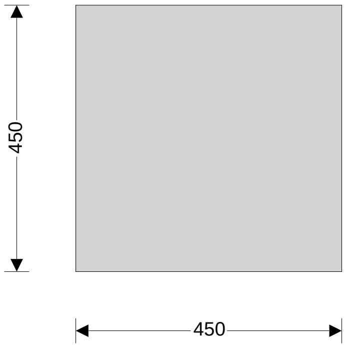
Analysis results

Maximum moment support A	M_{A_max} = **0** kNm	M_{A_red} = **0** kNm
Maximum moment span 1 at 1000 mm	M_{s1_max} = **47** kNm	M_{s1_red} = **47** kNm
Maximum moment support B	M_{B_max} = **0** kNm	M_{B_red} = **0** kNm
Maximum shear support A	V_{A_max} = **95** kN	V_{A_red} = **95** kN
Maximum shear support A span 1 at 397 mm	$V_{A\_s1\_max}$ = **57** kN	$V_{A\_s1\_red}$ = **57** kN
Maximum shear support B	V_{B_max} = **-95** kN	V_{B_red} = **-95** kN
Maximum shear support B span 1 at 1603 mm	$V_{B\_s1\_max}$ = **-57** kN	$V_{B\_s1\_red}$ = **-57** kN
Maximum reaction at support A	R_A = **95** kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **70** kN	

Maximum reaction at support B	R_B = **95** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **70** kN

Rectangular section details

Section width	b = **450** mm
Section depth	h = **450** mm



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

Concrete strength class	**C28/35**
Characteristic compressive cylinder strength	f_{ck} = **28** N/mm^2
Characteristic compressive cube strength	$f_{ck,cube}$ = **35** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck}$ + 8 N/mm^2 = **36** N/mm^2
Mean value of axial tensile strength	f_{ctm} = 0.3 N/mm^2 $\times$ (f_{ck}/ 1 N/mm^2)$^{2/3}$ = **2.8** N/mm^2
Secant modulus of elasticity of concrete	E_{cm} = 22 kN/mm^2 $\times$ [f_{cm}/10 N/mm^2]$^{0.3}$ = **32308** N/mm^2
Partial factor for concrete (Table 2.1N)	γ_C = **1.50**
Compressive strength coefficient (cl.3.1.6(1))	α_{cc} = **0.85**
Design compressive concrete strength (exp.3.15)	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **15.9** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm

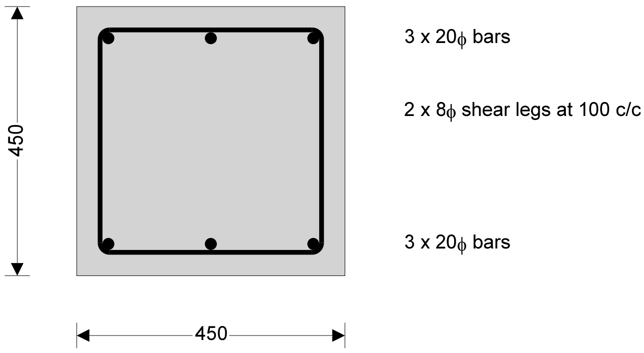
Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Partial factor for reinforcing steel (Table 2.1N)	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Nominal cover to reinforcement

Nominal cover to top reinforcement	c_{nom_t} = **35** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **35** mm
Nominal cover to side reinforcement	c_{nom_s} = **35** mm

<u>Support A</u>



Rectangular section in flexure (Section 6.1)

Minimum moment factor (cl.9.2.1.2(1))	β_1 = **0.25**

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Design bending moment	$M = \max(\text{abs}(M_{A\_red}), \beta_1 \times \text{abs}(M_{s1\_red}))$ = **12** kNm
Depth to tension reinforcement	$d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2$ = **397** mm
Percentage redistribution	m_{rA} = **0** %
Redistribution ratio	$\delta = \min(1 - m_{rA}, 1)$ = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.006**
	$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **377** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **50** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **72** mm^2
Tension reinforcement provided	$3 \times 20\phi$ bars
Area of tension reinforcement provided	$A_{s,prov}$ = **942** mm^2
Minimum area of reinforcement (exp.9.1N)	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **257** mm^2
Maximum area of reinforcement (cl.9.2.1.1(3))	$A_{s,max} = 0.04 \times b \times h$ = **8100** mm^2

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

Minimum bottom reinforcement at supports

Minimum reinforcement factor (cl.9.2.1.4(1))	β_2 = **0.25**
Area of reinforcement to adjacent span	$A_{s,span}$ = **942** mm^2
Minimum bottom reinforcement to support	$A_{s2,min} = \beta_2 \times A_{s,span}$ = **236** mm^2
Bottom reinforcement provided	$3 \times 20\phi$ bars
Area of bottom reinforcement provided	$A_{s2,prov}$ = **942** mm^2

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

Rectangular section in shear (Section 6.2)

Design shear force at support A	$V_{Ed,max} = \text{abs}(\max(V_{A\_max}, V_{A\_red}))$ = **95** kN
Angle of comp. shear strut for maximum shear	θ_{max} = 45 deg
Maximum design shear force (exp.6.9)	$V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **717** kN

PASS - Design shear force at support is less than maximum design shear force

Design shear force span 1 at 397 mm	$V_{Ed} = \max(V_{A\_s1\_max}, V_{A\_s1\_red})$ = **57** kN
Design shear stress	$v_{Ed} = V_{Ed} / (b \times z)$ = **0.334** N/mm^2
Strength reduction factor (cl.6.2.3(3))	$v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2]$ = **0.533**
Compression chord coefficient (cl.6.2.3(3))	α_{cw} = **1.00**
Angle of concrete compression strut (cl.6.2.3)	

$\theta = \min(\max(0.5 \times \text{Asin}[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8 \text{ deg}), 45\text{deg})$ = **21.8** deg

Area of shear reinforcement required (exp.6.13)	$A_{sv,req} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **138** mm^2/m
Shear reinforcement provided	$2 \times 8\phi$ legs at 100 c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **1005** mm^2/m
Minimum area of shear reinforcement (exp.9.5N)	$A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk}$ = **381** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N)	$s_{vl,max} = 0.75 \times d$ = **298** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Crack control (Section 7.3)

Maximum crack width	w_k = **0.3** mm
Design value modulus of elasticity reinf (3.2.7(4))	E_s = **200000** N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1)$ = **0.90**
Actual tension bar spacing	$s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{top}) / (N_{top} - 1)$ = **172** mm

Maximum stress permitted (Table 7.3N)	σ_s = **262** N/mm^2
Concrete to steel modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm}$ = **6.19**
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **221** mm
Area of concrete in the tensile zone	$A_{ct} = b \times y$ = **99424** mm^2
Minimum area of reinforcement required (exp.7.1)	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **375** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action	ψ_2 = **0.30**
Quasi-permanent limit state moment	$M_{QP} = abs(M_{A\_c21}) + \psi_2 \times abs(M_{A\_c22})$ = **0** kNm
Permanent load ratio	$R_{PL} = M_{QP} / M$ = **0.00**
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **0** N/mm^2
Maximum bar spacing (Tables 7.3N)	$s_{bar,max}$ = **300** mm

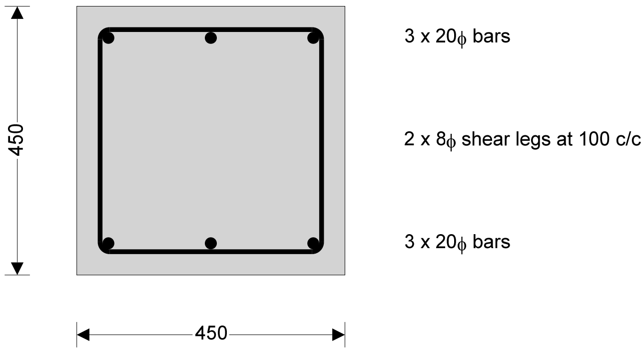
PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing	$s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1)$ = **172** mm
Minimum allowable bottom bar spacing	$s_{bar\_bot,min} = max(\phi_{bot}, h_{agg} + 5\text{ mm}, 20\text{ mm}) + \phi_{bot}$ = **45** mm
Minimum top bar spacing	$s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1)$ = **172** mm
Minimum allowable top bar spacing	$s_{bar\_top,min} = max(\phi_{top}, h_{agg} + 5\text{ mm}, 20\text{ mm}) + \phi_{top}$ = **45** mm

PASS - Actual bar spacing exceeds minimum allowable

<u>Mid span 1</u>



Rectangular section in flexure (Section 6.1) - Positive midspan moment

Design bending moment	$M = abs(M_{s1\_red})$ = **47** kNm
Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2$ = **397** mm
Percentage redistribution	$m_{rs1} = M_{s1\_red} / M_{s1\_max} - 1$ = **0** %
Redistribution ratio	$\delta = min(1 - m_{rs1}, 1)$ = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.024**
	$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **377** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **50** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **288** mm^2
Tension reinforcement provided	3 × 20ϕ bars
Area of tension reinforcement provided	$A_{s,prov}$ = **942** mm^2
Minimum area of reinforcement (exp.9.1N)	$A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **257** mm^2
Maximum area of reinforcement (cl.9.2.1.1(3))	$A_{s,max} = 0.04 \times b \times h$ = **8100** mm^2

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

Rectangular section in shear (Section 6.2)

Shear reinforcement provided	2 × 8ϕ legs at 100 c/c

Area of shear reinforcement provided	$A_{sv,prov}$ = **1005** mm^2/m
Minimum area of shear reinforcement (exp.9.5N)	$A_{sv,min} = 0.08$ N/mm^2 $\times$ b $\times$ (f_{ck} / 1 N/mm^2)$^{0.5}$ / f_{yk} = **381** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N)	$s_{vl,max} = 0.75 \times d$ = **298** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design shear resistance (assuming cot(θ) is 2.5)	$V_{prov} = 2.5 \times A_{sv,prov} \times z \times f_{yd}$ = **412.1** kN

Shear links provided valid between 0 mm and 2000 mm with tension reinforcement of 942 mm^2

Crack control (Section 7.3)

Maximum crack width	w_k = **0.3** mm
Design value modulus of elasticity reinf (3.2.7(4))	E_s = **200000** N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	k = min(max(1 + (300 mm - min(h, b)) $\times$ 0.35 / 500 mm, 0.65), 1) = **0.90**
Actual tension bar spacing	$s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{bot}) / (N_{bot} - 1)$ = **172** mm
Maximum stress permitted (Table 7.3N)	σ_s = **262** N/mm^2
Concrete to steel modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm}$ = **6.19**
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **221** mm
Area of concrete in the tensile zone	$A_{ct} = b \times y$ = **99424** mm^2
Minimum area of reinforcement required (exp.7.1)	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **375** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action	ψ_2 = **0.30**
Quasi-permanent limit state moment	$M_{QP} = abs(M_{s1\_c21}) + \psi_2 \times abs(M_{s1\_c22})$ = **35** kNm
Permanent load ratio	$R_{PL} = M_{QP} / M$ = **0.74**
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **99** N/mm^2
Maximum bar spacing (Tables 7.3N)	$s_{bar,max}$ = **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing	$s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1)$ = **172** mm
Minimum allowable bottom bar spacing	$s_{bar\_bot,min} = max(\phi_{bot}, h_{agg} + 5 mm, 20 mm) + \phi_{bot}$ = **45** mm
Minimum top bar spacing	$s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1)$ = **172** mm
Minimum allowable top bar spacing	$s_{bar\_top,min} = max(\phi_{top}, h_{agg} + 5 mm, 20 mm) + \phi_{top}$ = **45** mm

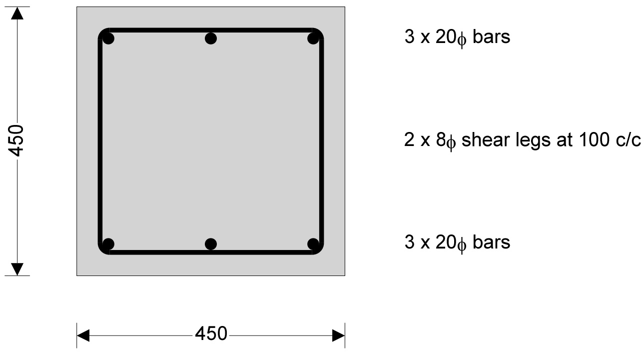
PASS - Actual bar spacing exceeds minimum allowable

Deflection control (Section 7.4)

Reference reinforcement ratio	$\rho_{m0} = (f_{ck} / 1 N/mm^2)^{0.5} / 1000$ = **0.005**
Required tension reinforcement ratio	$\rho_m = A_{s,req} / (b \times d)$ = **0.002**
Required compression reinforcement ratio	$\rho'_m = A_{s2,req} / (b \times d)$ = **0.000**
Structural system factor (Table 7.4N)	K_b = **1.0**
Basic allowable span to depth ratio (7.16a)	$span\_to\_depth_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1 N/mm^2)^{0.5} \times \rho_{m0} / \rho_m + 3.2 \times (f_{ck} / 1 N/mm^2)^{0.5} \times (\rho_{m0} / \rho_m - 1)^{1.5}]$ = **95.223**
Reinforcement factor (exp.7.17)	$K_s = min(A_{s,prov} / A_{s,req} \times 500 N/mm^2 / f_{yk}, 1.5)$ = **1.500**
Flange width factor	F1 = **1.000**
Long span supporting brittle partition factor	F2 = **1.000**
Allowable span to depth ratio	$span\_to\_depth_{allow} = min(span\_to\_depth_{basic} \times K_s \times F1 \times F2, 40 \times K_b)$ = **40.000**
Actual span to depth ratio	$span\_to\_depth_{actual} = L_{s1} / d$ = **5.038**

PASS - Actual span to depth ratio is within the allowable limit

Support B



Rectangular section in flexure (Section 6.1)

Minimum moment factor (cl.9.2.1.2(1))	β_1 = **0.25**
Design bending moment	$M = \max(\mathrm{abs}(M_{B\_red}), \beta_1 \times \mathrm{abs}(M_{s1\_red}))$ = **12** kNm
Depth to tension reinforcement	$d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2$ = **397** mm
Percentage redistribution	m_{rB} = **0** %
Redistribution ratio	$\delta = \min(1 - m_{rB}, 1)$ = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.006**
	$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **377** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **50** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **72** mm^2
Tension reinforcement provided	3 × 20ϕ bars
Area of tension reinforcement provided	$A_{s,prov}$ = **942** mm^2
Minimum area of reinforcement (exp.9.1N)	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **257** mm^2
Maximum area of reinforcement (cl.9.2.1.1(3))	$A_{s,max} = 0.04 \times b \times h$ = **8100** mm^2

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

Minimum bottom reinforcement at supports

Minimum reinforcement factor (cl.9.2.1.4(1))	β_2 = **0.25**
Area of reinforcement to adjacent span	$A_{s,span}$ = **942** mm^2
Minimum bottom reinforcement to support	$A_{s2,min} = \beta_2 \times A_{s,span}$ = **236** mm^2
Bottom reinforcement provided	3 × 20ϕ bars
Area of bottom reinforcement provided	$A_{s2,prov}$ = **942** mm^2

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

Rectangular section in shear (Section 6.2)

Design shear force at support B	$V_{Ed,max} = \mathrm{abs}(\max(V_{B\_max}, V_{B\_red}))$ = **95** kN
Angle of comp. shear strut for maximum shear	θ_{max} = 45 deg
Maximum design shear force (exp.6.9)	$V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **717** kN

PASS - Design shear force at support is less than maximum design shear force

Design shear force span 1 at 1603 mm	$V_{Ed} = \mathrm{abs}(\min(V_{B\_s1\_max}, V_{B\_s1\_red}))$ = **57** kN
Design shear stress	$v_{Ed} = V_{Ed} / (b \times z)$ = **0.334** N/mm^2
Strength reduction factor (cl.6.2.3(3))	$v_1 = 0.6 \times [1 - f_{ck} / 250\ \mathrm{N/mm^2}]$ = **0.533**
Compression chord coefficient (cl.6.2.3(3))	α_{cw} = **1.00**
Angle of concrete compression strut (cl.6.2.3)	

$\theta = \min(\max(0.5 \times \mathrm{Asin}[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8\ \mathrm{deg}), 45\mathrm{deg})$ = **21.8** deg

Area of shear reinforcement required (exp.6.13)	$A_{sv,req} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **138** mm^2/m
Shear reinforcement provided	2 × 8ϕ legs at 100 c/c

Area of shear reinforcement provided	$A_{sv,prov}$ = **1005** mm^2/m
Minimum area of shear reinforcement (exp.9.5N)	$A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk}$ = **381** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N)	$s_{vl,max} = 0.75 \times d$ = **298** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Crack control (Section 7.3)

Maximum crack width	w_k = **0.3** mm
Design value modulus of elasticity reinf (3.2.7(4))	E_s = **200000** N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300\ mm - \min(h, b)) \times 0.35 / 500\ mm, 0.65), 1)$ = **0.90**
Actual tension bar spacing	$s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{top}) / (N_{top} - 1)$ = **172** mm
Maximum stress permitted (Table 7.3N)	σ_s = **262** N/mm^2
Concrete to steel modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm}$ = **6.19**
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **221** mm
Area of concrete in the tensile zone	$A_{ct} = b \times y$ = **99424** mm^2
Minimum area of reinforcement required (exp.7.1)	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **375** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action	ψ_2 = **0.30**
Quasi-permanent limit state moment	$M_{QP} = abs(M_{B\_c21}) + \psi_2 \times abs(M_{B\_c22})$ = **0** kNm
Permanent load ratio	$R_{PL} = M_{QP} / M$ = **0.00**
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **0** N/mm^2
Maximum bar spacing (Tables 7.3N)	$s_{bar,max}$ = **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing	$s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1)$ = **172** mm
Minimum allowable bottom bar spacing	$s_{bar\_bot,min} = \max(\phi_{bot}, h_{agg} + 5\ mm, 20\ mm) + \phi_{bot}$ = **45** mm
Minimum top bar spacing	$s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1)$ = **172** mm
Minimum allowable top bar spacing	$s_{bar\_top,min} = \max(\phi_{top}, h_{agg} + 5\ mm, 20\ mm) + \phi_{top}$ = **45** mm

PASS - Actual bar spacing exceeds minimum allowable

PILE WALL 2

Use same as pile wall 1

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INTERNAL WALL 1

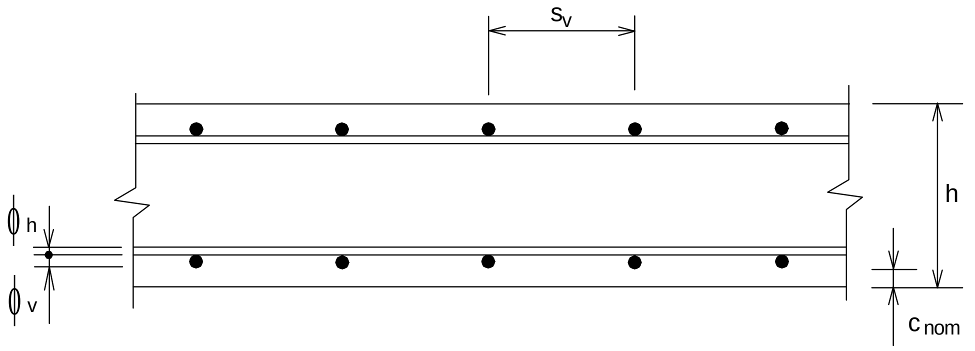
RC WALL DESIGN (EN1992)

Loadings
Dead loadDL=32kN/m
Live loadLL=17kN/m

RC WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.08



Wall geometry

Thickness	h = **300** mm	Length	b = **1000** mm/m
Stability about minor axis	**Braced**		

Concrete details

Concrete strength class	**C28/35**	Safety factor for concrete	γ_C = **1.50**
Coefficient α_{cc}	α_{cc} = **0.85**		
Maximum aggregate size	d_g = **20** mm		

Reinforcement details

Reinforcement in outer layer	**Vertical**	Nominal cover to outer layer	c_{nom} = **30** mm
Vertical bar diameter	ϕ_v = **16** mm	Horizontal bar diameter	ϕ_h = **10** mm
Spacing of vertical reinf	s_v = **100** mm	Spacing of horizontal reinft	s_h = **100** mm
Area of vertical reinft (per face)	A_{sv} = **2011** mm^2/m	Area of horiz. reinft (per face)	A_{sh} = **785** mm^2/m
Partial safety factor for reinft	γ_S = **1.15**	Modulus of elasticity of reinft	E_s = **200000** MPa

Fire resistance details

Fire resistance period R = **60** min Exposure to fire **Exposed on two sides**

Ratio of fire design axial load to design resistance μ_{fi} = **0.70**

Axial load and bending moments from frame analysis

Design axial load	N_{Ed} = **73.5** kN/m		
Mt about minor axis at top	M_{top} = **7.0** kNm/m	Mt about minor axis at bottom	M_{btm} = **7.0** kNm/m

Wall effective length

Effective length l_0 = **4000** mm

Check nominal cover for fire and bond requirements

Min. cover reqd for bond	$c_{min,b}$ = **16** mm	Min axis distance for fire	a_{fi} = **10** mm

Allowance for deviations Δc_{dev} = **10** mm   Min allowable nominal cover c_{nom_min} = **26.0** mm

PASS - the nominal cover is greater than the minimum required

Wall slenderness

Slenderness ratio λ = **46.2**   Slenderness limit λ_{lim} = **103.9**

$\lambda<\lambda_{lim}$ - Second order effects may be ignored

Design bending moment

Design mt about minor axis M_{Ed} = **7.7** kNm/m

Moment of resistance

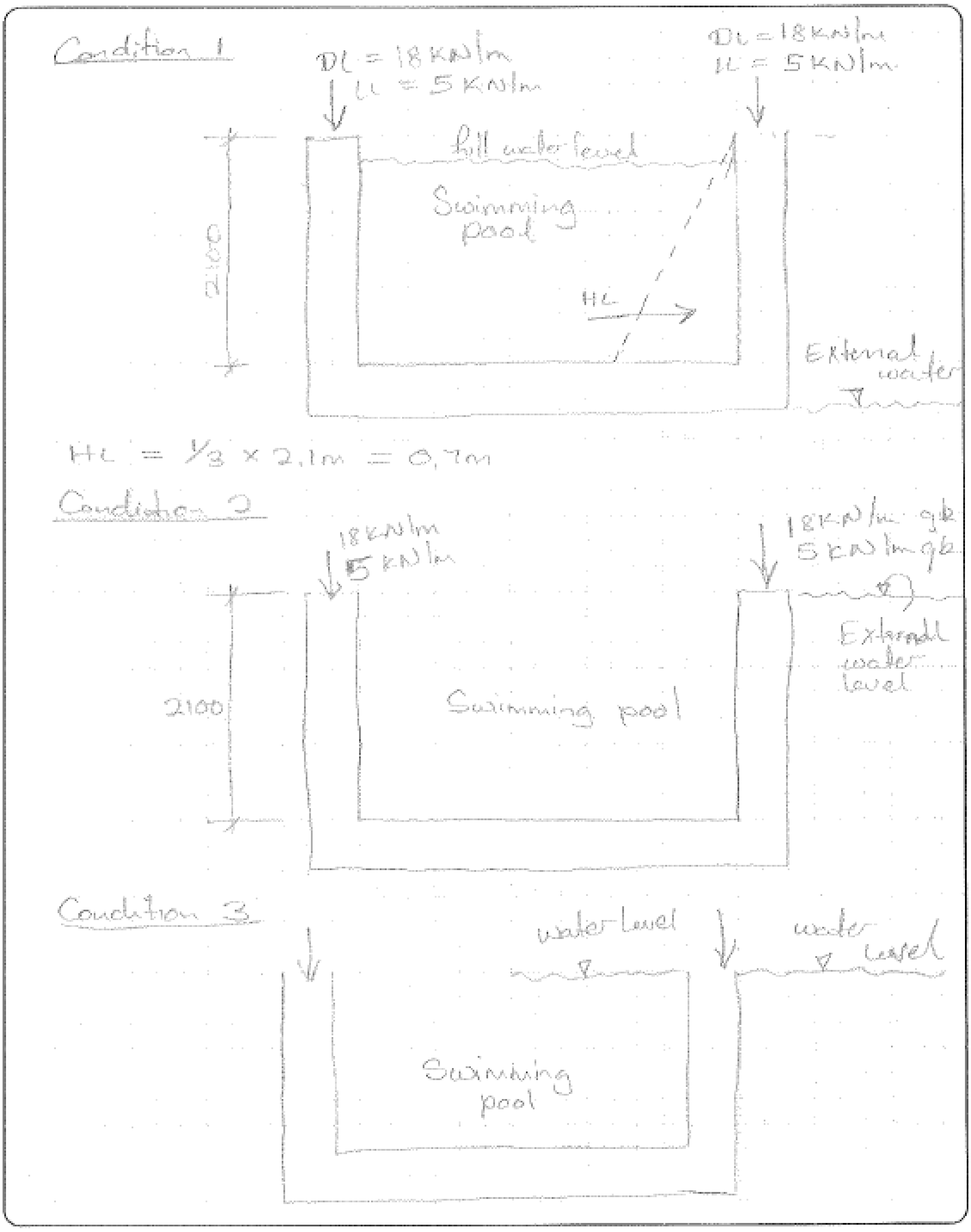
Mt of resist. about minor axis M_{Rd} = **215.4** kNm/m

PASS - The moment of resistance about the minor axis exceeds the design bending moment

INTERNAL WALL 2

Use same as internal wall 1.

WALL 2 (CONDITION 1)



RETAINING WALL ANALYSIS & DESIGN (EN1992/EN1996/EN1997)

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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 2.6.04

Retaining wall details

Stem type	Cantilever		
Stem height	h_{stem} = **2100** mm		
Prop height	h_{prop} = **2000** mm		
Stem thickness	t_{stem} = **300** mm		
Angle to rear face of stem	α = **90** deg		
Stem density	γ_{stem} = **25** kN/m^3		
Toe length	l_{toe} = **1000** mm		
Heel length	l_{heel} = **1000** mm		
Base thickness	t_{base} = **350** mm		
Base density	γ_{base} = **25** kN/m^3		
Height of retained soil	h_{ret} = **2100** mm	Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm		

Retained soil properties

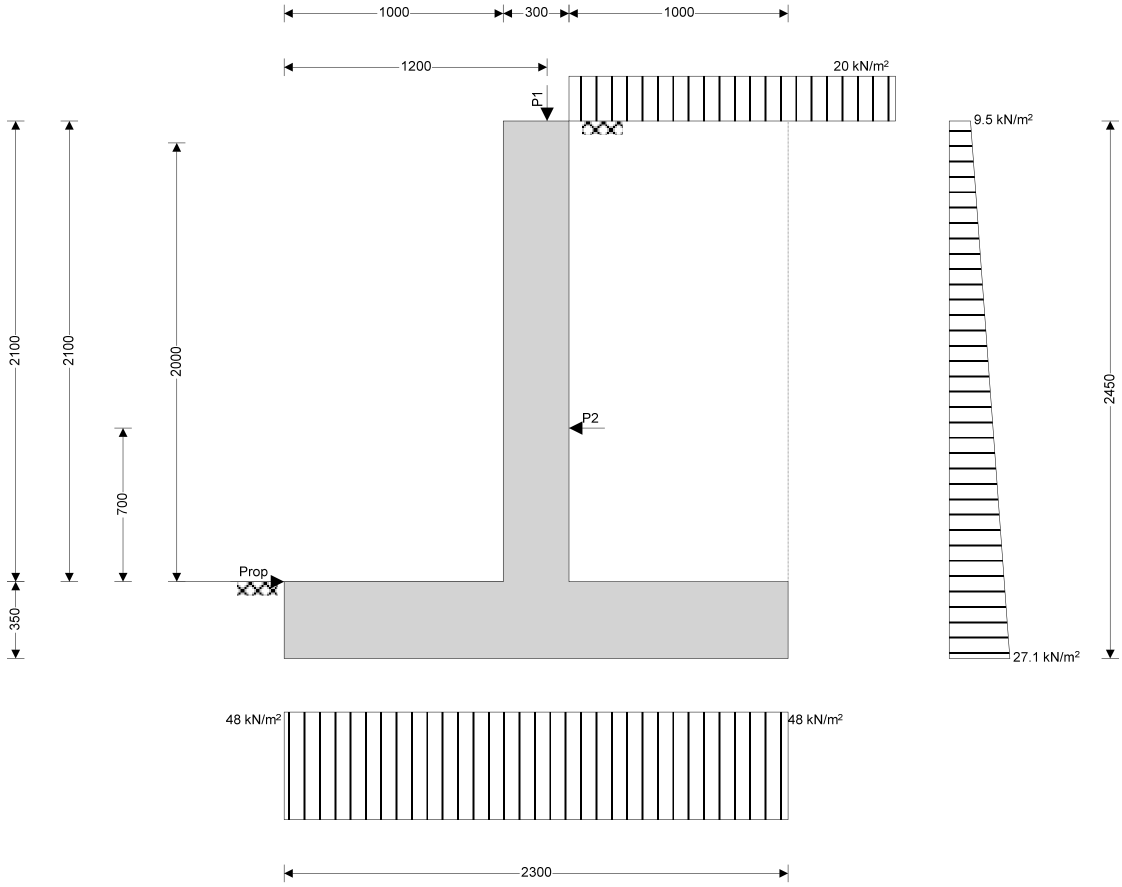
Soil type	Organic clay	
Moist density	γ_{mr} = **15** kN/m^3	
Saturated density	γ_{sr} = **15** kN/m^3	
Characteristic effective shear resistance angle		$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **9** deg	

Base soil properties

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

Loading details

Permanent surcharge load	$Surcharge_G$ = **10** kN/m^2
Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 1200 mm	P_{G1} = **18** kN/m
	P_{Q1} = **5** kN/m
Horizontal line load at 700 mm	P_{G2} = **-10** kN/m
	P_{Q2} = **-10** kN/m



Calculate retaining wall geometry

Base length	l_{base} = **2300** mm		
Moist soil height	h_{moist} = **2100** mm		
Length of surcharge load	l_{sur} = **1000** mm		
Vertical distance	x_{sur_v} = **1800** mm		
Effective height of wall	h_{eff} = **2450** mm		
Horizontal distance	x_{sur_h} = **1225** mm		
Area of wall stem	A_{stem} = **0.63** m^2	Vertical distance	x_{stem} = **1150** mm
Area of wall base	A_{base} = **0.805** m^2	Vertical distance	x_{base} = **1150** mm
Area of moist soil	A_{moist} = **2.1** m^2	Vertical distance	x_{moist_v} = **1800** mm
		Horizontal distance	x_{moist_h} = **817** mm

Using Coulomb theory

Active pressure coefficient	K_A = **0.483**	Passive pressure coefficient	K_P = **4.977**

Bearing pressure check

Vertical forces on wall

Total $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{sur\_v} + F_{P\_v}$ = **110.4** kN/m

Horizontal forces on wall

Total $F_{total\_h} = F_{moist\_h} + F_{pass\_h} + F_{sur\_h} + F_{P\_h}$ = **19.6** kN/m

Moments on wall

Total $M_{total} = M_{stem} + M_{base} + M_{moist} + M_{sur} + M_P$ = **136.4** kNm/m

Check bearing pressure

Propping force	F_{prop_base} = **19.6** kN/m		
Bearing pressure at toe	q_{toe} = **48** kN/m^2	Bearing pressure at heel	q_{heel} = **48** kN/m^2
Factor of safety	FoS_{bp} = **3.126**		

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PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

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

Tedds calculation version 2.6.04

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

Concrete strength class C30/37

Char.comp.cylinder strength f_{ck} = **30** N/mm^2 Mean axial tensile strength f_{ctm} = **2.9** N/mm^2

Secant modulus of elasticity E_{cm} = **32837** N/mm^2 Maximum aggregate size h_{agg} = **20** mm

Design comp.concrete strength f_{cd} = **17.0** N/mm^2 Partial factor γ_C = **1.50**

Reinforcement details

Characteristic yield strength f_{yk} = **500** N/mm^2 Modulus of elasticity E_s = **200000** N/mm^2

Design yield strength f_{yd} = **435** N/mm^2 Partial factor γ_S = **1.15**

Cover to reinforcement

Front face of stem c_{sf} = **40** mm Rear face of stem c_{sr} = **50** mm

Top face of base c_{bt} = **50** mm Bottom face of base c_{bb} = **75** mm

Check stem design at base of stem

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **37.9** kNm/m K = **0.021** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{sr.req}$ = **376** mm^2/m

Tens.reinforcement provided 12 dia.bars @ 200 c/c Tens.reinforcement provided $A_{sr.prov}$ = **565** mm^2/m

Min.area of reinforcement $A_{sr.min}$ = **368** mm^2/m Max.area of reinforcement $A_{sr.max}$ = **12000** mm^2/m

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

Deflection control - Section 7.4

Limiting span to depth ratio 67.1 Actual span to depth ratio 8.6

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.189** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **39.9** kN/m Design shear resistance $V_{Rd.c}$ = **123** kN/m

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Min.area of reinforcement $A_{sx.req}$ = **300** mm^2/m Max.spacing of reinforcement s_{sx_max} = **400** mm

Trans.reinforcement provided 10 dia.bars @ 200 c/c Trans.reinforcement provided $A_{sx.prov}$ = **393** mm^2/m

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

Check base design at toe

Depth of section h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **27** kNm/m K = **0.012** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{bb.req}$ = **243** mm^2/m

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Tens.reinforcement provided	12 dia.bars @ 200 c/c	Tens.reinforcement provided	$A_{bb.prov}$ = **565** mm^2/m
Min.area of reinforcement	$A_{bb.min}$ = **405** mm^2/m	Max.area of reinforcement	$A_{bb.max}$ = **14000** mm^2/m

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

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm	Maximum crack width	w_k = **0.259** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **54** kN/m	Design shear resistance	$V_{Rd.c}$ = **131.1** kN/m

PASS - Design shear resistance exceeds design shear force

Rectangular section in flexure - Section 6.1

Design bending moment	M = **8.5** kNm/m	K = **0.003**	K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required	$A_{bt.req}$ = **70** mm^2/m		
Tens.reinforcement provided	12 dia.bars @ 200 c/c	Tens.reinforcement provided	$A_{bt.prov}$ = **565** mm^2/m
Min.area of reinforcement	$A_{bt.min}$ = **443** mm^2/m	Max.area of reinforcement	$A_{bt.max}$ = **14000** mm^2/m

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

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm	Maximum crack width	w_k = **0.043** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

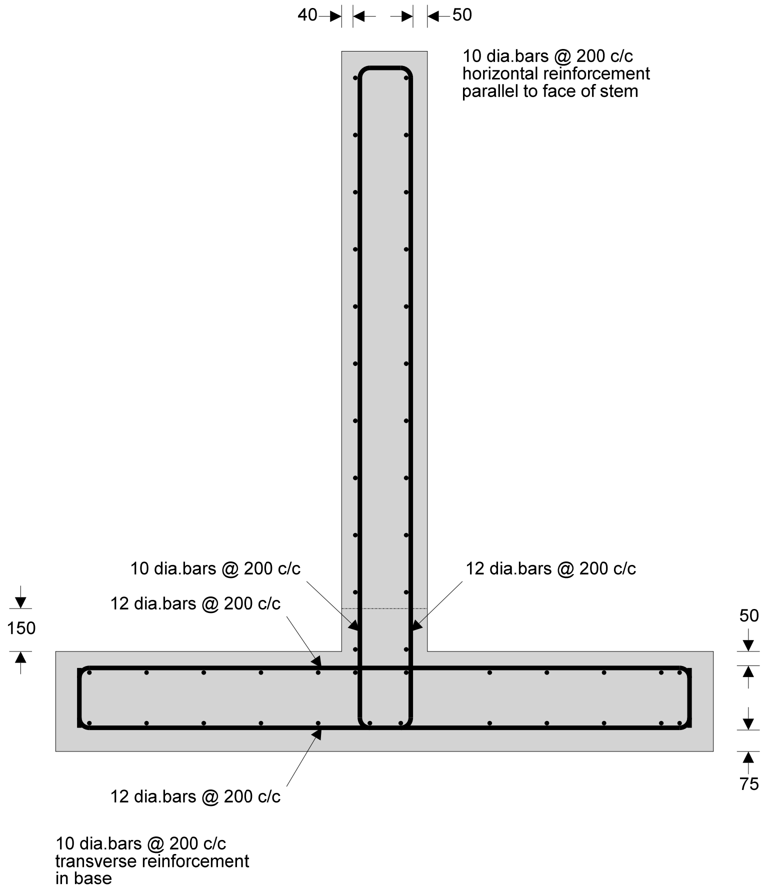
Design shear force	V = **17.1** kN/m	Design shear resistance	$V_{Rd.c}$ = **138.9** kN/m

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Min.area of reinforcement	$A_{bx.req}$ = **113** mm^2/m	Max.spacing of reinforcement	s_{bx_max} = **450** mm
Trans.reinforcement provided	10 dia.bars @ 200 c/c	Trans.reinforcement provided	$A_{bx.prov}$ = **393** mm^2/m

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


40
50
10 dia.bars @ 200 c/c
horizontal reinforcement
parallel to face of stem
10 dia.bars @ 200 c/c
12 dia.bars @ 200 c/c
12 dia.bars @ 200 c/c
150
50
75
12 dia.bars @ 200 c/c
10 dia.bars @ 200 c/c
transverse reinforcement
in base

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WALL 2 (CONDITION 2)

RETAINING WALL ANALYSIS & DESIGN (EN1992/EN1996/EN1997)

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 2.6.04

Retaining wall details

Stem type	Cantilever		
Stem height	h_{stem} = **2100** mm		
Prop height	h_{prop} = **2000** 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		
Heel length	l_{heel} = **300** mm		
Base thickness	t_{base} = **350** mm		
Base density	γ_{base} = **25** kN/m^3		
Height of retained soil	h_{ret} = **2100** mm	Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm		
Height of water	h_{water} = **2100** mm		
Water density	γ_w = **9.8** kN/m^3		

Retained soil properties

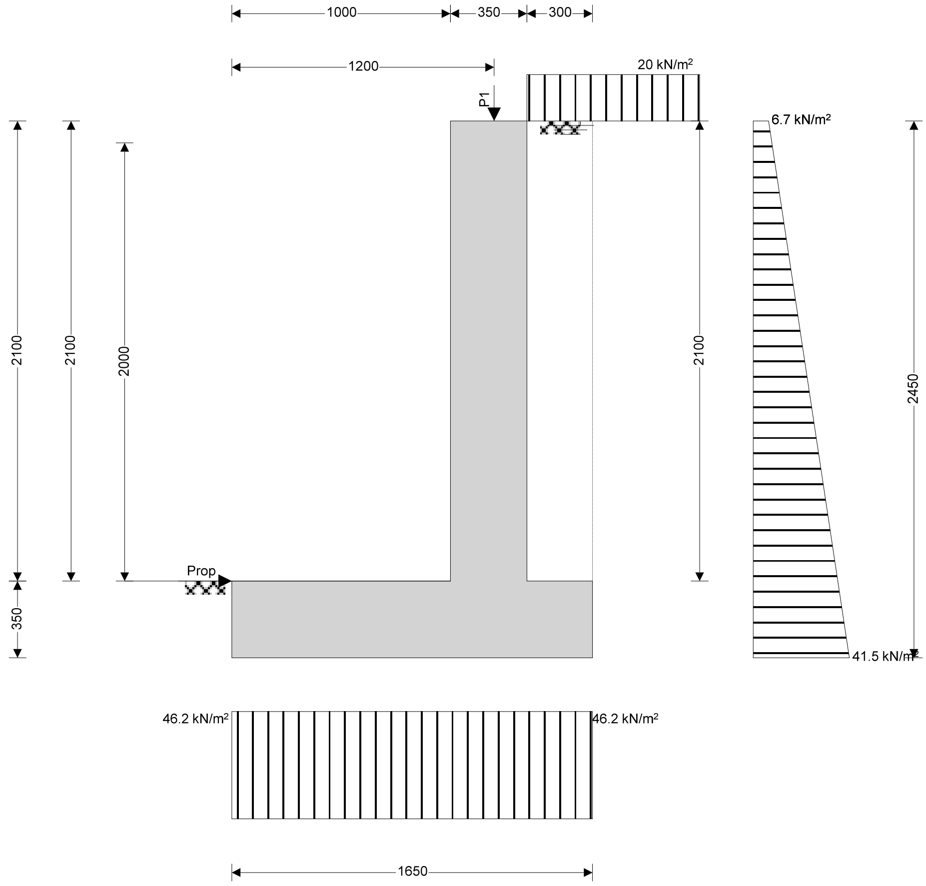
Soil type	Medium dense well graded sand		
Moist density	γ_{mr} = **21** kN/m^3		
Saturated density	γ_{sr} = **23** kN/m^3		
Characteristic effective shear resistance angle		$\phi'_{r.k}$ = **30** deg	
Characteristic wall friction angle	$\delta_{r.k}$ = **0** deg		

Base soil properties

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

Loading details

Permanent surcharge load	$Surcharge_G$ = **10** kN/m^2
Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 1200 mm	P_{G1} = **18** kN/m
	P_{Q1} = **5** kN/m



Calculate retaining wall geometry

Base length	l_{base} = **1650** mm		
Saturated soil height	h_{sat} = **2100** mm		
Moist soil height	h_{moist} = **0** mm		
Length of surcharge load	l_{sur} = **300** mm		
Vertical distance	x_{sur_v} = **1500** mm		
Effective height of wall	h_{eff} = **2450** mm		
Horizontal distance	x_{sur_h} = **1225** mm		
Area of wall stem	A_{stem} = **0.735** m^2	Vertical distance	x_{stem} = **1175** mm
Area of wall base	A_{base} = **0.578** m^2	Vertical distance	x_{base} = **825** mm
Area of saturated soil	A_{sat} = **0.63** m^2	Vertical distance	x_{sat_v} = **1500** mm
		Horizontal distance	x_{sat_h} = **817** mm
Area of water	A_{water} = **0.63** m^2	Vertical distance	x_{water_v} = **1500** mm
		Horizontal distance	x_{water_h} = **817** mm

Using Coulomb theory

Active pressure coefficient	K_A = **0.333**	Passive pressure coefficient	K_P = **4.977**

Bearing pressure check

Vertical forces on wall

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v}$ = **76.3** kN/m

Horizontal forces on wall

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **53.7** kN/m

Moments on wall

Total $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P$ = **37** kNm/m

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Check bearing pressure

Propping force F_{prop_base} = **53.7** kN/m

Bearing pressure at toe q_{toe} = **46.2** kN/m^2 Bearing pressure at heel q_{heel} = **46.2** kN/m^2

Factor of safety FoS_{bp} = **3.244**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

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

Tedds calculation version 2.6.04

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

Concrete strength class C30/37

Char.comp.cylinder strength f_{ck} = **30** N/mm^2 Mean axial tensile strength f_{ctm} = **2.9** N/mm^2

Secant modulus of elasticity E_{cm} = **32837** N/mm^2 Maximum aggregate size h_{agg} = **20** mm

Design comp.concrete strength f_{cd} = **17.0** N/mm^2 Partial factor γ_C = **1.50**

Reinforcement details

Characteristic yield strength f_{yk} = **500** N/mm^2 Modulus of elasticity E_s = **200000** N/mm^2

Design yield strength f_{yd} = **435** N/mm^2 Partial factor γ_S = **1.15**

Cover to reinforcement

Front face of stem c_{sf} = **40** mm Rear face of stem c_{sr} = **50** mm

Top face of base c_{bt} = **50** mm Bottom face of base c_{bb} = **75** mm

Check stem design at base of stem

Depth of section h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **50.6** kNm/m K = **0.019** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{sr.req}$ = **416** mm^2/m

Tens.reinforcement provided 12 dia.bars @ 100 c/c Tens.reinforcement provided $A_{sr.prov}$ = **1131** mm^2/m

Min.area of reinforcement $A_{sr.min}$ = **443** mm^2/m Max.area of reinforcement $A_{sr.max}$ = **14000** mm^2/m

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

Deflection control - Section 7.4

Limiting span to depth ratio 76.8 Actual span to depth ratio 7.1

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.115** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **62.2** kN/m Design shear resistance $V_{Rd.c}$ = **138.9** kN/m

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Min.area of reinforcement $A_{sx.req}$ = **350** mm^2/m Max.spacing of reinforcement s_{sx_max} = **400** mm

Trans.reinforcement provided 10 dia.bars @ 200 c/c Trans.reinforcement provided $A_{sx.prov}$ = **393** mm^2/m

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

Check base design at toe

Depth of section h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **25.7** kNm/m K = **0.012** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{bb.req}$ = **231** mm^2/m

Tens.reinforcement provided 12 dia.bars @ 200 c/c Tens.reinforcement provided $A_{bb.prov}$ = **565** mm^2/m

Min.area of reinforcement $A_{bb.min}$ = **405** mm^2/m Max.area of reinforcement $A_{bb.max}$ = **14000** mm^2/m

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.247** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **51.3** kN/m Design shear resistance $V_{Rd.c}$ = **131.1** kN/m

PASS - Design shear resistance exceeds design shear force

Rectangular section in flexure - Section 6.1

Design bending moment M = **1.9** kNm/m K = **0.001** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{bt.req}$ = **16** mm^2/m

Tens.reinforcement provided 12 dia.bars @ 200 c/c Tens.reinforcement provided $A_{bt.prov}$ = **565** mm^2/m

Min.area of reinforcement $A_{bt.min}$ = **443** mm^2/m Max.area of reinforcement $A_{bt.max}$ = **14000** mm^2/m

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.013** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **12.7** kN/m Design shear resistance $V_{Rd.c}$ = **138.9** kN/m

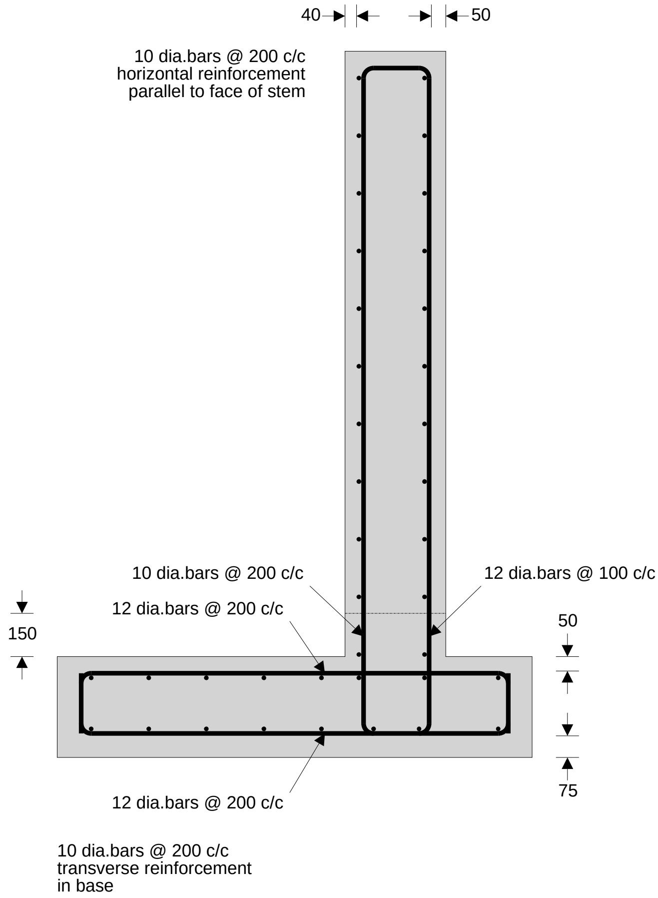
PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Min.area of reinforcement $A_{bx.req}$ = **113** mm^2/m Max.spacing of reinforcement s_{bx_max} = **450** mm

Trans.reinforcement provided 10 dia.bars @ 200 c/c Trans.reinforcement provided $A_{bx.prov}$ = **393** mm^2/m

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



WALL 2 (CONDITION 3)

Water is on both sides of the wall therefer the wall is more stable

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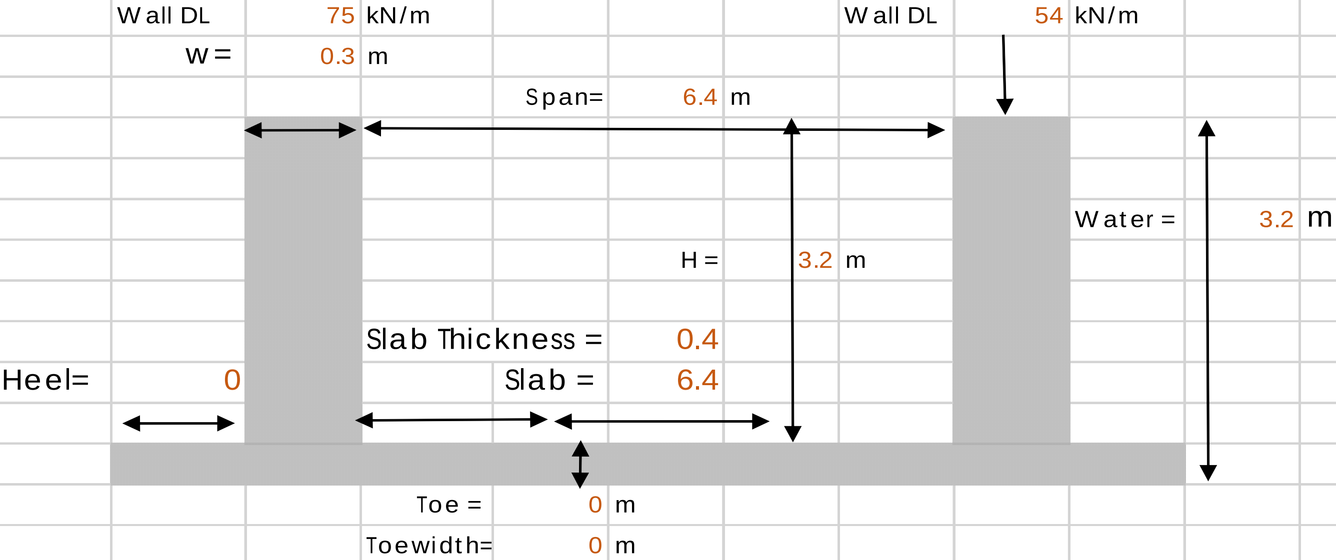
Reference

Uplift



Uplift Calc

Total Dead Load =	Slab=	64	kN/m				
	Toe and heel =	0	kN/m				
	Wall =	48					
	Soil=(	0	+	0	) x 2=	0	6.4
	Total Dead load =	241	kN/m				
Total Uplift Force=		224	kN/m		f.o.s.=	**1.08**	**No Global Uplift**

Slab Uplift

Slab =	10	kN/m		Uplift =	32
Service Moment =	-112.64	kNm/m			
Factored Design moment=	-134.14	kNm/m			
Factored Design shear =	-83.84	kN/m			

Global Heave

Weight of building =	170	kN/m			
Weight of soil removed =	403.2				
% change	58%		place	58%	of Slab area as heave protection
Wide of Heave protection =	4.04861	m	place	4.05	m of Slab area as heave protection

CROFT STRUCTURAL ENGINEERS

Noise and Nuisance Control	The contractor is to follow the good working practices and guidance laid down in the "Considerate Constructors Scheme". The hours of working will be limited to those allowed; 8am to 5pm Monday to Friday and Saturday Morning 8am to 1pm. None of the practices cause undue noise that one would typically expect from a construction site. The conveyor belt typically runs at around 70dB. The site has car parking to the front to which the skip will be stored. The site will be hoarded with 8' site hoarding to prevent access. The hours of working will further be defined within the Party Wall Act. The site is to be hoarded to minimise the level of direct noise from the site. Ground floor slab is not being removed minimising the vibration and sound to adjacent properties. While working in the basement the work generally requires hand tools to be used. The level of noise generally will be no greater than that of digging of soil. The noise is reduced and muffled by the works being undertaken underground. A level of noise from a basement is lower than typical ground level construction due to this.
CTMP	The council may require a Construction Traffic Management plan to be produced. This is outside the brief of the Basement impact assessment and is not covered within Croft's Brief

Appendix A ; Construction Method Statement

CROFT STRUCTURAL ENGINEERS

Basement Method Statement

1B St Johns Wood Park:
London
W8

Client Information:
Mike Ofori

CROFT STRUCTURAL ENGINEERS

Contents

CROFT STRUCTURAL ENGINEERS

1B St Johns Wood Park

1. Basement Formation Suggested Method Statement.

1.1. This method statement provides an approach which will allow the basement design to be correctly considered during construction, and the temporary support to be provided during the works. The Contractor is responsible for the works on site and the final temporary works methodology and design on this site and any adjacent sites.

1.2. This method statement for 1B St Johns Wood Park has been written by a Chartered Engineer. The sequencing has been developed considering guidance from ASUC.

1.3. This method has been produced to allow for improved costings and for inclusion in the party wall Award. Should the contractor provide alternative methodology the changes shall be at their own costs, and an Addendum to the Party Wall Award will be required.

4.0

1.4. Contact party wall surveyors to inform them of any changes to this method statement.

1.5. The approach followed in this design is; to remove load from above and place loads onto supporting steelwork, then to cast retaining walls in underpin sections at the new basement level.

1.6. A soil investigation has been undertaken. The soil conditions are London Clay formation

5.0

1.7. The Chemical laboratory testing revealed below. Lead specialists are to be called in before work commences to remove the lead from the ground and treat the soil. Work should only commence once lead contamination has been eliminated.

6.0

Chemical laboratory testing revealed an elevated level of lead in one sample of Made Ground. A level of 470mg/kg was noted within BH1/0.30m bgl in excess of the LQM/CIEH S4ULs of 210mg/kg for a ***"Residential with homegrown produce"*** scenario.

1.8. The bearing pressures have been limited to 150kN/m^2. This is standard loadings for local ground conditions and acceptable to building control and their approvals.

1.9. The water table is expected to encountered at 0.5m BGL

7.0

1.10. Structural Water proofer (Not Croft) must comment on the design proposed and ensure they are satisfied that proposals will provide adequate water proofing.

8.0

1.1. Provide engineers with concrete mix, supplier, deliver and placement methods 2 weeks prior to first pour. Site mixing of concrete should not be employed apart from in small sections <1m^3. Contractor must provide method on how to achieve site mixing to correct specification, contractor must undertake tool box talks with staff to ensure site quality is maintained.

2. Enabling Works

2.1. The site is to be hoarded with ply sheet to 2.2m to prevent unauthorised public access.

2.2. Licenses for Skips and conveyors to be posted on hoarding

2.3. Provide protection to public where conveyor extends over footpath. Depending on the requirements of the local authority, construct a plywood bulkhead onto the pavement. Hoarding to have a plywood roof covering, night-lights and safety notices.

9.0

2.4. Dewater: Water is expected at 0.5 depths

10.0

2.4.1. Place a bore hole to the rear of the property down to a depth of 6m

2.4.2. Pump water away from site.

2.5. On commencement of construction the contractor should report any discrepancies to the structural engineer in order that the detailed design may be modified as necessary.

3. Piling Sequencing

3.1. Piles are to be installed at different levels and positions around the development. All piles are installed from the same level and cut down as required.

11.0

3.1.1. Prior to bringing the piling rig on site, check with the piling contractor the requirements of a working platform and install to their design and specification if required.

3.1.2. Mark out datum line to determine various surface heights

12.0

3.1.3. Mark out pile sequence locations as specified by Engineer's drawings.

13.0

3.1.4. Following the sequencing guidance from the Engineers drawings mark out proposed pile position with a pair of reference markers at 1.0m from the pile pin, approximately 90 degrees apart.

14.0

3.1.5. Rig operator to set up over the pile pin position and position auger relative to reference marks. Directed and checked by banks man.

15.0

3.1.6. The flap at the tip of the auger is closed and secured. Auger tip lowered to ground level and position rechecked. Drilling to commence upon banks man approval.

16.0

3.1.7. Concrete is prepared while piling gang grout up concrete pump, hoses and flight, concrete pump operator to check concrete complies with design mix. Concrete held in agitator.

17.0

3.1.8. Rig operator augers to require design depth. Reference makers are to be used to check pile position during the first few meters of drilling.

18.0

3.1.9. If obstruction encountered, Engineer to be notified of pile number and depth. Move rig to next pile position whilst obstruction removal is dealt with. Contractor to be advised on procedure should obstruction not be removable. If necessary, pile bores to be backfilled and made safe. Open excavation to be protected when open.

19.0

20.0

3.1.10. When design depth reached, the auger is to be kept rotating to allow spoil in the bore to rise.

3.1.11. Concrete can be pumped to rig while rig operator monitors instrumentation and adjust auger rate of withdrawal accordingly.

21.0

3.1.12. Pressure, concrete flow and over-break to be monitored throughout operation.

22.0

3.1.13. During the withdrawal the rig operator is to activate the flight cleaner. If an automatic cleaner is not fitted to the rig then the piling gang must clean the flight manually to prevent spoil/ arising travelling above head height – this will be controlled by the piling foreman who must ensure the auger is not rotating when it is manually cleaned.

23.0

3.1.14. When auger tip reaches platform level, concrete pumping is stopped.

24.0

3.1.15. Attendant excavator as directed by the banks man clears spoil and concrete slurry from pile heap.

25.0

3.1.16. Banks man to check position of the cage in the pile, centrering where necessary. Reinforcement generally to be installed flush with Piling Platform Level (PPL). Anchor pile reinforcement or threaded bars that project above piling platform to have protective caps.

26.0

3.1.17. Concrete testing cube samples to be taken as per engineering specification.

27.0

3.1.18. Rig is moved onto next pile in the sequence and positioned as above, with piles installed as per points 3.1.5 – 3.1.12

28.0

3.1.19. Equipment to be cleaned and maintained as per normal methods.

29.0

3.1.20. This sequence of piling is to continue until all perimeter piles have been installed.

30.0

3.1.21. Cast internal bases and columns from basement to ground floor level.

31.0

3.2. Once all piles have been installed, bases and steel columns have been installed and additional temporary piles included, the next step sequence is to cast capping beams and install the steelwork at ground level that which in permanent condition will prop the external perimeter of the basement.

3.3. When steelwork has been set up, the excavation of the central mass can begin using mechanic excavators (an opening big enough to allow for access for machinery and spoil removal should be left.

32.0

3.4. As excavation continues down, a dewatering system will need to be considered. There are several method of doing this but the most common method is to install well points from which ground water can be pumped as mentioned in point 2.4.1

33.0

3.5. Once excavation is level done to the intermediate floor level the steelwork is installed: this will prop the external perimeter of the basement in permanent condition as the ground floor steelwork. Effectively the basement is constructed in a top down method for other works to be the development to be undertaken at the same time as the basement dig out.
34.0

4. Demolition, Recycling, Dust/Noise Control and Site Hoarding

4.1. Demolition work is to take place within the hoarded confines of the materials such as stock bricks, timber etc. are to be recycled where possible. To minimise dust and dirt from demolition the following measures shall be implemented:

4.1.1. Any debris or dust or dirt falling on the street and public highway will be cleared as it occurs by designated cleaners and washed down fully every night.
4.1.2. Demolished materials are to be removed to a skip placed in front of the site which will be emptied regularly as required.
4.1.3. All brickwork and concrete demolition work is to be constantly watered to reduce airborne dust

4.2. Building work which can be heard at the boundary of the site will not be carried out on Sundays or bank holidays and will be carried out within working hours as agreed by the council.

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5. Trench sheet design and temporary prop Calculations

This calculation has been provided for the trench sheet and prop design of standard underpins in the temporary condition. There are gaps left between the sheeting and as such no water pressure will occur. Any water present will flow through the gaps between the sheeting and will be required to pump out.

Trench sheets should be placed at centres to deal with the ground. It is expected that the soil between the trench sheeting will arch. Looser soil will required tighter centres. It is typical for underpins to be placed at 1200c/c, in this condition the highest load on a trench sheet is when 2 nos trench sheets are used. It is for this design that these calculations have been provided.

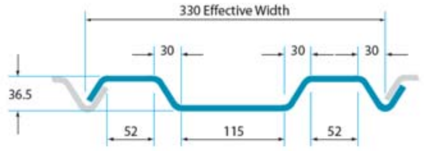
Soil and ground conditions are variable. Typically one finds that in the temporary condition clays are more stable and the C_u (cohesive) values in clay reduce the risk of collapse. It is this cohesive nature that allows clays to be cut into a vertical slope. For these calculations weak sand and gravels have been assumed The soil properties are:

Surcharge	sur = **10.** kN/m^2	
Soil density	δ = **20** kN/m^3	
Angle of friction	ϕ = **25** °	
Soil depth	Dsoil = **3000.000** mm	
	$k_a = (1 - \sin(\phi)) / (1 + \sin(\phi))$	= **0.406**
	$k_p = 1 / k_a$	= **2.464**
Soil Pressure bottom	soil = $k_a * \delta *$ Dsoil	= **21.916**kN/m^2
Surcharge pressure	surcharge = sur * k_a	= **4.059** kN/m^2

STANDARD LAP TRENCH SHEETING

STANDARD LAP

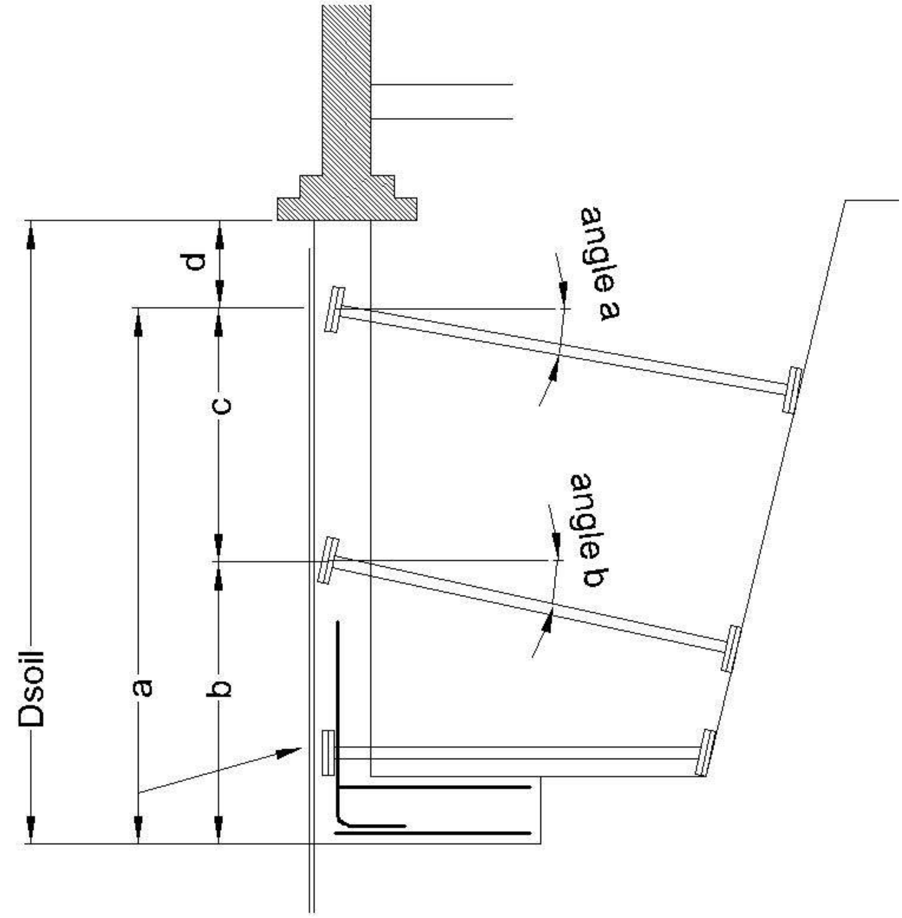
The overlapping trench sheeting profile is designed primarily for construction work and also temporary deployment.



Technical Information

Effective width per sheet (mm)	330
Thickness (mm)	3.4
Depth (mm)	35
Weight per linear metre (kg/m)	10.8
Weight per m² (kg)	32.9
Section modulus per metre width (cm³)	48.3
Section modulus per sheet (cm³)	15.9
I value per metre width (cm⁴)	81.7
I value per sheet (cm⁴)	26.9
Total rolled metres per tonne	92.1

Sxx = 15.9 cm^3
py = 275N/mm^2
Ixx = 26.9cm^4
A = (1m^2 * 32.9kg/m^2) / (330mm * 7750kg/m^3) = **12864.125**mm^2

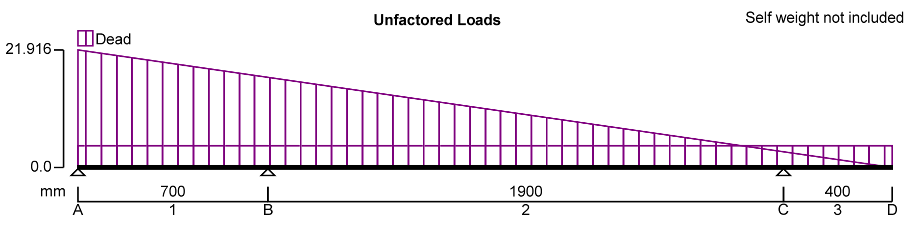


Length a a = **2.600** m

Length b bottom b = **0.700** m

Length c Middle c = a – b = **1.900**m

Length d top d = Dsoil – a = **0.400**m



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **3**

Material Properties:

Modulus of elasticity = **205** kN/mm^2 Material density = **7860** kg/m^3

Support Conditions:

Support A Vertically **"Restrained"** Rotationally **"Free"**

Support B Vertically **"Restrained"** Rotationally **"Free"**

Support C Vertically **"Restrained"** Rotationally **"Free"**

CROFT STRUCTURAL ENGINEERS

Support D	Vertically **"Free"**		Rotationally **"Free"**

Span Definitions:

Span 1	Length = **700** mm	Cross-sectional area = **12864** mm^2	Moment of inertia = **269.×10³** mm^4
Span 2	Length = **1900** mm	Cross-sectional area = **12864** mm^2	Moment of inertia = **269.×10³** mm^4
Span 3	Length = **400** mm	Cross-sectional area = **12864** mm^2	Moment of inertia = **269.×10³** mm^4

LOADING DETAILS

Beam Loads:

Load 1 UDL Dead load **4.1** kN/m

Load 2 VDL Dead load **21.9** kN/m to **0.0** kN/m

LOAD COMBINATIONS

Load combination 1

Span 1 1×Dead

Span 2 1×Dead

Span 3 1×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Unfactored support reactions

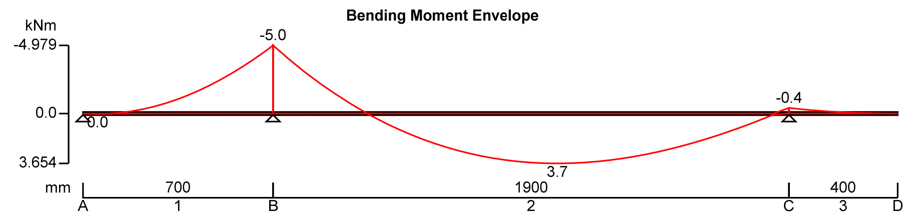
	Dead (kN)							
Support A	**-1.4**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
Support B	**-32.8**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
Support C	**-10.8**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
Support D	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**

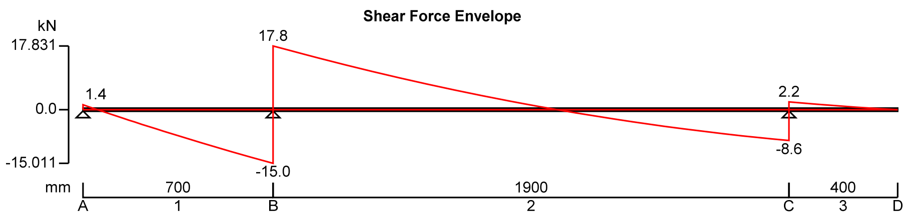
Support Reactions - Combination Summary

Support A	Max react = **-1.4** kN	Min react = **-1.4** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-32.8** kN	Min react = **-32.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-10.8** kN	Min react = **-10.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support D	Max react = **0.0** kN	Min react = **0.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm

Beam Max/Min results - Combination Summary

Maximum shear = **17.8** kN	Minimum shearF_{min} = **-15.0** kN
Maximum moment = **3.7** kNm	Minimum moment = **-5.0** kNm
Maximum deflection = **21.0** mm	Minimum deflection = **-14.3** mm





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Number of sheets Nos = 2

Mallowable = Sxx * py * Nos = **8.745**kNm

SRU 4.0

Safe working loads for Acrow Props — loads given in kN

For normal purposes 1 kilo Newton (kN) = 100 kg	Height	m ft	2.0 6.6	2.25 7.4	2.5 8.2	2.75 9.0	3.0 9.8	3.25 10.7	3.5 11.5	3.75 12.3	4.0 13.1	4.25 13.9	4.5 14.8	4.75 15.6
TABLE A Props loaded concentrically and erected vertically	Prop size 1 or 2		35	35	35	34	27	23						-
	Prop size 3					34	27	23	21	19	17			
	Prop size 4							32	25	21	18	16	14	12
TABLE B Props loaded concentrically and erected 1½° max. out of vertical	Prop size 1 or 2 or 3		35	32	26	23	19	17	15	13	12			
	Prop size 4							24	19	15	12	11	10	9
TABLE C Props loaded 25 mm eccentricity and erected 1½° max. out of vertical	Prop size 1 or 2 or 3		17	17	17	17	15	13	11	10	9			
	Prop size 4							17	14	11	10	9	8	7
TABLE D Props loaded concentrically and erected 1½° out of vertical and laced with scaffold tubes and fittings	Prop size 3					35	33	32	28	24	20			
	Prop size 4							35	35	35	35	27	25	21

Shear V = (14.6kN + 13.4kN) /2 = 14.000kN

Any Acro Prop is accetpable

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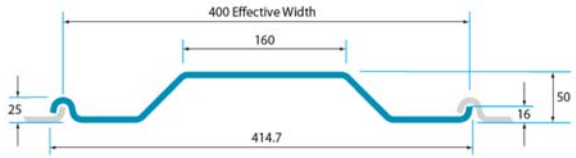
KD4 SHEETS

KD4

The overlapping trench sheeting profile is a heavier version of the Standard Lap, with a wider gauge and width coverage, designed in large for construction work.



Technical Information

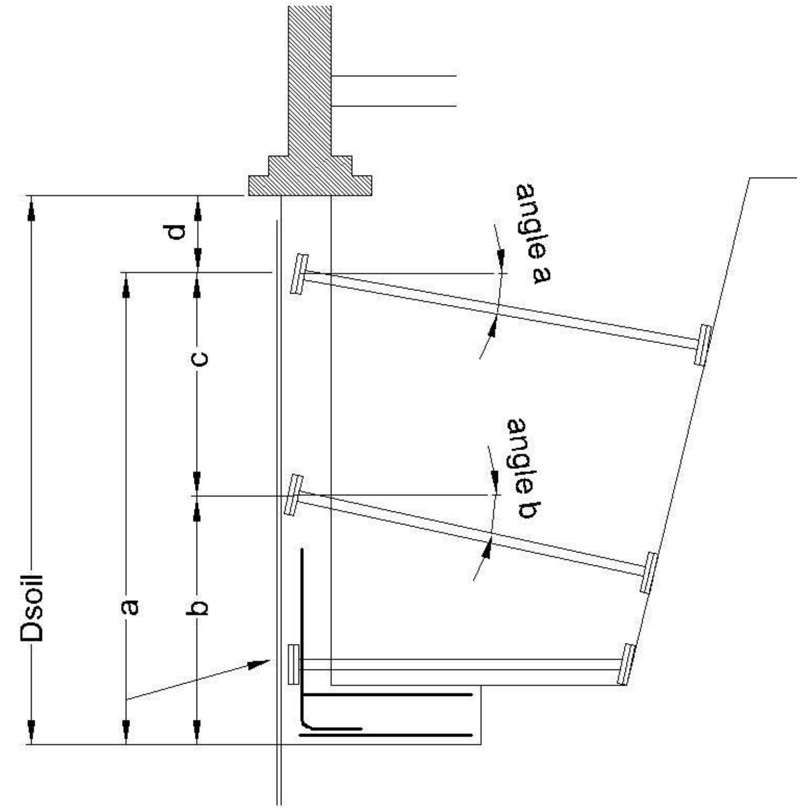
Effective width per sheet (mm)	400
Thickness (mm)	6.0
Depth (mm)	50
Weight per linear metre (kg/m)	21.90
Weight per m^2 (kg)	55.2
Section modulus per metre width (cm^3)	101
Section modulus per sheet (cm^3)	40.34
I value per metre width (cm^4)	250
I value per sheet (cm^4)	101
Total rolled metres per tonne	45.659

Sxx = 48.3cm^3

py = 275N/mm^2

Ixx = 26.9cm^4

A = (1m^2 * 55.2kg/m^2) / (400mm * 7750kg/m^3) = **17806.452**mm^2

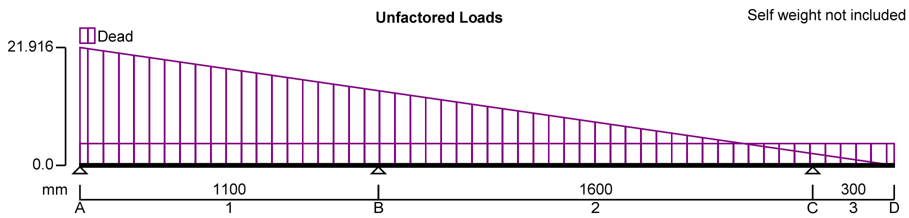


Length a a = **2.700** m

Length b bottom b = **1.100** m

Length c Middle c = a – b = **1.600**m

Length d top d = Dsoil – a = **0.300**m



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **3**

Material Properties:

Modulus of elasticity = **205** kN/mm^2 Material density = **7860** kg/m^3

Support Conditions:

Support A	Vertically **"Restrained"**	Rotationally **"Free"**
Support B	Vertically **"Restrained"**	Rotationally **"Free"**
Support C	Vertically **"Restrained"**	Rotationally **"Free"**
Support D	Vertically **"Free"**	Rotationally **"Free"**

Span Definitions:

Span 1	Length = **1100** mm	Cross-sectional area = **17806** mm^2	Moment of inertia = **269.×10^3** mm^4
Span 2	Length = **1600** mm	Cross-sectional area = **17806** mm^2	Moment of inertia = **269.×10^3** mm^4

Span 3 Length = **300** mm Cross-sectional area = **17806** mm^2 Moment of inertia = **269.**$\times$**10**^{3} mm^4

LOADING DETAILS

Beam Loads:

Load 1 VDL Dead load **21.9** kN/m to **0.0** kN/m

Load 2 UDL Dead load **4.1** kN/m

LOAD COMBINATIONS

Load combination 1

Span 1 1×Dead

Span 2 1×Dead

Span 3 1×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Support Reactions - Combination Summary

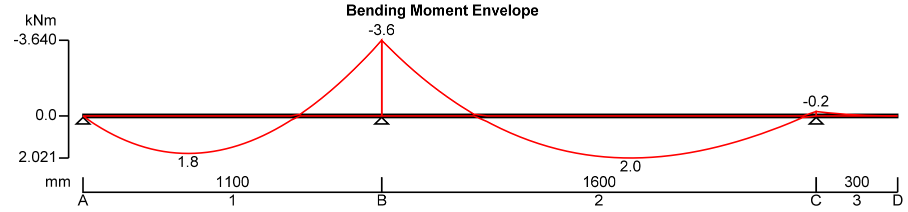
Support A	Max react = **-9.5** kN	Min react = **-9.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-28.0** kN	Min react = **-28.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-7.5** kN	Min react = **-7.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support D	Max react = **0.0** kN	Min react = **0.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm

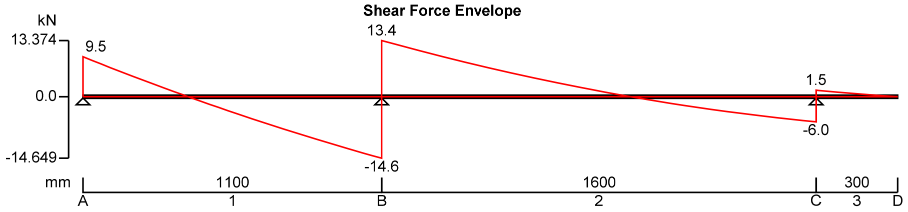
Beam Max/Min results - Combination Summary

Maximum shear = **13.4** kN Minimum shearF_{min} = **-14.6** kN

Maximum moment = **2.0** kNm Minimum moment = **-3.6** kNm

Maximum deflection = **7.7** mm Minimum deflection = **-4.9** mm





Number of sheets Nos = 2

Mallowable = Sxx * py * Nos = **26.565**kNm

CROFT STRUCTURAL ENGINEERS

SRU 4.0

Safe working loads for Acrow Props — loads given in kN

For normal purposes 1 kilo Newton (kN) = 100 kg	Height m / ft	2.0 / 6.6	2.25 / 7.4	2.5 / 8.2	2.75 / 9.0	3.0 / 9.8	3.25 / 10.7	3.5 / 11.5	3.75 / 12.3	4.0 / 13.1	4.25 / 13.9	4.5 / 14.8	4.75 / 15.6
TABLE A Props loaded concentrically and erected vertically	Prop size 1 or 2	35	35	35	34	27	23						-
	Prop size 3				34	27	23	21	19	17			
	Prop size 4						32	25	21	18	16	14	12
TABLE B Props loaded concentrically and erected 1½° max. out of vertical	Prop size 1 or 2 or 3	35	32	26	23	19	17	15	13	12			
	Prop size 4						24	19	15	12	11	10	9
TABLE C Props loaded 25 mm eccentricity and erected 1½° max. out of vertical	Prop size 1 or 2 or 3	17	17	17	17	15	13	11	10	9			
	Prop size 4						17	14	11	10	9	8	7
TABLE D Props loaded concentrically and erected 1½° out of vertical and laced with scaffold tubes and fittings	Prop size 3				35	33	32	28	24	20			
	Prop size 4						35	35	35	35	27	25	21

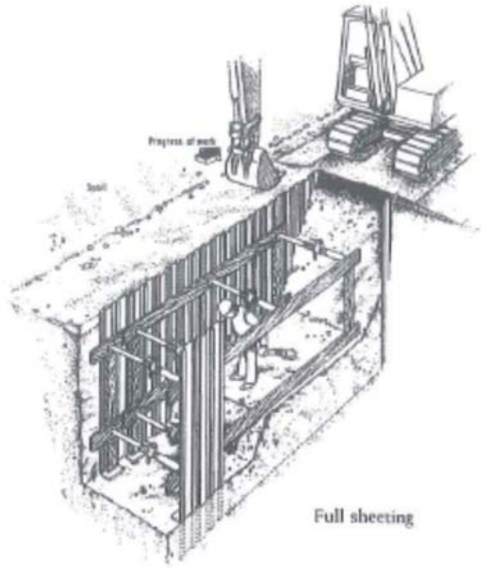
Shear V = (14.6kN + 13.4kN) /2 = 14.000kN

Any Acro Prop is accetpable

Sheeting requirements

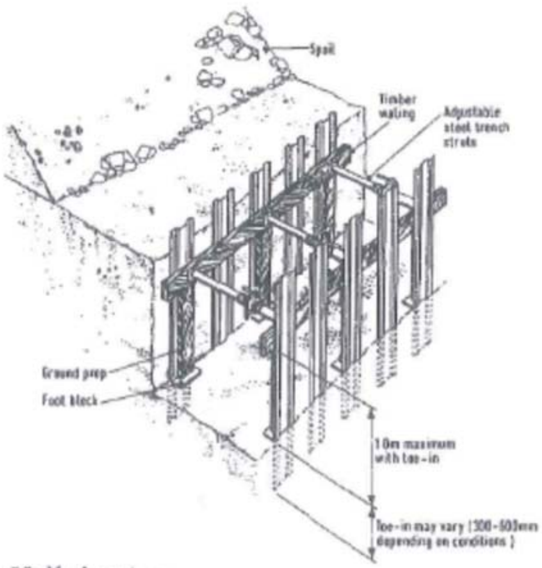
Ground Type	Trench Depth, D: ~~less than 1.2m~~ [1]	1.2 to 3m	3 to 4.5m	4.5 to 6m
Sands and gravels Silt Soft Clay High compressibility Peat	~~Close, ½, ¼, ⅛ or nil~~	Close	Close	Close
Firm/stiff Clay Low compressibility Peat	~~¼, ⅛ or nil~~	½ or ¼	½ or ¼	Close or ½
Rock[2]	From ½ for incompetent rock to nil for competent rock[3]			

Sheeting requirements



Full sheeting

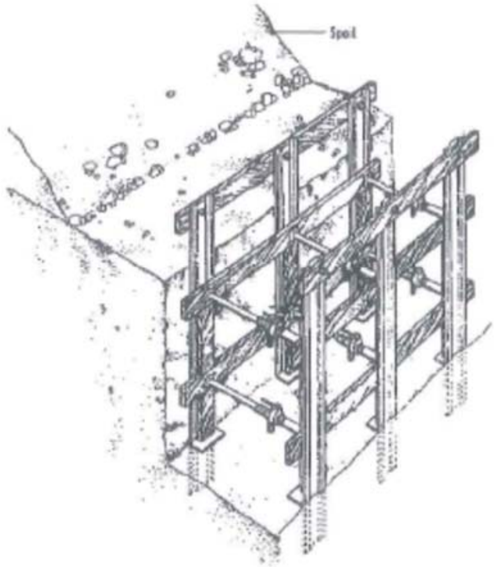
Sheeting requirements



Half sheeting
shown for 1.5 m deep trench

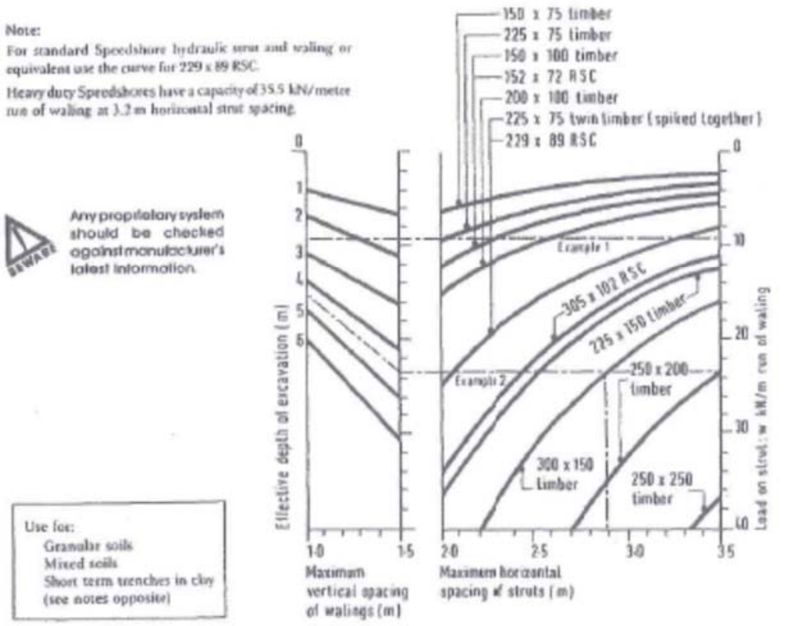
11/04/2012

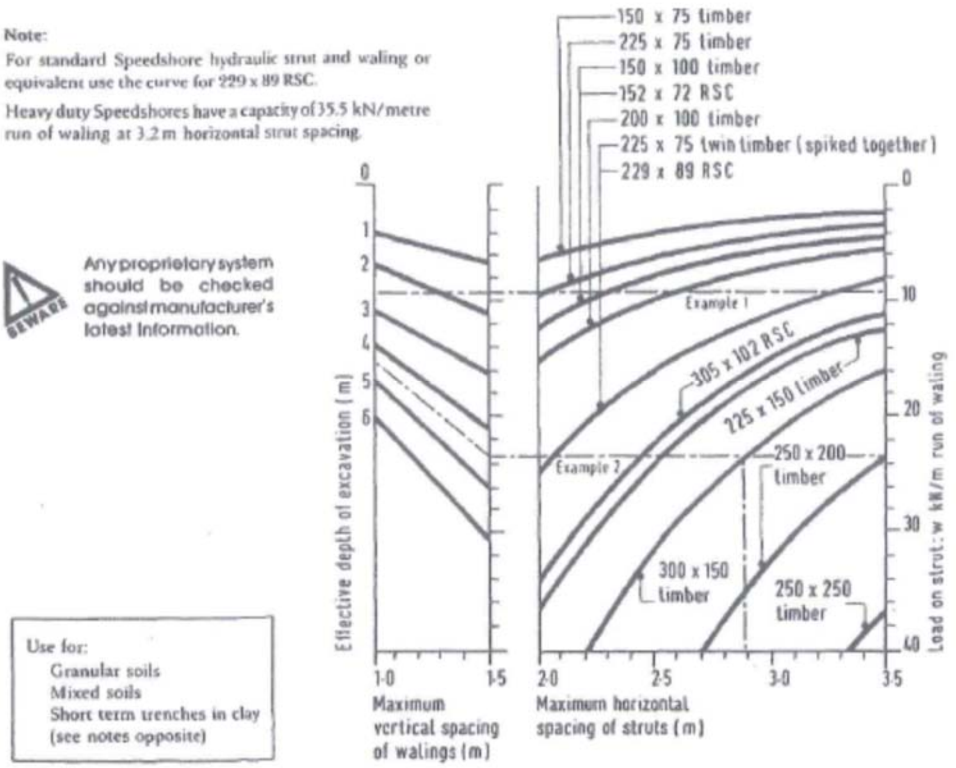
Sheeting requirements



Quarter sheeting

Design to CIRIA 97




Note:
For standard Speedshore hydraulic strut and waling or equivalent use the curve for 229 x 89 RSC.
Heavy duty Speedshores have a capacity of 35.5 kN/metre run of waling at 3.2 m horizontal strut spacing.
Any proprietary system should be checked against manufacturer's latest information.
BEWARE
Use for:
Granular soils
Mixed soils
Short term trenches in clay
(see notes opposite)
150 x 75 timber
225 x 75 timber
150 x 100 timber
152 x 72 RSC
200 x 100 timber
225 x 75 twin timber (spiked together)
229 x 89 RSC
305 x 102 RSC
225 x 150 timber
250 x 200 timber
300 x 150 timber
250 x 250 timber
Example 1
Example 2
Effective depth of excavation (m)
Load on strut: w kN/m run of waling
Maximum vertical spacing of walings (m)
Maximum horizontal spacing of struts (m)

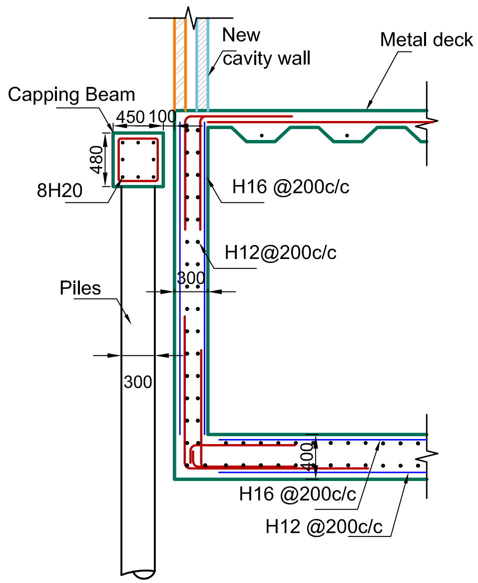
CROFT STRUCTURAL ENGINEERS

Appendix B : Structural Drawings

1:100 Basement Plan on A3 Showing Neighbouring basements if present

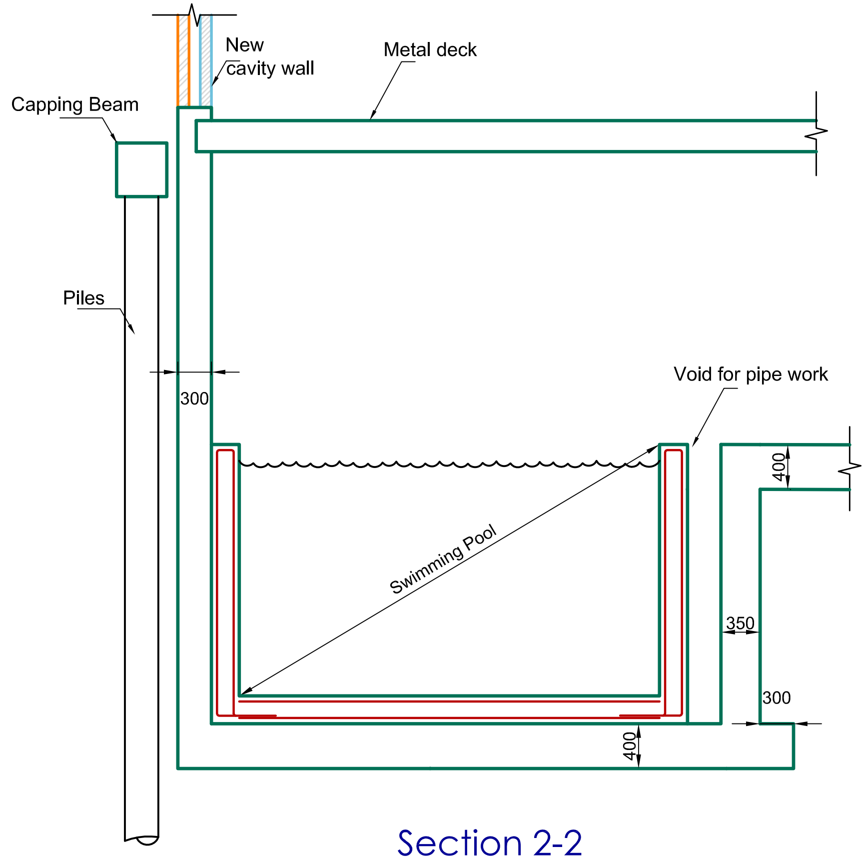
1:100 Ground Floor plan on A3 Showing Neighbouring property

1:50 Section on A3 Including section through Neighbouring Footings



Section 1-1

Scale (1:50)



Section 2-2

for Reinforcement see section 1-1

Scale (1:50)

Rev	Date	Amendments
-	14/08/15	First issue for comment

Job No.s 150607		Client: Mike Ofori
Dwg Nos SL-30		Project: 1b St Johns Wood Park
	Date July15	Title : Sections
Drawn SB	Chk'd NM	
Scale As shown @ A3	Rev -	

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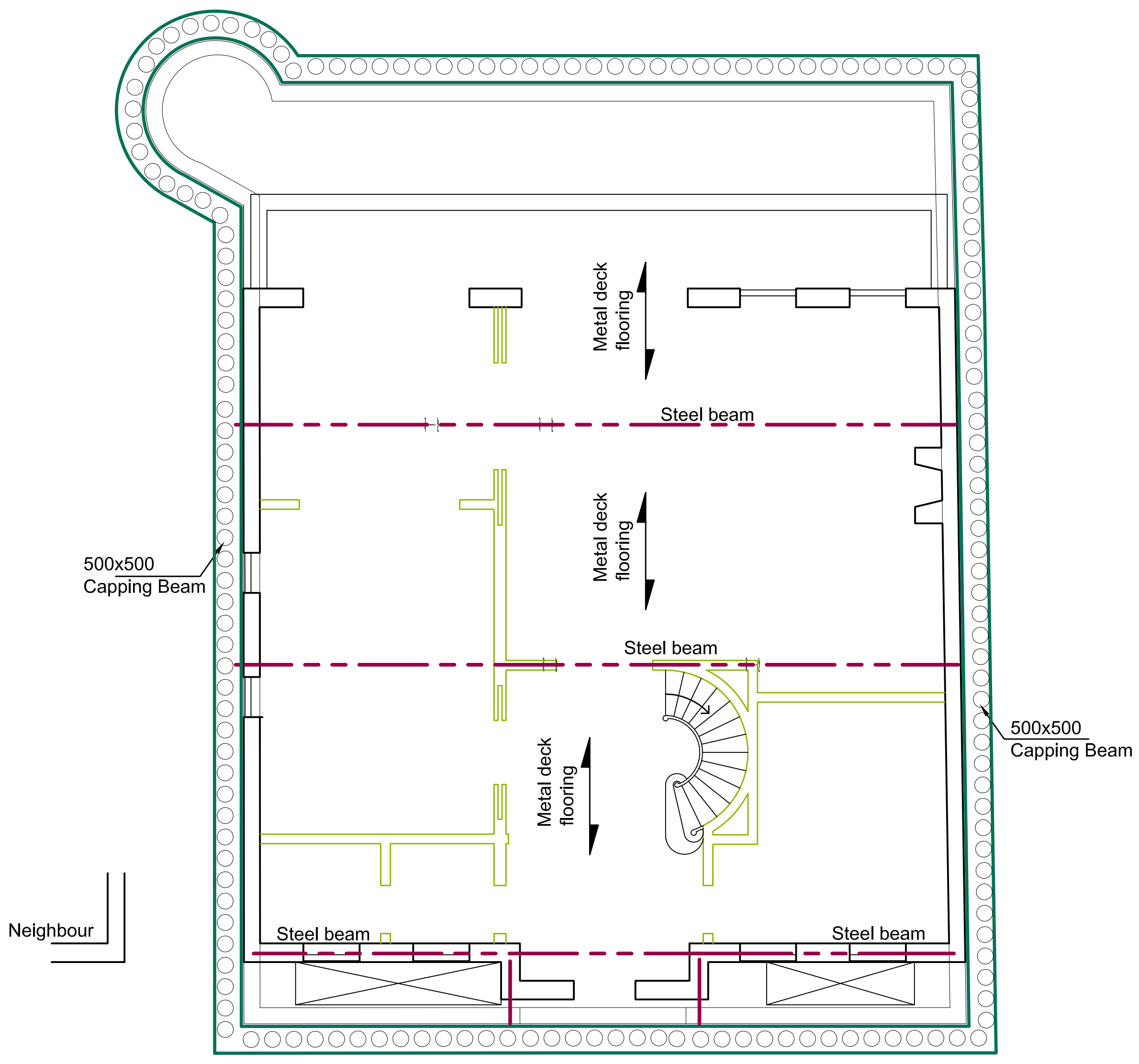

Very High Levels Of Lead Were Found In The Soil. Lead Specialists Are To Be Called To Treat Soil Before Construction

Ground floor plan

Scale (1:100)

General Notes:

1. Structural timber to be strength class C24, unless noted otherwise, in accordance with BS5268. All double & multiple timbers to be bolted with M10 at450c/c. All new timber in works to be tanalised with cut ends treated before fixing. New timber connections to have proprietary galvanised steel fixings. Timber to masonry connections by joist hangers unless shown otherwise. Noggings, minimum 38mm width to extend at least 1/2 the depth of the joist, spaced at 1/3 points along joist span.

2. Double up timber joists under all new partition walls and velux windows.

3. Masonry to be in accordance with BS5628, Class (ii) above DPC and Class (i) ot be used below DPC's and to chimneys. New brickwork to be carefully bonded to existing. Block bonding is not permitted for exposed masonry brickwork. Block contraction joints required at 6m c/c and brickwork expansion joints at 12m c/c. Where existing new masonry meets existing masonry stainless steel furfix connections are required. Provide stainless steel EML Bed joint reinforcement two course (150 and 300 spacing) above and below all new window and door openings. Below DPCS, all masony to the Frost resistant. Block work below DPC to be sulphate resistant.

4. Padstones, required under all new beam bearing onto masonry, to be 1:1.5:3 mix, (C30). Or PC Lintels if noted. 15mm thick Plate can be used with engineers approval.

5. Dry packing to be to be 2:1 Sand:Cement mixed to a "damp" consitstance. Beams over 5m and underpins Dry pack to contain Fosroc CBex 100. Dry pack to be well rammed in. 48 hours to be left from drypacking to removal of any temporary supports.

6. The main contractor is to be responsible for the temporary stability of existing structures and earthworks on the site and adjoining sites, and must take all necessary precautions to safeguard this stability. Details of propping/needling and method statement to be provided to Engineer prior to commencing works.

7. Foundations designed on an assumed bearing pressure of 100kN/m². Formation level to be 1200 min to external foundations and 1000 for internal. Footings to extend 300mm below any roots found. Formation to be inspected & agreed on site to the satisfaction of the Building Control Officer.

8. Any drain run undermining existing foundations to be encased in minimum 100mm, grade C20 concrete.

9. Existing lintels to be inspected and replaced if showing signs of deterioration.

10. Existing masonry to be inspected. Where cracked or debonded repairs as specified if not repairs are specified contact the engineer. Existing walls to be checked for lateral restraint. If restraint is inadequate provide lateral restraint.

11. Provide Lateral Restraint straps (1200x30x3) at 1200centres to floors and roof. Provide Holding Down straps (1200x30x3) at 1200centres roof sole plate.

12. Use Ancon ST1 Wall ties for new cavity over 75mm. Fix at standard spacing. Less than 75mm cavity standard wall ties to be used.

Rev	Date	Amendments
1	26/08/15	Ground floor layout changed
-	14/08/15	First issue for comment

Job No.s 150607	Client: Mike Ofori
Dwg Nos SL-20	Project: 1b St Johns Wood Park
Date July15; Drawn SB; Chk'd NM	Title : Ground Floor plan BIA
Scale As shown @ A3; Rev 1	

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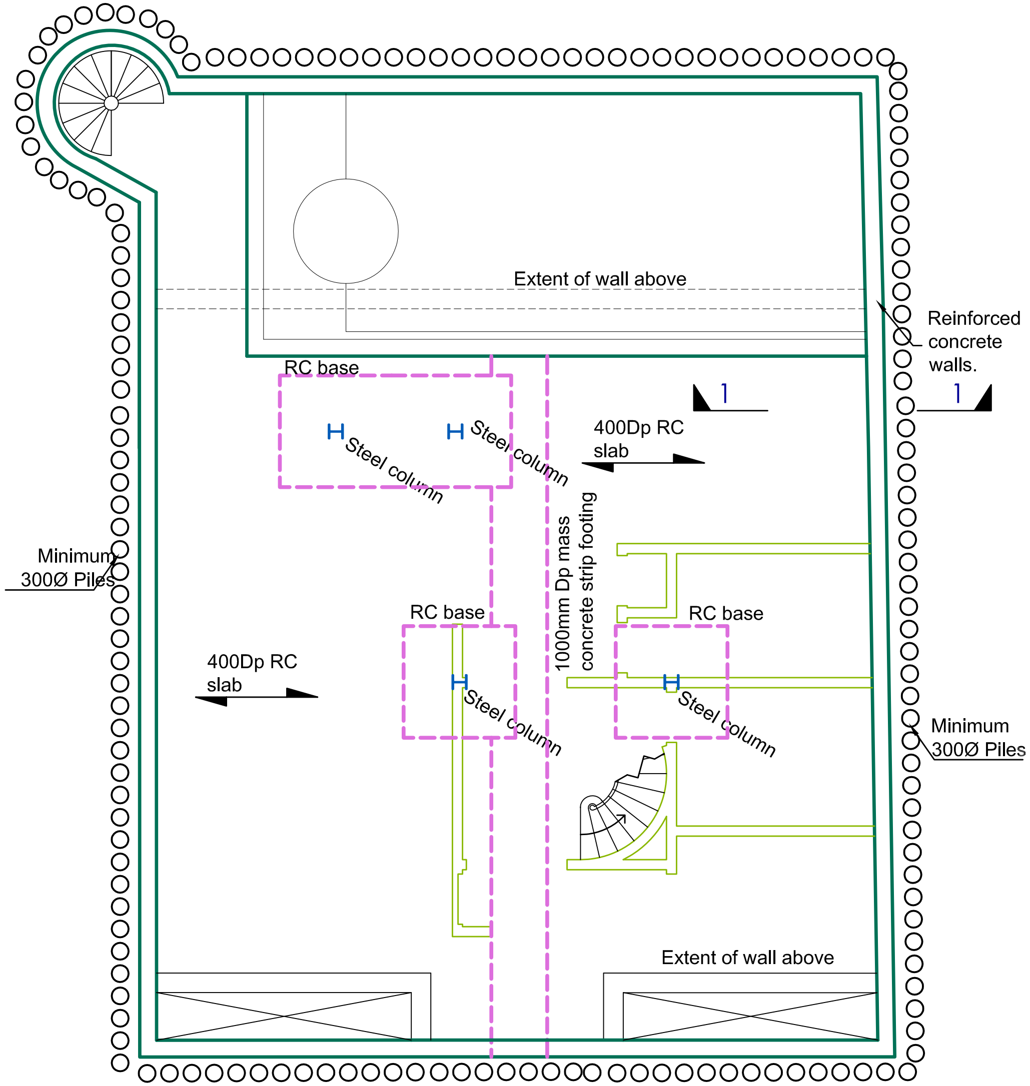

Basement plan

Scale (1:100)

General Notes:

1. USE ONLY FIGURED DIMENSIONS. All dimensions in mm's. Refer to Architect's drawings for setting out. This drawing is to be read in conjunction with all relevant Architects, subcontractors and engineers drawings and specifications. Final co-ordination of cladding, drainage, insulation, steelwork, and other elements is the responsibility of the contractor.

2. All dimensions and levels shown are based on survey drawings. The contractor is to satisy themselves that dimensions levels etc are sufficiently accurate to complete construction to the necessary tolerances. Existing structure to be verified on site by the contractor and any discrepancies reported immediately to Engineer.

3. Domestic jobs: the contractor is to notify the local H.S.E. area office of the works using form F10 (rev.) in accordance with the C.D.M. regulations, 2007. A copy of the notification is to be displayed on site and copied to the Engineer. The client must appoint a CDM co-ordinator and comply with CDM Regulations for all projects which are not their private residency.

4. Imposed load design Typical Domestic 1.5kN/m²

5. Concrete to be in accordance with BS8110. Concrete for mass concrete foundations to be To FND 3 in accordance with BS8500 (minimum strength 35N/mm², 20mm maximum aggregate size, 75mm slump and ordinary Portland Cement). Reinforced concrete to be RC28/35 min (previous designation C35N/mm²) unless noted otherwise.Minimum Cement contents 320kg/m3, Water cement ratio 0.55.2 Cubes to be taken for every 10m3, or every pour, and 1 tested at 28 days with the results provided to the engineer.

6. Reinforcement required is noted on the drawings or in the calculations as either areas of reinforcement or bar/mesh requirements. Schedules are to be completed by the contractor and provided tot he engineer 1 week before ordering. Reinforcement schedules to be completed in accordance with BS8666:2005or BS EN ISO 3766

7. Water proofing, damp proofing and all weather proofing are not the responsibility of Croft Structural Engineers. Basement water proofing is the responsibility of the specialist waterproofing contractor. Croft are not the Structural Water-proofer. The specialist water proofing contractor must review our drawings and design and if greater waterproofing resistance is required then Croft are to be informed and the additional requirements will be added to the plans.
   7.1. The Specialist water-proofer must provide their drainage layout and sump locations to Croft Structural Engineers 2 weeks prior to installation.
   7.2. Pipes below slab to have be encased in 150mm of concrete. Pipes within slab to have a minimum of 150mm concrete around them.
   7.3. Grace Adcor ES waterstop is to be added to all day joints and construction joints in the basement. If high water table encountered include Caltite admixture to the concrete.
   7.4. Dewatering must be turned off 2 weeks before internal drain cavity is fixed. Any leaks are to be plugged in accordance to SIKA's specifications.

8. Structural steelwork to be in accordance with ADVANCED275JR internally , for high grade steel use ADVANCED355JR internally. BS5950 for design detail and workmanship. Steelwork must be fabricated in accordance with BS EN 1090. Fabricated Steelwork must be provided with a CE Mark, FPC, RWC and WQMS. All structural work and fire protection to the satisfaction of the Building Control Officer.

External Steel - ADVANCED275JR up to 15mm, above 15mm use ADVANCED275J0. For high grade steel use ADVANCED355JR up to 11mm for external use and ADVANCED355J0 above 11mm

9. All Steel to be painted: prepared by grit blasting in accordance with BS7079, the standard of surface cleanliness is to Swedish Standard SA2.5. Paint specification to be in accordance with BS5493. In shop applied high build Red zinc phosphate modified alkyd, to 75 microns. On site, degrease and touch up as necessary using high build zinc phosphate modified alkyd to 60 microns. Thicknesses are dry film thicknesses. Steelwork built into cavity to be galvanised inaccordance to EN ISO 1461 with a minimum 85μm thickness. Site repairs to galvanising to be completed with Cold Galvafroid or similar. Concrete Encased steelwork to have 2 additional site coats of bitumen paint.

10. Unless noted otherwise, steelwork welds to be minimum 6mm fillet weld, all bolts to be grade 8.8 with minimum 16mm diameter. Overall lengths & Connection design is the responsibility of the contractor and details where shown are indicative. Where loads are shown on the drawings, V = Factored Shear loads, M = Factored Moments. Connection Calculations, Fabrication details are to be provided by fabricator to the Engineer prior to fabrication for connection approval and to the Architect for setting out approval. Minimum 2M16 per connection and take 75kN tie force, 80kN shear unless noted otherwise. Bolt all double beams together with M16 at 600c/c with Spacer tubes. Where columns sit against masonry bolt back with M16 Resin Anchors at 450c/c staggered either side of flange. Welding to comply with BS EN 288. Site welding if essential to be tested in accordance with the national steelwork specification and results provided to the engineer.

11. Contractor MUST provide fabrication drawings & connection calculations to the engineers two weeks prior to fabrication for approval, final appearance to be agreed with the architect.

Rev	Date	Amendments
2	28/08/15	Basement floor plan changed
1	26/08/15	Basement floor plan changed
-	14/08/15	First issue for comment

Job No.s 150607		Client: Mike Ofori
Dwg Nos SL-10		Project: 1b St Johns Wood Park
	Date July15	Title : Basement plan BIA
Drawn SB	Chk'd NM	
Scale As shown @ A3	Rev 2	

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