Job Number: 150525 (35 Greville Road)
Date: 24 Aug 15

CROFT STRUCTURAL ENGINEERS

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Basement Impact Assessment

Property Details
35 Greville Road
London
NW6 5JB

Client Information
Igor Gokhberg
55 Peiantho CRT
Vaughan
ON
LGA 1G1
Canada

Structural Design Reviewed by Chris Tomlin MEng CEng MIStructE	**Above Ground Drainage Reviewed by** Phil Henry BEng MEng MICE
Hydrology Report Hannah Fraser CGeol Separate report	**Geology Report** Jon Smithson CGeol Separate Report
Report compiled by Noma Manzini MEng BEng	

Revision	Date	Comment
-	Aug 2015	First issue

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 property is a detached dwerlling over three floors with a loft storage space area. The construction is load bearing masonry walls externally and internally with concrete floors at lower ground floor, ground floor and at first floor. Timber floors in loft storage space.

Figure 1: Side elevation

	Proposed Works The proposed works require the construction of: • A new basement under the property. • Light wells to the front and rear • Garden basement o Roof slab to the garden o SUDS (Storm water storage above the garden area) o Covering garden slab with new top soil 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 35 GREVILLE ROAD is: 1. Place a contiguous pile wall around the perimeter of the garden area & light wells 2. Excavate front to allow for conveyor to be erected. 3. Safely and securely support the existing building above 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. The property was inspected and 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.

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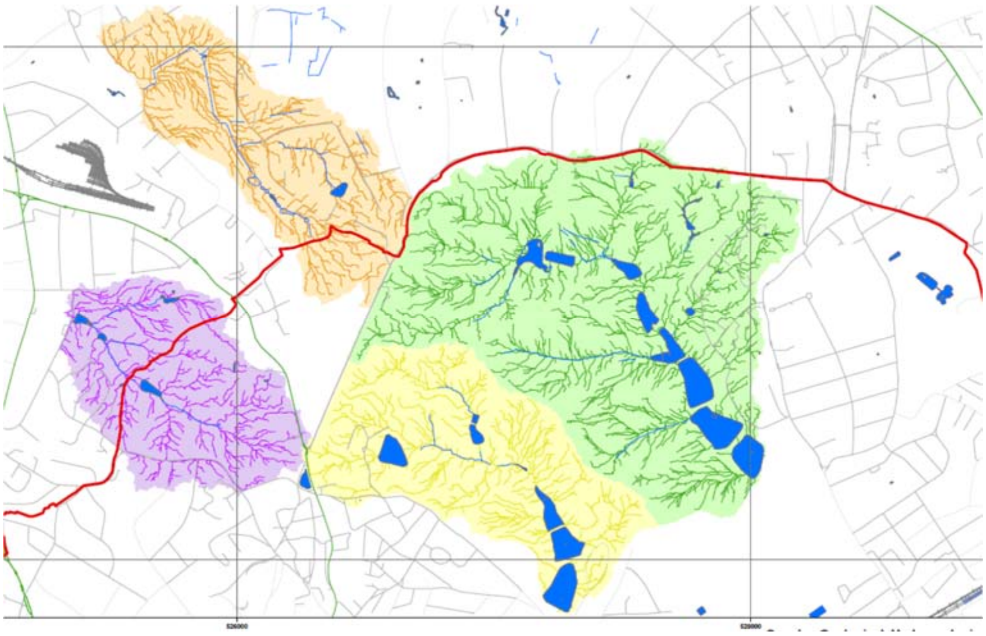
Stage 3 – Site investigation and study	A Chartered Structural engineer inspected the building to determine the current condition of the property. Visual inspections were completed of the adjacent properties to determine if there were signs of structural movement. The neighbouring land has not been excavated 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 deep boreholes has been completed. • London Clay Formation Laboratory testing was undertaken on the soil samples. Ground water has been measured over repeat visits to determine water levels and flows. • Perched water was found at 0.85m BGL
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 is concluded that with the construction of the new basement at 35 Greville Road should not have significant impacts on land stability provided that: • Groundwater inflow, if encountered is properly controlled and is monitored before, during and after construction. • The construction of the basement is carried out by a competent who will adopt suitable measures to maintain the stability of the excavations • Care is taken to minimise disturbance to trees and their roots. • Concrete is designed to account for the sulphate conditions anticipated. • Monitoring of the structures is carried out before, during and after construction. **Hydrogeology** Groundwater inflow if encountered is reduced to a minimum and properly controlled such that there is no significant wash out of fine material. Groundwater levels should be monitored before and during construction.

	Drainage & Surface Water Flow Ground water was not observed during drilling at the site, however, groundwater levels were observed at 0.83 and 0.94mBGL on subsequent monitoring visits. The direction of groundwater flow is not known at the site.

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 2: Extract from figure 14 of the Hydreological Sturdy* **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? **Unknown** –The Garden basement may reduce the impermeable areas. Carry forward to scoping
	Question 3. Will the proposed basement development result in a change to the hard surfaced /paved external areas? **Unknown** –The Garden basement may reduce the impermeable areas. Carry forward to scoping
	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 scoping
	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.

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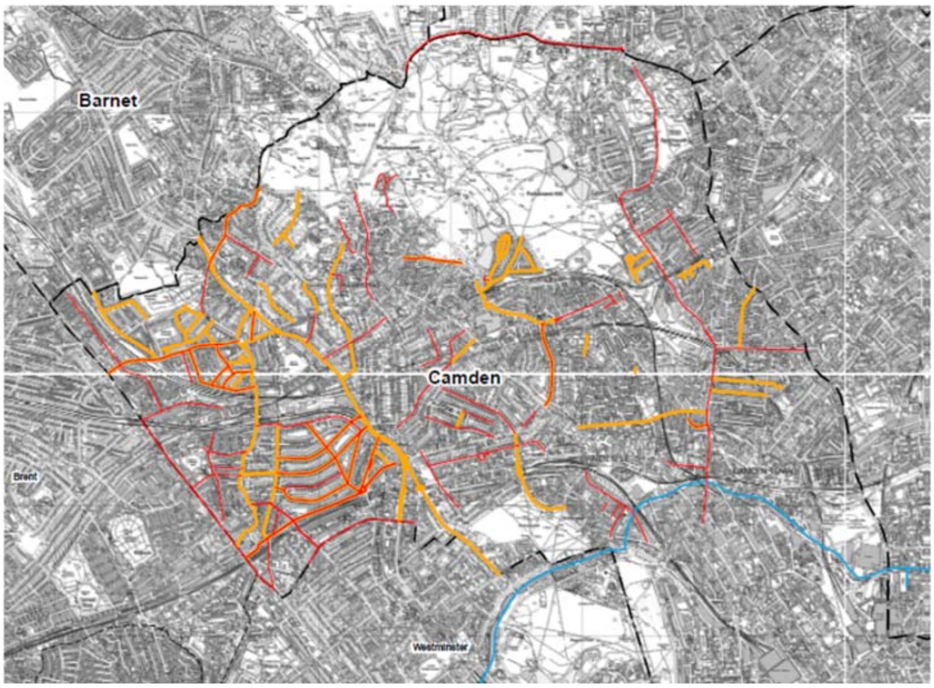
Surface water (pluvial) flooding	NO	The 35 GREVILLE ROAD is noted on the flood street list and maps from 1975 or 2002



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.

Yes the site is noted Carry forward to scoping stage

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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 35 GREVILLE ROAD require in insertion of a basement. The basement is under the footing print of the property which will not affect the overall flow. The basement enlarges the existing single dwelling and is not an additional unit. The Garden basement may decrease the permeable areas and this may increase the surface water flows and further investigations should be undertaken.
	Question 1: **Is the site within the catchment of the pond chains on Hampstead Heath?** **No** further info required from Scoping stage **Yes** – Carry forward to Basement Impact Assessment 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? **Unknown** –The Garden basement may reduce the impermeable areas. Carry forward to Site Investigation & desk Study
	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 Garden basement may reduce the impermeable areas. Carry forward to Site Investigation & desk Study

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	~~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 35 GREVILLE ROAD 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 Noma Manzini, an Engineer from Croft Structural Engineers visited 35 GREVILLE ROAD. Date of inspection was on the 16th of June
Proposed Development	The existing property is a detached dwerlling over three floors with a loft storage space area. The construction is load bearing masonry walls externally and internally with concrete floors at lower ground floor, ground floor and at first floor. Timber floors in loft storage space. **Location** The property is located in a built up area. Mature trees are present in the vicinity. The surrounding area is relatively flat with a slight slope downwards from north-west to south-east.  *Figure 3: Rear elevation*

Figure 4: Rear ground elevation

Site History

What was the previous usage of the site?

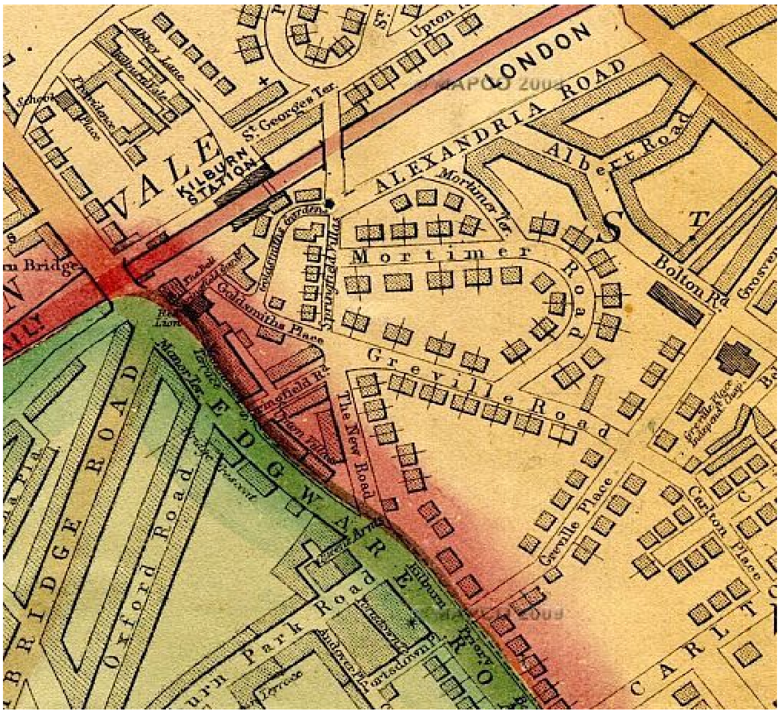

Figure 5: Map of London 1868 - Edward Weller

The site and visicnity have been residential for over one hundred years.

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'.

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<table>
<tr>
<td>Local Bombing</td>
<td>


Figure 6: Bomb sight map

A high-explosive bomb, is recorded in the Aggregate Night Time Bomb Census as having been dropped between 7 October 1940 and 6 June 1941, close to the site.
</td>
</tr>
<tr>
<td>Listed Buildings</td>
<td>

Is the building or Adjacent buildings listed

No the propert is not listed but the adjacent building is listed

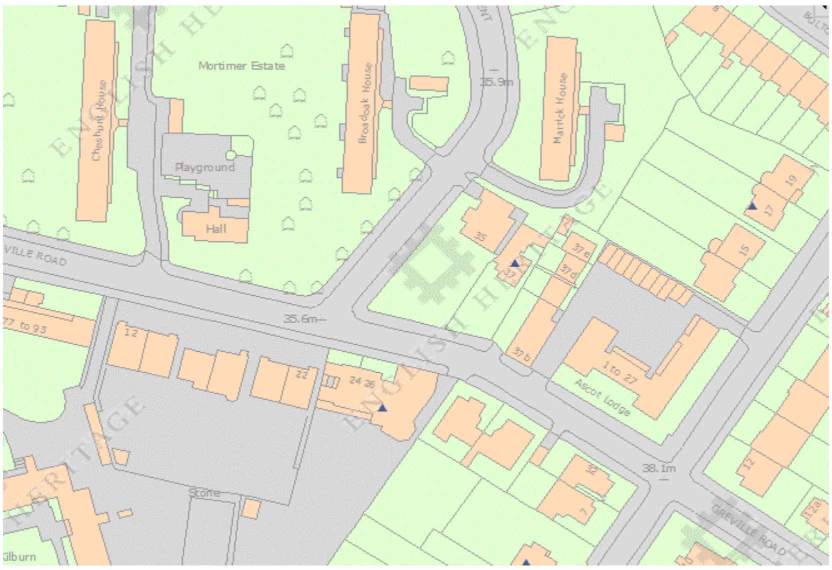


Figure 7: Listed building map
</td>
</tr>
</table>

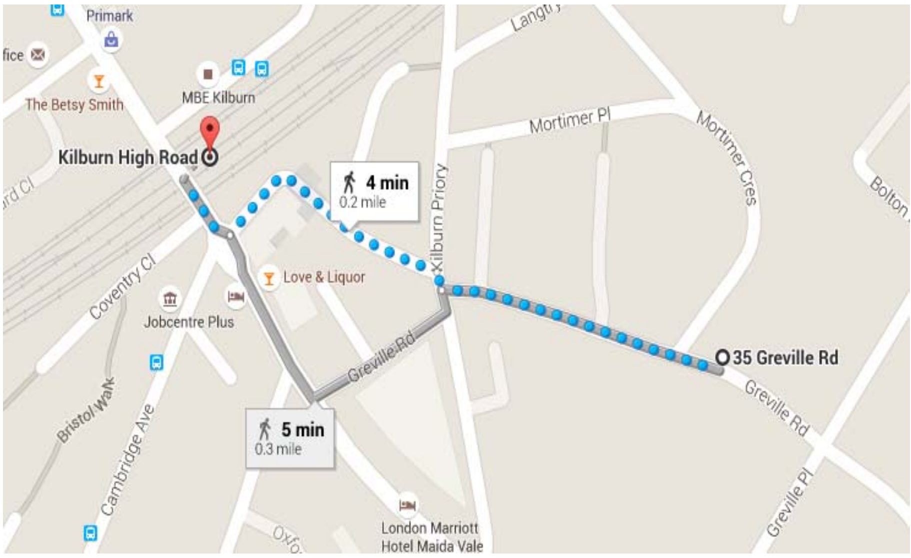
	Highways, Rail and London Underground **Yes.** Site is within 5m of the footpath/alleyway and the road surface is further than 5m from the front lightwell.
London Underground and Network Rail	**Is the site over (or within the exclusion zone) of any tunnels, e.g. railway lines?** **No.** Nearest is the 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?** (Substations etc)
Vicinity of Trees	Some shrubbery and general vegetation in the neighbouring garden; A mature tree is also present in the neighbouring garden. Are any trees to be removed due to the basement? No trees will be removed
Building Defects	A visual inspection was undertaken of the existing building with particular attention given to movement to the building. The defects noted were: • Fine cracking was noted above ground floor door lintel • Fine to moderate cracking was noted on the garage walls Structural Assessment of ongoing movement:

	 *Figure 9: Cracking on garage walls*
	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 10: Areal map of 35 Greville Road and 37 Greville Road*
Nos 37 Greville Rpad Property to right	Property Age : Over 100 years old Property use : Residential Number of storeys : 3

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	Is a basement present? Planning permission for a new basement was granted on the 18-04-13. It is not know whether this was subsequently built.  Figure 11: 37 Greville Road front elevation  Figure 12: 37 Greville Road rear elevation
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	Refer to the ground investigation report by Ground and Water, which is submitted as a separate document.

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Geology	Refer to the ground investigation report, the hydrogeology report and the land stability assessment, submitted separately.
Surface Flow & Flooding	Refer to the ground investigation report, the hydrogeology report and the land stability assessment, submitted separately.
Areas of Hard Standing present on site	Existing Area of hardstanding outside is ; Area = approximately 325m^2
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 Ground investigation specialists visited the site and subsequently produced are report for the existing ground and groundwater conditions.
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.

Site Investigation

Soil investigation Brief	The Soil investigation was completed by (Soil investigation Company). From the Scoping stage we considered that their brief should cover: • A trial pit to the front side to confirm the existing foundations. The purpose is to consider the effect of the works on the neighbouring properties and the find the ground conditions below the site. • It would have been preferred to complete two bore holes on this site.. With the size of site, and our knowledge of the area it is not expect for there to be a large variation across the small site, therefore one borehole 5m deep was completed. • 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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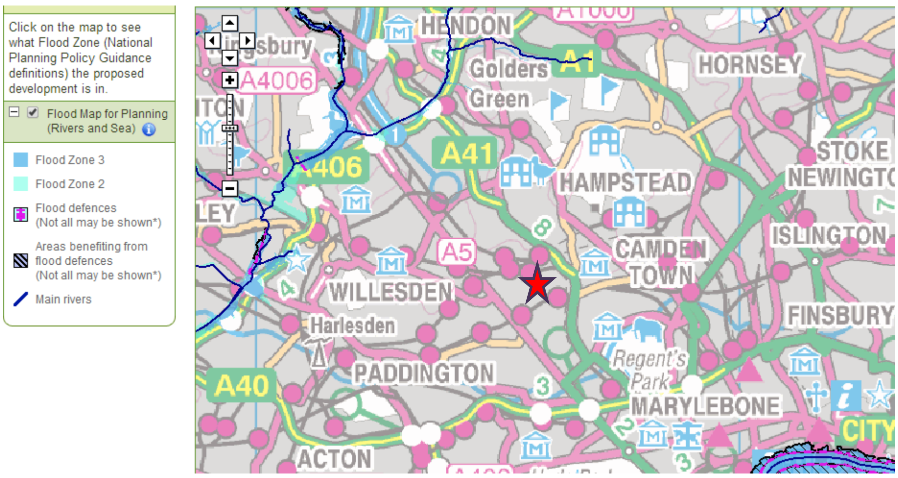
4. Basement Impact Assessment

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 the this BIA document and is not within the Croft Structural Engineers Brief.

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. This must be preceded by a Flood Risk Assessment (FRA), and is staged as follows: • A subsequent scoping study to consider further the identified sources, assessing the risks proposing measures to mitigate them.

Site Location	The site is approximately 325m² in size. It is located in a densely built-up area. From inspection of OS contours, the site appears to lie on ground which slopes down from north to south, by approximately 1 in 50. Residential houses exist either side of the site. These buildings are at the same level. There are gardens to the front and rear of the site. Immediately to the front, this road is relatively flat. The nearest water course is 250km away The EA has not identified any flood risks associated with the nearby water courses.  *Figure 13: Flood map for planning (Environment Agency)* The site is within Zone 1, a low probability flood risk area.
Potential surface water (pluvial) flooding	35 GREVILLE ROAD basement lies on low point on Greencroft Gardens. Any surface water runoff would be directed to this section of the road. It is likely that this area of road would have been flooded in 2002. 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.

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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 35 GREVILLE ROAD. Blockage or failure of either of these may result in the following sequential events: • Excess flow from 35 GREVILLE ROAD may accumulate in the area of road in front of the site. • This flow would travel in the direction away from the front elevation of the property owing to the site being on a slightly higher level than the opposite side of the street, and the raised level of the pavement above the road (see photo below). The likelihood of flow into the front light wells is also reduced by the existing landscaped areas in the front garden: these would partially relieve any excess flow that would migrate towards the front of the building. 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.
Mitigation measures	We would recommend the following measures to reduce the risks mentioned above: • 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.
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.

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SUDS Assessment

<table>
<tr><td>Hard standing</td><td>Existing Hard Standing = 325 m^2

Proposed Hardstanding = 328 m^2

Percentage Increase in Hard standing = 0.9 %</td></tr>
<tr><td>SUDS Assessment</td><td>From review of the existing and proposed hardstanding the increase will be?

0.9 %

<table><tr><td>Percentage Increase < 5%</td><td>No SUDS to be incorporated into scheme</td></tr><tr><td>Percentage Increase Between 5% to 10%</td><td></td></tr></table>
Where garden basements are present then a soil band of a minimum of 1m should be provided.

Where the soil cover is greater than 1m of soil is not present then SUDs is not required</td></tr>
<tr><td>SUDS Calculations</td><td>As explained above. SUDS calculation is not required</td></tr>
<tr><td>Mitigation Measures</td><td>As explained above. SUDS calculation is not required</td></tr>
<tr><td>Drainage effects on Structure</td><td>Not build over agreements known of.

Flooding. The site is not in an area of high risk flooding.</td></tr>
</table>

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Trees	
Root Protection Zone	RPA = 1.5 x Crown diameter. The basement is within the RPA of the trees.
Conclusion	The Basement does Not 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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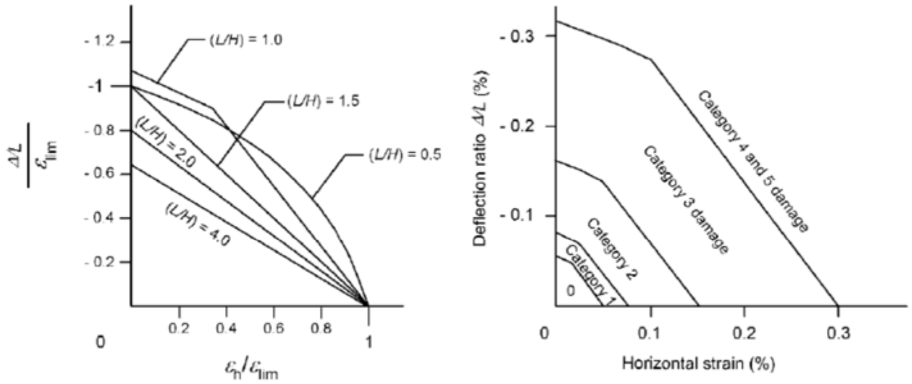
Ground Movement Assessment & Predicted Damage Category

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)

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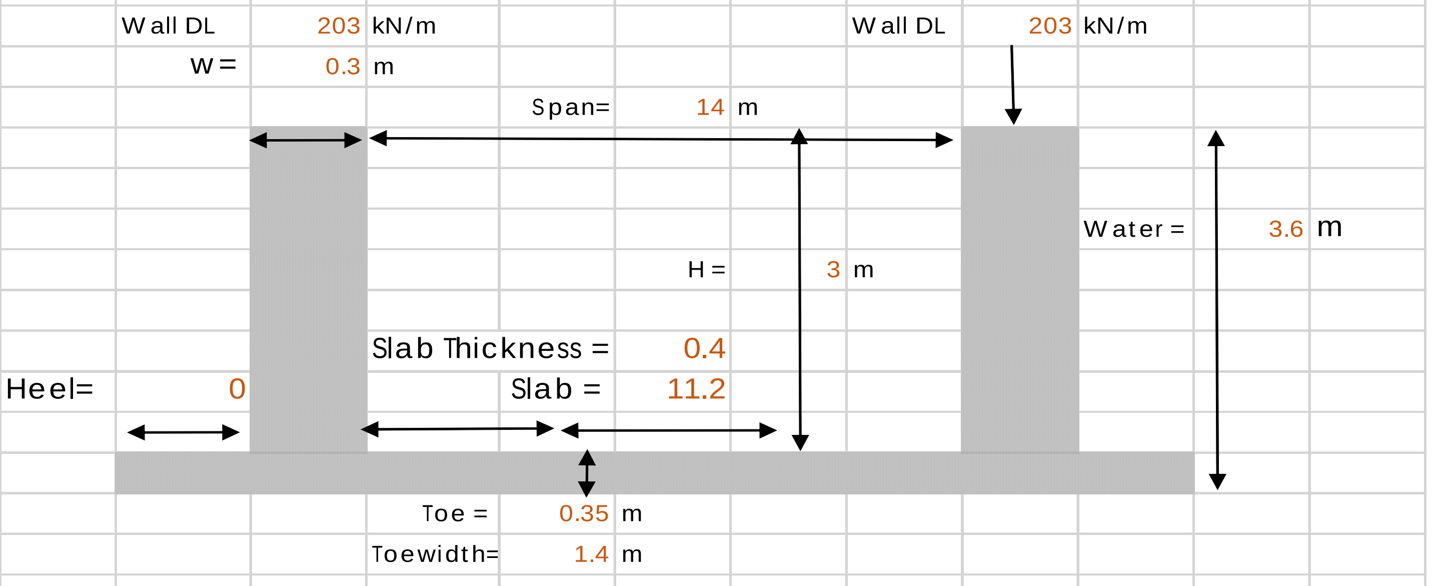


Uplift Calc

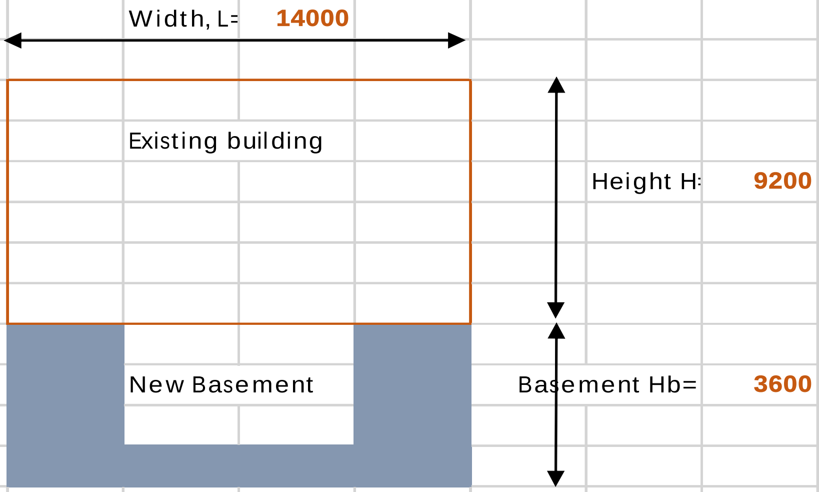
Total Dead Load =	Slab=	112	kN/m				
	Toe and heel =	29.75	kN/m				
	Wall =	45					
	Soil=(	0	+	0	) x 2=	0	-0.72
	Total Dead load =	592.75	kN/m				
Total Uplift Force=		525.6	kN/m	f.o.s.=	1.13	No Global Uplift	

Slab Uplift

Slab =	10	kN/m	Uplift =	36
Service Moment =	-637	kNm/m		
Factored Design moment=	-749.7	kNm/m		
Factored Design shear =	-214.2	kN/m		

Global Heave

Weight of building =	170	kN/m			
Weight of soil removed =	788.4				
% change	78%		place	78%	of Slab area as heave protection
Wide of Heave protection =	11.4519	m	place	11.45	m of Slab area as heave protection



L/H = 1.52174

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

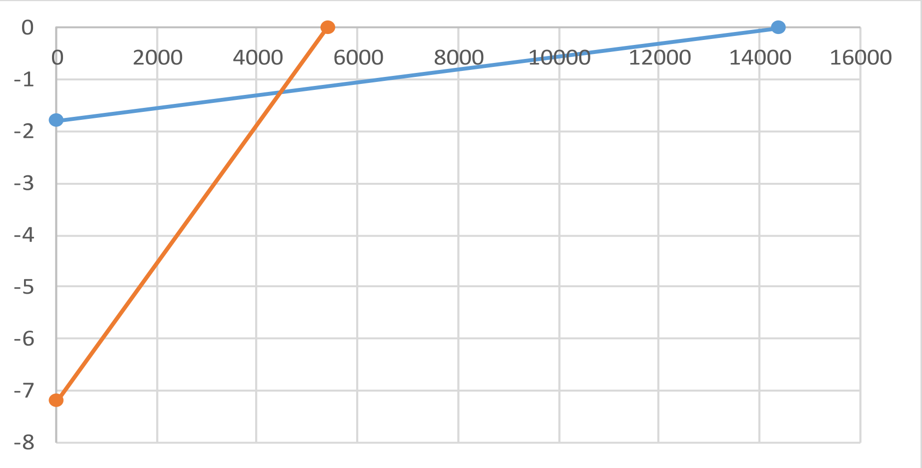
Potential Movement Due to wall installation

Horizontal surface movement =				0.05%				
DeltaH =	0.05%	x	3600	=	1.8	mm		
Vertical Surface Movement =				0.05%		1.8		
Delta V =	0.05%	x	3600	=	1.8	mm	=	0.33333 mm/m
Distance behind wall wall to neglibible movement								
lh =	3600	x	1.5	=	5400	mm		

Potential Movement Due to wall Excavation

Horizontal surface movement =				0.15%		5.4		
DeltaH =	0.15%	x	3600	=	5.4	mm		
							=	0.375 mm/m
Vertical Surface Movement =				0.10%				
Delta V =	0.10%	x	3600	=	3.6	mm		
Distance behind wall wall to neglibible movement								
lh =	3600	x	4	=	14400	mm		

		Excavation movement		Installation movement		
		Distance	delta V	Distance	delta V	
Nodes	x	14400	0	5400	0	
	y	0	-1.8	0	-7.2	



Determine Horizontal Movement

delta l =	7.2	mm	=	0.05%
	14400	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"**

	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.

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.

Burland Scale

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

Category of Damage	Approximate crack width	Limiting Tensile strain	Definitions of cracks and repair types/considerations
0	**Up to 0.1**	0.0-0.05	**HAIRLINE – Internally cracks can be filled or covered by wall covering, and redecorated. Externally, cracks rarely visible and remedial works rarely justified.**
1	0.2 to 2	0.05-0.075	FINE – 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.

The anticipated damage Category for the new basement is **0- 1**

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.	Loft conversions, 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. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical monitoring movement by standard optical equipment	Lowering of existing basement and cellars more than 2.5m 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	New basements greater than 2.5m and shallower than 4m Deep in gravels

	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. Vertical monitoring movement by standard optical equipment Lateral movement between walls by laser measurements	Basements up to 4.5m deep in clays Underpinning works to grade I listed building
	Monitoring 5 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. Vertical & Lateral monitoring movement by theodolite at specific times during the projects.	Underpinning works to Grade I listed buildings Basements to Listed building Basements deeper than 4m in Gravels Basements deeper than 4.5m in clays Underpinning, basements to buildings that are expressing defects.
	Monitoring 6 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. Vertical & Lateral monitoring movement by electronic means with live data gathering. Weekly interpretation	Double storey basements supported by piled retaining walls in gravels and soft sands. (N<12)

	Monitoring 7 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. Vertical & Lateral monitoring movement by electronic means with live data gathering with data transfer.	Larger Multi storey basements on particular projects.
Monitoring Conclusion	The level of Monitoring Croft recommend on 35 Greville Road is:	
	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. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical monitoring movement by standard optical equipment Lateral movement between walls by laser measurements	New basements greater than 2.5m and shallower than 4m Deep in gravels Basements up to 4.5m deep in clays Underpinning works to grade I listed building
	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
- Monitoring of movement
- 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

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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

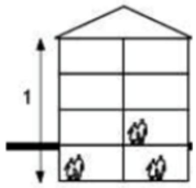
Family/domestic use

Loading Requirements (EC1-1)

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

<table>
<tr><td></td><td>The basement does not line within a 45° angle of the highway.
Therefore Highways HA loading is not required to be applied.</td></tr>
<tr><td>Part A3 Progressive collapse</td><td>Number of Storeys 4

Is the Building Multi Occupancy? No

<table><tr><td></td><td></td></tr><tr><td>Class 1</td><td>Single occupancy houses not exceeding 4 storeys</td></tr></table></td></tr>
<tr><td></td><td>To NHBC guidance compliance is only required to other floors if a material change of use occurs to the property.
<table><tr><td>Initial Building Class</td><td>1</td></tr><tr><td>Proposed Building Class</td><td>1</td></tr><tr><td>If class has changed material change has occurred</td><td>No</td></tr></table>


3 storey over basement</td></tr>
<tr><td>Lateral Stability</td><td></td></tr>
<tr><td>Exposure and wind loading conditions</td><td>Basic wind speed Vb = 21 m/s to EC1-2
Topography not considered significant.</td></tr>
<tr><td>Stability Design</td><td>The cantilevered walls are suitable to carry the lateral loading applied from above</td></tr>
<tr><td>Lateral Actions</td><td>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.</td></tr>
<tr><td>Retained soil Parameters</td><td>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.</td></tr>
</table>

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<table>
<tr><td>Water Table</td><td>Has a soil investigation been carried out Yes
<u>Known water table from boreholes</u>
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 <u>can</u> be ignored when water table lower than basement. BS8102 only indicates guidance.</td></tr>
<tr><td>Drainage and Damp Waterproofing</td><td>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.

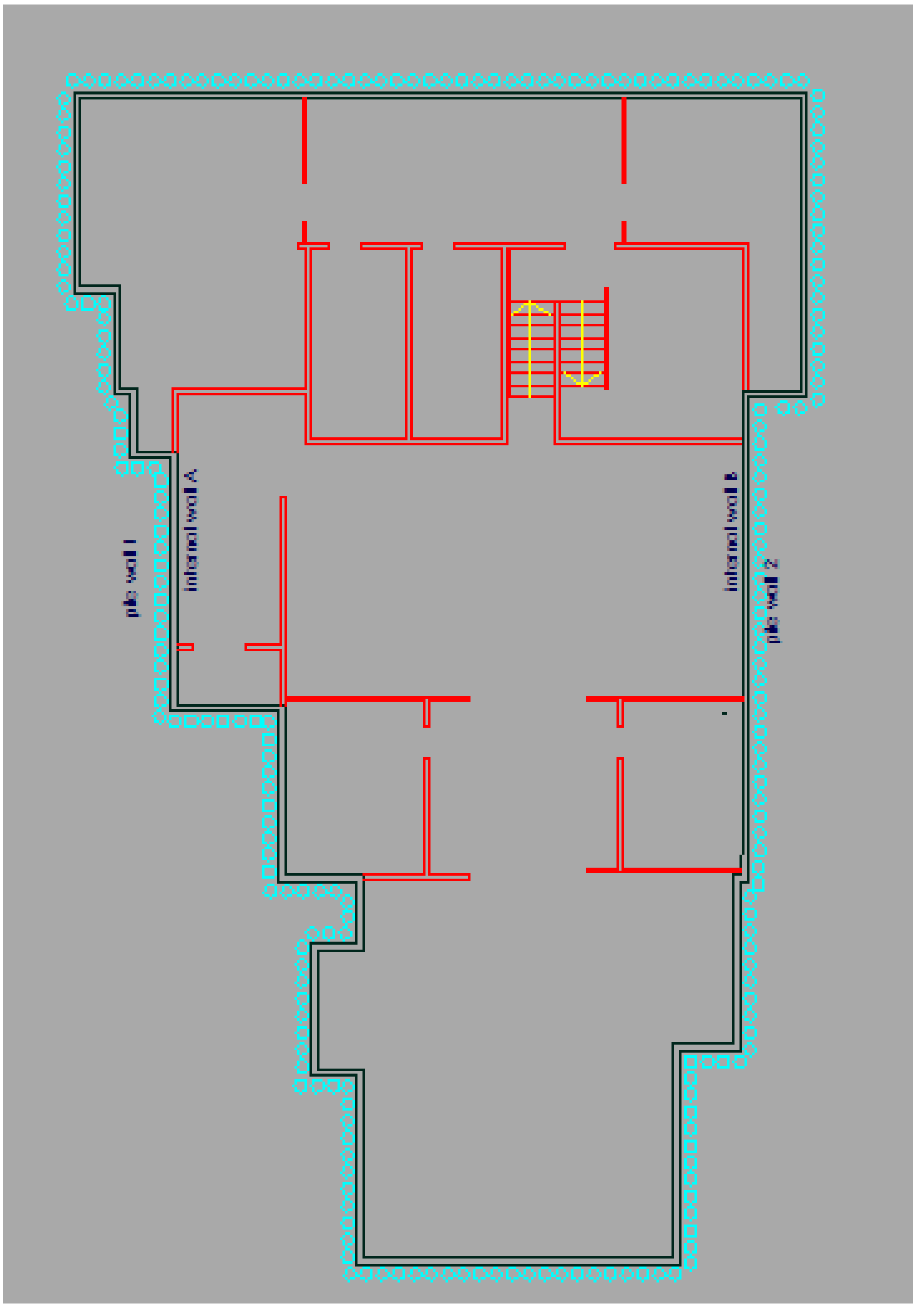
Croft are not the structural waterproofer. The waterproofing specialist must name who is their structural waterproofer. The Structural waterproofer must inspect the structural details and confirm that are happy with the robustness.

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</td></tr>
<tr><td>Localised Dewatering</td><td>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</td></tr>
</table>

	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.

Retaining Wall Calculation


pile wall 1
internal wall A
internal wall B
pile wall 2

Job Number: 150525 (35 Greville Road)
Date: 24 Aug 15

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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%

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PILED WALL 1 (TEMPORARY CASE)

Location	Area			Type	L	Load	Load kN			
	L	W	m2			kN/m2	Dead	%	Live	Total
pile wall 1										
roof DL	4.0	1.0	4.0	g_k		1.05	4.2			
roof LL				q_k		0.75			3.0	
loft DL	4.0	1.0	4.0	g_k		0.63	2.5			
loft DL				q_k		1.50			6.0	
2nd fl DL	4.0	1.0	4.0	g_k		4.62	18.5			
2nd fl LL				q_k		1.50			6.0	
2nd fl partitions	3.0	1.0	3.0	g_k		2.60	7.8			
1st fl DL	4.0	1.0	4.0	g_k		4.62	18.5			
1st fl LL				q_k		1.50			6.0	
1st fl partitions	3.0	1.0	3.0	g_k		2.60	7.8			
ground fl DL	4.0	1.0	4.0	g_k		4.62	18.5			
ground fl LL				q_k		1.50			6.0	
grd fl partitions	3.0	1.0	3.0	g_k		2.60	7.8			
walls	9.0	1.0	9.0	g_k		10.00	90.0			
							175.6	kN/m	27.0	kN/m

RETAINING WALL ANALYSIS & DESIGN (BS8002)

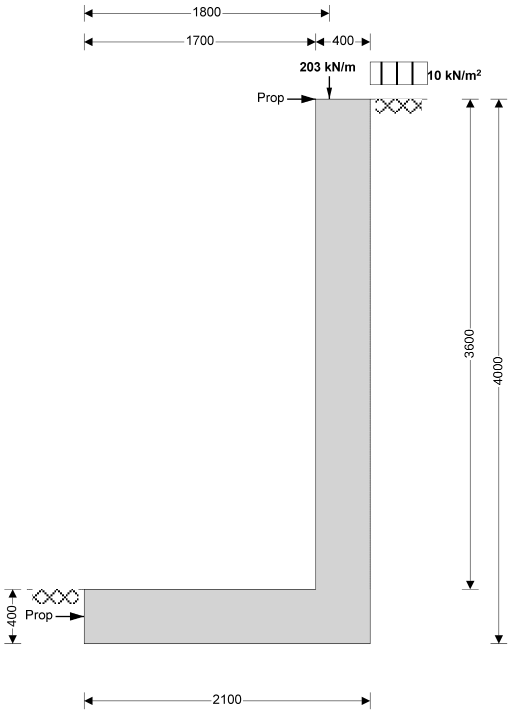
Loadings

Dead loadDL=176kN/m

Live loadLL=27kN/m

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3600** mm	Wall stem thickness	t_{wall} = **400** mm
Length of toe	l_{toe} = **1700** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **2100** mm	Base thickness	t_{base} = **400** mm
Height of retaining wall	h_{wall} = **4000** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **400** mm
Position of downstand	l_{ds} = **1500** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **0** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **4000** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg	Angle of wall friction	δ = **0.0** deg
Design shear strength	ϕ'_b = **24.2** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **18.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **130** kN/m^2

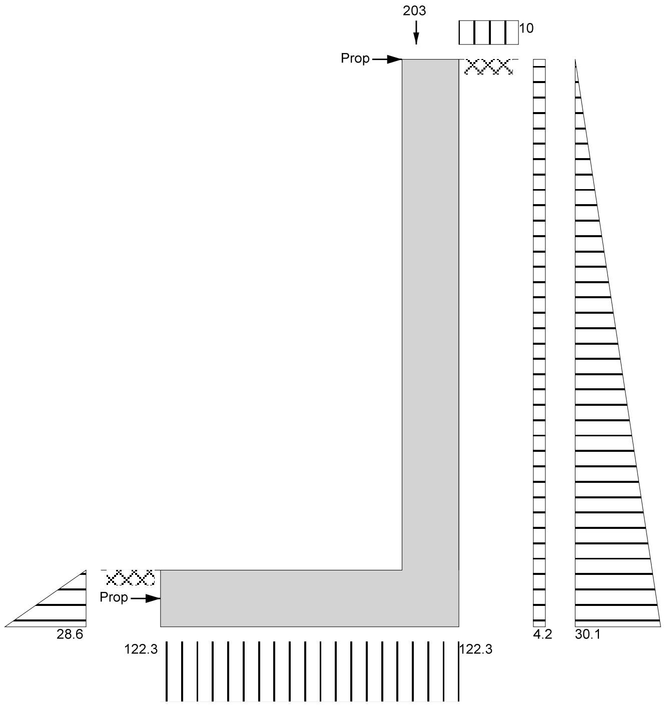
Using Coulomb theory

Active pressure	K_a =**0.419**	Passive pressure	K_p = **4.187**
At-rest pressure	K_0 = **0.590**		

Loading details

Surcharge load	Surcharge = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **176.0** kN/m	Vertical live load	W_{live} = **27.0** kN/m
Horizontal dead load	F_{dead} = **0.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m
Position of vertical load	l_{load} = **1800** mm	Height of horizontal load	h_{load} = **0** mm

CROFT STRUCTURAL ENGINEERS



Loads shown in kN/m, pressures shown in kN/m²

Calculate propping force

Propping force F_{prop} = **0.0** kN/m

Check bearing pressure

Total vertical reaction R = **256.8** kN/m — Distance to reaction x_{bar} = **1050** mm

Eccentricity of reaction e = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe p_{toe} = **122.3** kN/m² — Bearing pressure at heel p_{heel} = **122.3** kN/m²

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall F_{prop_top} = **-4.921** kN/m — Propping force to base of wall F_{prop_base} = **4.921** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4** Live load factor γ_{f_l} = **1.6**

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

Calculate propping force

Propping force F_{prop} = **0.0** kN/m

Calculate propping forces to top and base of wall

Propping force to top of wall $F_{prop\_top\_f}$ = **-8.313** kN/m Propping force to base of wall $F_{prop\_base\_f}$ = **48.764** kN/m

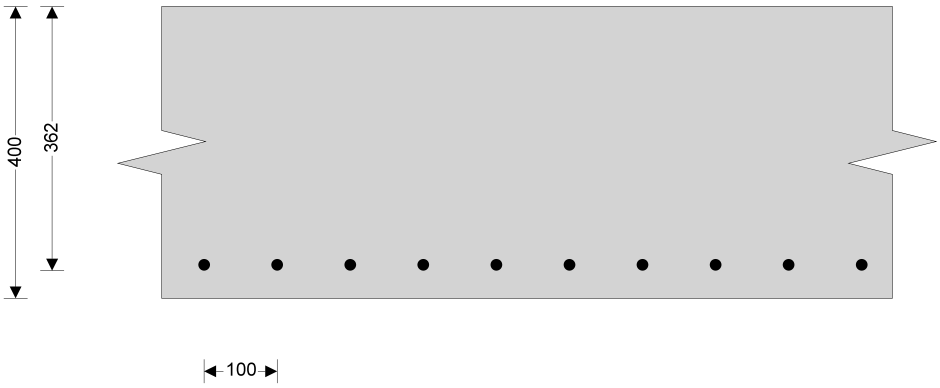
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **35** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** % Cover in toe c_{toe} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **273.0** kN/m Moment at heel M_{toe} = **289.8** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **16 mm dia.bars @ 100 mm centres**

Area required $A_{s\_toe\_req}$ = **1978.9** mm^2/m Area provided $A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.750** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.490** N/mm^2

v_{toe} > v_{c_toe} - Shear reinforcement required

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

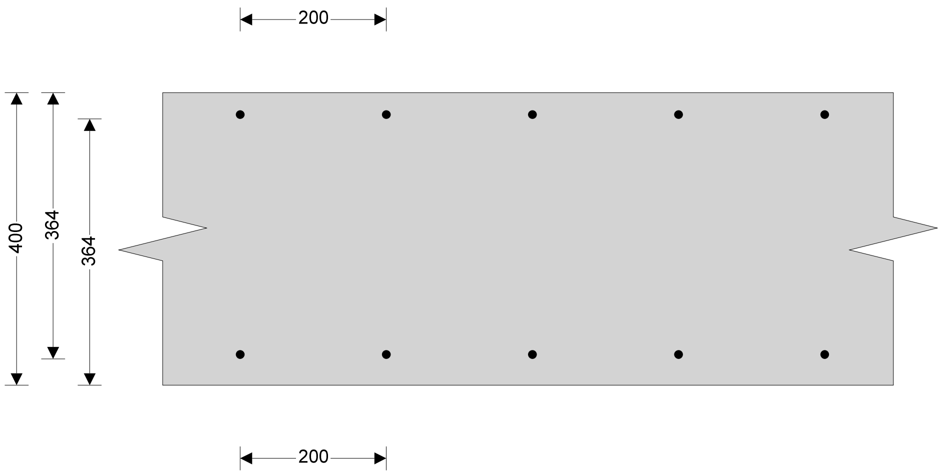
Material properties

Strength of concrete f_{cu} = **35** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

Cover in stem c_{stem} = **30** mm Cover in wall c_{wall} = **30** mm



Design of retaining wall stem

Shear at base of stem V_{stem} = **96.1** kN/m Moment at base of stem M_{stem} = **69.5** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **520.0** mm^2/m Area provided $A_{s\_stem\_prov}$ = **565** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.264** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.428** N/mm^2

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

Design of retaining wall at mid height

Moment at mid height M_{wall} = **31.4** kNm/m

Compression reinforcement is not required

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

Area required $A_{s\_wall\_req}$ = **520.0** mm^2/m Area provided $A_{s\_wall\_prov}$ = **565** mm^2/m

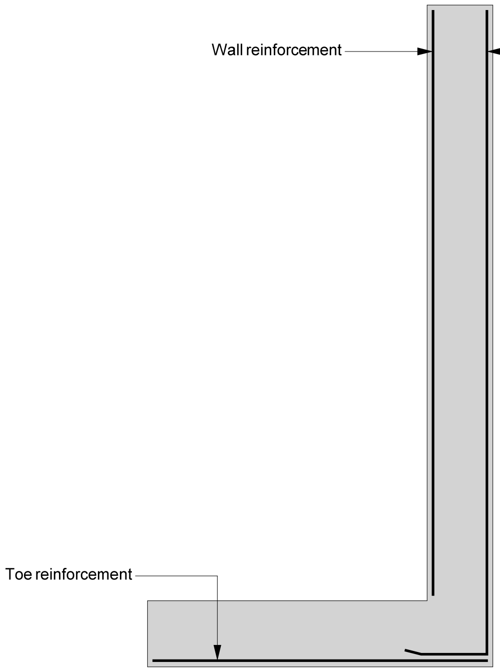
PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Max span/depth ratio $ratio_{max}$ = **30.94** Actual span/depth ratio $ratio_{act}$ = **9.89**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)

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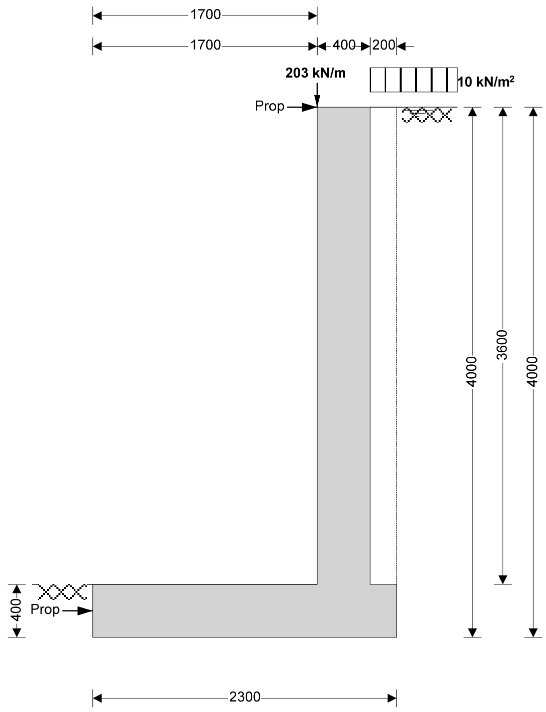
PILED WALL 1 (PERMANENT CASE)

RETAINING WALL ANALYSIS & DESIGN (BS8002)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details			
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3600** mm	Wall stem thickness	t_{wall} = **400** mm
Length of toe	l_{toe} = **1700** mm	Length of heel	l_{heel} = **200** mm
Overall length of base	l_{base} = **2300** mm	Base thickness	t_{base} = **400** mm
Height of retaining wall	h_{wall} = **4000** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **400** mm
Position of downstand	l_{ds} = **1200** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **4000** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **4000** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg	Angle of wall friction	δ = **0.0** deg
Design shear strength	ϕ'_b = **24.2** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **18.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **130** kN/m^2

Using Coulomb theory

Active pressure	K_a =**0.419**	Passive pressure	K_p = **4.187**
At-rest pressure	K_0 = **0.590**		

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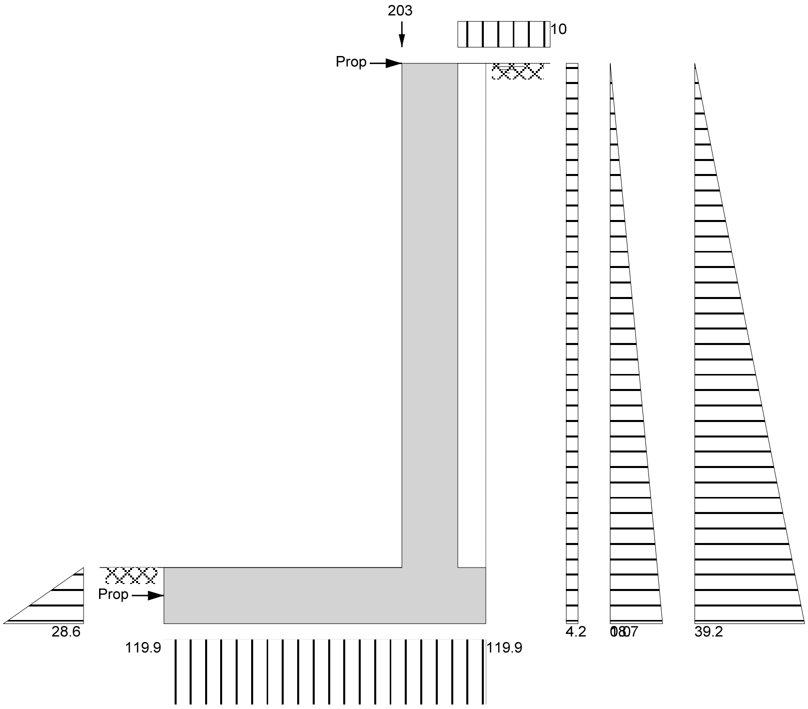
Loading details

Surcharge load	Surcharge = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **176.0** kN/m	Vertical live load	W_{live} = **27.0** kN/m
Horizontal dead load	F_{dead} = **0.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m
Position of vertical load	l_{load} = **1700** mm	Height of horizontal load	h_{load} = **0** mm



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

Calculate propping force

Propping force	F_{prop} = **43.9** kN/m		
Check bearing pressure			
Total vertical reaction	R = **275.8** kN/m	Distance to reaction	x_{bar} = **1150** mm
Eccentricity of reaction	e = **0** mm		

Reaction acts within middle third of base

Bearing pressure at toe	p_{toe} = **119.9** kN/m^2	Bearing pressure at heel	p_{heel} = **119.9** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall F_{prop_top} = **19.600** kN/m Propping force to base of wall F_{prop_base} = **24.310** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**
Live load factor γ_{f_l} = **1.6**
Earth pressure factor γ_{f_e} = **1.4**

Calculate propping force

Propping force F_{prop} = **43.9** kN/m
Calculate propping forces to top and base of wall
Propping force to top of wall $F_{prop\_top\_f}$ = **21.216** kN/m
Propping force to base of wall $F_{prop\_base\_f}$ = **76.087** kN/m

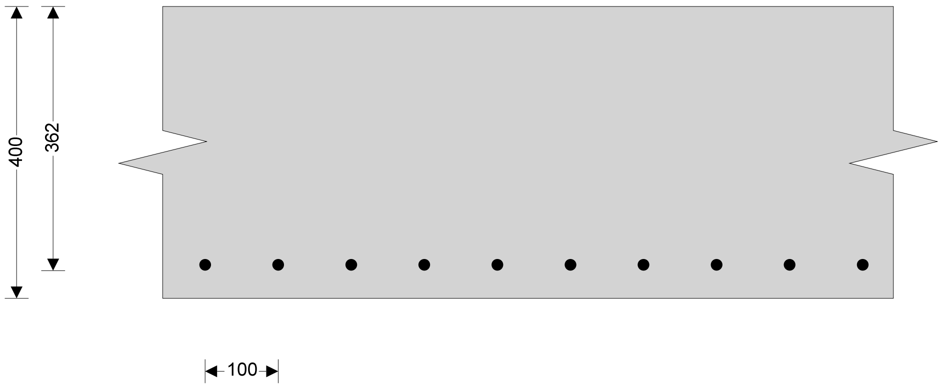
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **35** N/mm^2
Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** %
Cover in toe c_{toe} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **267.2** kN/m
Moment at heel M_{toe} = **283.7** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **16 mm dia.bars @ 100 mm centres**
Area required $A_{s\_toe\_req}$ = **1933.8** mm^2/m
Area provided $A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.734** N/mm^2
Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.428** N/mm^2

v_{toe} > v_{c_toe} - Shear reinforcement required

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

Material properties

Strength of concrete f_{cu} = **35** N/mm^2
Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** %
Cover in heel c_{heel} = **30** mm

As the moment is negative the design of the retaining wall heel is beyond the scope of this calculation

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

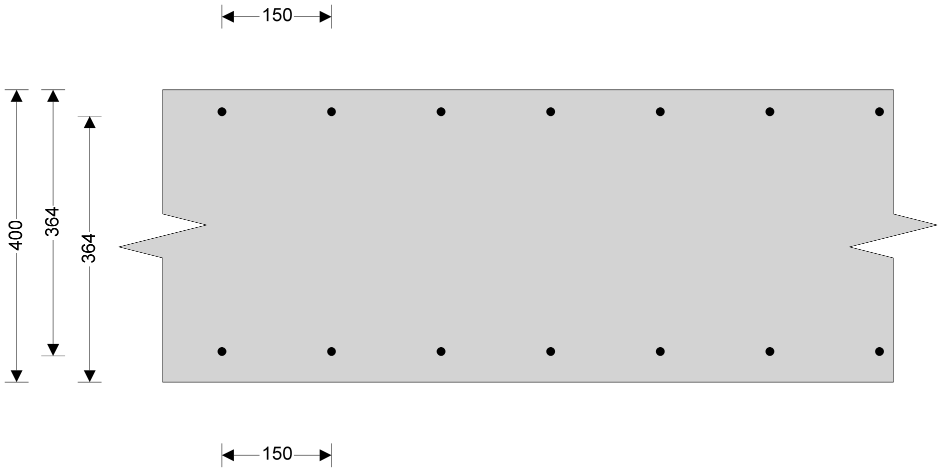
Material properties

Strength of concrete f_{cu} = **35** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

Cover in stem c_{stem} = **30** mm Cover in wall c_{wall} = **30** mm



Design of retaining wall stem

Shear at base of stem V_{stem} = **140.4** kN/m Moment at base of stem M_{stem} = **91.6** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **608.9** mm^2/m Area provided $A_{s\_stem\_prov}$ = **754** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.386** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.428** N/mm^2

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

Design of retaining wall at mid height

Moment at mid height M_{wall} = **42.8** kNm/m

Compression reinforcement is not required

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

Area required $A_{s\_wall\_req}$ = **520.0** mm^2/m Area provided $A_{s\_wall\_prov}$ = **754** mm^2/m

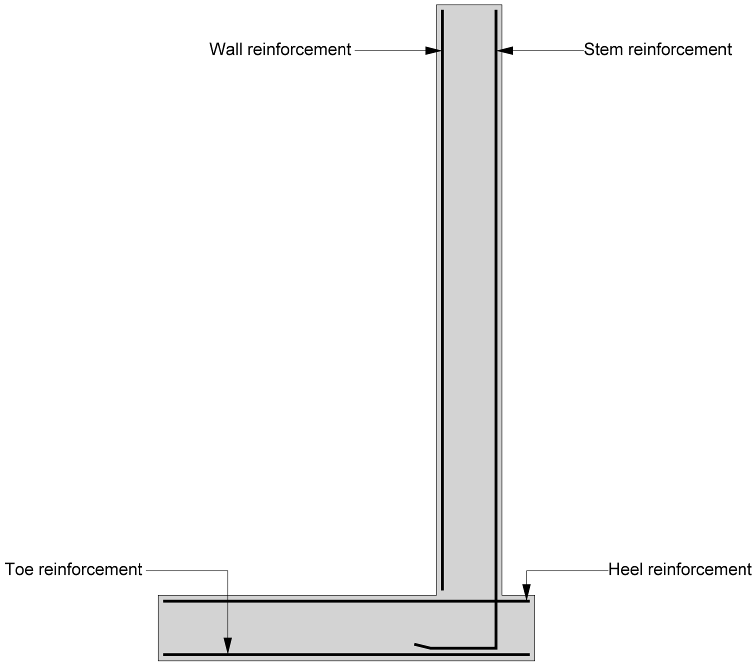
PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Max span/depth ratio $ratio_{max}$ = **32.77** Actual span/depth ratio $ratio_{act}$ = **9.89**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)

The design of the retaining wall heel is beyond the scope of this calculation!

Wall bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)

Stem bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)

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CAPPING BEAM FOR PILED WALL 1

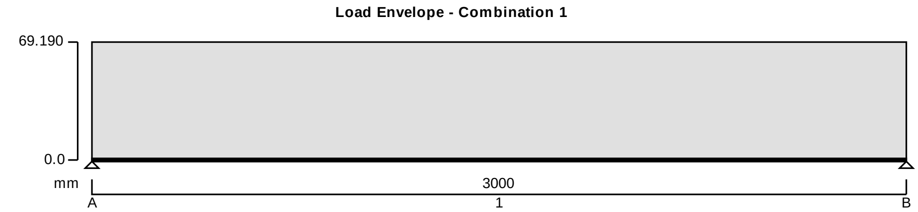
Propping force = 43.9kN/m

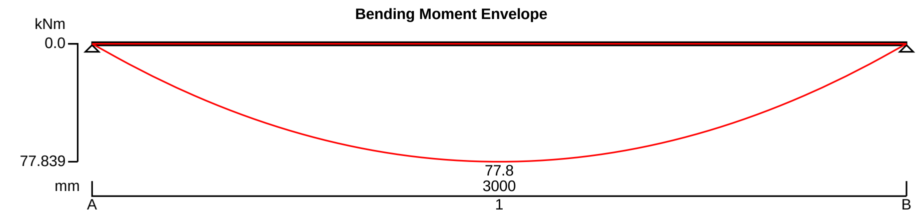
RC BEAM ANALYSIS & DESIGN (EN1992)

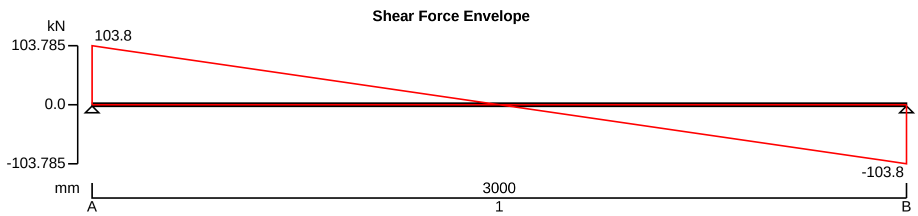
RC BEAM ANALYSIS & DESIGN (EN1992-1)

In accordance with UK national annex

TEDDS calculation version 2.1.15







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 45 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

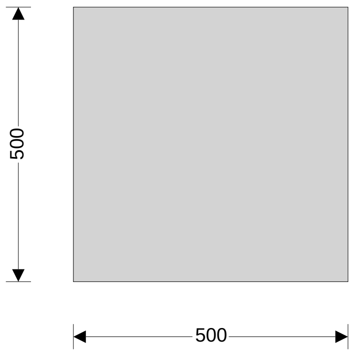
Variable × 1.50

Analysis results

Maximum moment support A	M_{A_max} = **0** kNm	M_{A_red} = **0** kNm
Maximum moment span 1 at 1500 mm	M_{s1_max} = **78** kNm	M_{s1_red} = **78** kNm
Maximum moment support B	M_{B_max} = **0** kNm	M_{B_red} = **0** kNm
Maximum shear support A	V_{A_max} = **104** kN	V_{A_red} = **104** kN
Maximum shear support A span 1 at 449 mm	$V_{A\_s1\_max}$ = **73** kN	$V_{A\_s1\_red}$ = **73** kN
Maximum shear support B	V_{B_max} = **-104** kN	V_{B_red} = **-104** kN
Maximum shear support B span 1 at 2551 mm	$V_{B\_s1\_max}$ = **-73** kN	$V_{B\_s1\_red}$ = **-73** kN
Maximum reaction at support A	R_A = **104** kN	
Maximum reaction at support B	R_B = **104** kN	

Rectangular section details

Section width	b = **500** mm	Section depth	h = **500** mm



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	Char.comp.cube strength	$f_{ck,cube}$ = **35** N/mm^2
Mean comp.cylinder strength	f_{cm} = **36** 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
Partial factor for concrete	γ_C = **1.50**	Comp.strength coefficient	α_{cc} = **0.85**
Design compressive strength	f_{cd} = **15.9** N/mm^2		

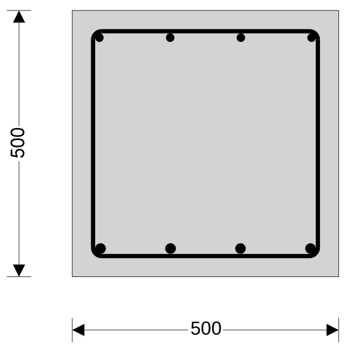
Reinforcement details

Characteristic yield strength	f_{yk} = **500** N/mm^2	Partial factor for reinforcment	γ_S = **1.15**
Design yield strength	f_{yd} = **435** N/mm^2		

Nominal cover to reinforcement

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

<u>Support A</u>



4 x 16ϕ bars

2 x 8ϕ shear legs at 200 c/c

4 x 20ϕ bars

Rectangular section in flexure (Section 6.1) -

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Design bending moment	M = **19** kNm	K = **0.007**	K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required	$A_{s,req}$ = **105** mm^2		
Tens.reinforcement provided	4 × 16ϕ bars	Tens.reinforcement provided	$A_{s,prov}$ = **804** mm^2
Min area of reinforcement	$A_{s,min}$ = **323** mm^2	Max area of reinforcement	$A_{s,max}$ = **10000** mm^2

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

Minimum bottom reinforcement at supports (cl.9.2.1.4(1))

Adj span reinforcement	$A_{s,span}$ = **1257** mm^2	Min btm reinforcement rqd	$A_{s2,min}$ = **314** mm^2
Btm reinforcement provided	4 × 20ϕ bars	Btm reinforcement provided	$A_{s2,prov}$ = **1257** mm^2

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

Rectangular section in shear (Section 6.2)

Des.shear force at support support $V_{Ed,max}$ = **104** kN Max.design shear force $V_{Rd,max}$ = **901** kN

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

Des.shear span 1 at 449 mm	V_{Ed} = **73** kN		
Shear reinforcement required	$A_{sv,req}$ = **157** mm^2/m	Min shear reinforcement	$A_{sv,min}$ = **423** mm^2/m
Shear reinforcement provided	2 × 8ϕ legs at 200 c/c	Shear reinforcement provided	$A_{sv,prov}$ = **503** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing $s_{vl,max}$ = **337** 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	Modulus of elasticity reinf	E_s = **200000** N/mm^2
Mean conc. tensile strength	$f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2	Stress distribution coefficient	k_c = **0.4**
Self-equilibrating stress coef	k = **0.86**	Actual tension bar spacing	s_{bar} = **133** mm
Max stress permitted (T.7.3N)	σ_s = **294** N/mm^2	Conc/steel mod of elast. ratio	α_{cr} = **6.19**
Distance of the ENA	y = **247** mm	Area of conc in tensile zone	A_{ct} = **123366** mm^2
Min area of reinf reqd (exp.7.1)	$A_{sc,min}$ = **399** mm^2		

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

Quasi-perm value	ψ_2 = **0.30**	Quasi-perm limit state moment	M_{QP} = **0** kNm
Permanent load ratio	R_{PL} = **0.00**	Service stress in reinf	σ_{sr} = **0** N/mm^2
Max bar spacing (Table 7.3N)	$s_{bar,max}$ = **300** mm		

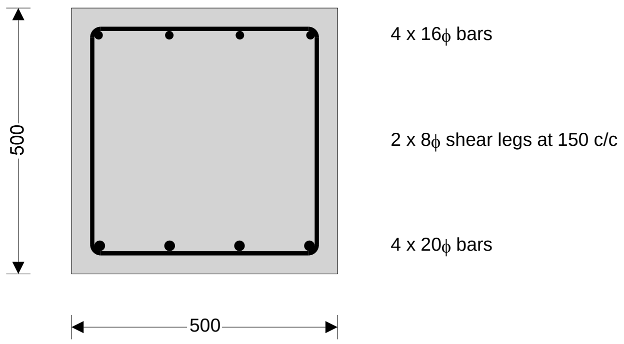
PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing	$s_{bot,min}$ = **131** mm	Min allowable bottom spacing	$s_{bar\_bot,min}$ = **45** mm
Minimum top bar spacing	$s_{top,min}$ = **133** mm	Min allowable top spacing	$s_{bar\_bot,min}$ = **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 = **78** kNm K = **0.028** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{s,req}$ = **422** mm^2

Tens.reinforcement provided 4 × 20ϕ bars Tens.reinforcement provided $A_{s,prov}$ = **1257** mm^2

Min area of reinforcement $A_{s,min}$ = **321** mm^2 Max area of reinforcement $A_{s,max}$ = **10000** 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 150 c/c Shear reinforcement provided $A_{sv,prov}$ = **670** mm^2/m

Min shear reinforcement $A_{sv,min}$ = **423** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing $s_{vl,max}$ = **335** 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 Modulus of elasticity reinf E_s = **200000** N/mm^2

Mean conc. tensile strength $f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2 Stress distribution coefficient k_c = **0.4**

Self-equilibrating stress coef k = **0.86** Actual tension bar spacing s_{bar} = **131** mm

Max stress permitted (T.7.3N) σ_s = **295** N/mm^2 Conc/steel mod of elast. ratio α_{cr} = **6.19**

Distance of the ENA y = **245** mm Area of conc in tensile zone A_{ct} = **122496** mm^2

Min area of reinf reqd (exp.7.1) $A_{sc,min}$ = **395** mm^2

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

Quasi-perm value ψ_2 = **0.30** Quasi-perm limit state moment M_{QP} = **58** kNm

Permanent load ratio R_{PL} = **0.74** Service stress in reinf σ_{sr} = **108** N/mm^2

Max bar spacing (Table 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 $s_{bot,min}$ = **131** mm Min allowable bottom spacing $s_{bar\_bot,min}$ = **45** mm

Minimum top bar spacing $s_{top,min}$ = **133** mm Min allowable top spacing $s_{bar\_bot,min}$ = **45** mm

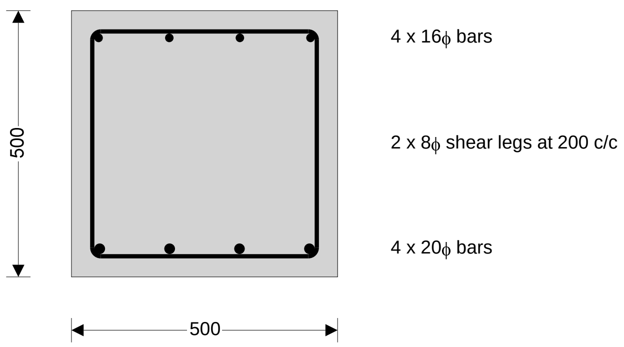
PASS - Actual bar spacing exceeds minimum allowable

Deflection control (Section 7.4)

Allowable span to depth ratio $span\_to\_depth_{allow}$ = **40.0** Actual span to depth ratio $span\_to\_depth_{actual}$ = **6.7**

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

<u>Support B</u>



Rectangular section in flexure (Section 6.1) -

Design bending moment M = **19** kNm K = **0.007** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{s,req}$ = **105** mm^2

Tens.reinforcement provided 4 × 16ϕ bars Tens.reinforcement provided $A_{s,prov}$ = **804** mm^2

Min area of reinforcement $A_{s,min}$ = **323** mm^2 Max area of reinforcement $A_{s,max}$ = **10000** mm^2

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

Minimum bottom reinforcement at supports (cl.9.2.1.4(1))

Adj span reinforcement	$A_{s,span}$ = **1257** mm^2	Min btm reinforcement rqd	$A_{s2,min}$ = **314** mm^2
Btm reinforcement provided	4 × 20ϕ bars	Btm reinforcement provided	$A_{s2,prov}$ = **1257** mm^2

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

Rectangular section in shear (Section 6.2)

Des.shear force at support support $V_{Ed,max}$ = **104** kN Max.design shear force $V_{Rd,max}$ = **901** kN

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

Des.shear span 1 at 2551 mm	V_{Ed} = **73** kN		
Shear reinforcement required	$A_{sv,req}$ = **157** mm^2/m	Min shear reinforcement	$A_{sv,min}$ = **423** mm^2/m
Shear reinforcement provided	2 × 8ϕ legs at 200 c/c	Shear reinforcement provided	$A_{sv,prov}$ = **503** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing $s_{vl,max}$ = **337** 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	Modulus of elasticity reinf	E_s = **200000** N/mm^2
Mean conc. tensile strength	$f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2	Stress distribution coefficient	k_c = **0.4**
Self-equilibrating stress coef	k = **0.86**	Actual tension bar spacing	s_{bar} = **133** mm
Max stress permitted (T.7.3N)	σ_s = **294** N/mm^2	Conc/steel mod of elast. ratio	α_{cr} = **6.19**
Distance of the ENA	y = **247** mm	Area of conc in tensile zone	A_{ct} = **123366** mm^2
Min area of reinf reqd (exp.7.1)	$A_{sc,min}$ = **399** mm^2		

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

Quasi-perm value	ψ_2 = **0.30**	Quasi-perm limit state moment	M_{QP} = **0** kNm
Permanent load ratio	R_{PL} = **0.00**	Service stress in reinf	σ_{sr} = **0** N/mm^2
Max bar spacing (Table 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	$s_{bot,min}$ = **131** mm	Min allowable bottom spacing	$s_{bar\_bot,min}$ = **45** mm
Minimum top bar spacing	$s_{top,min}$ = **133** mm	Min allowable top spacing	$s_{bar\_bot,min}$ = **45** mm

PASS - Actual bar spacing exceeds minimum allowable

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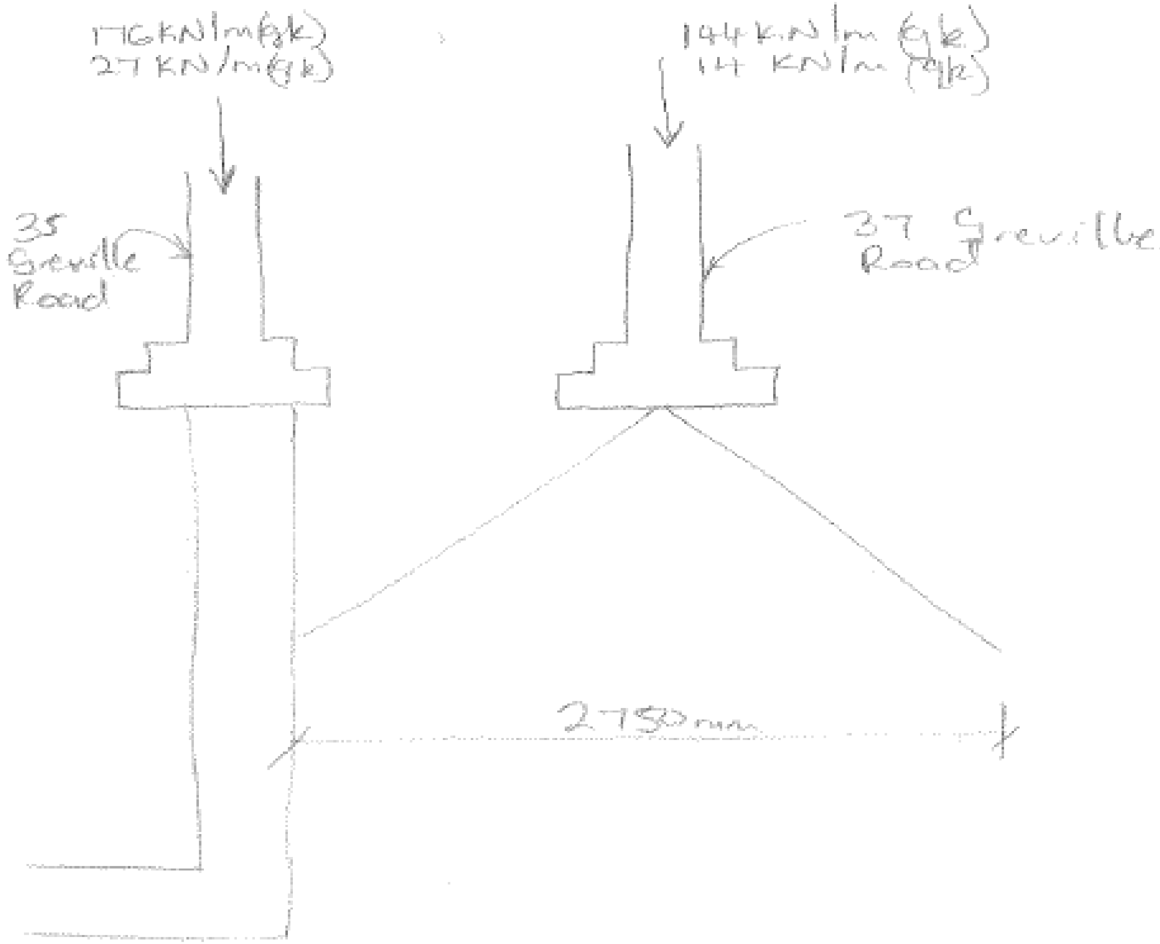
PILED WALL 2 (TEMPORARY CASE)

Location	Area L	Area W	Area m2	Type	L	Load kN/m2	Load kN Dead	%	Live	Total
pile wall 2										
roof DL	4.0	1.0	4.0	g_k		1.05	4.2			
roof LL				q_k		0.75			3.0	
loft DL	4.0	1.0	4.0	g_k		0.63	2.5			
loft DL				q_k		1.50			6.0	
2nd fl DL	4.0	1.0	4.0	g_k		4.62	18.5			
2nd fl LL				q_k		1.50			6.0	
2nd fl partitions	3.0	1.0	3.0	g_k		2.60	7.8			
1st fl DL	4.0	1.0	4.0	g_k		4.62	18.5			
1st fl LL				q_k		1.50			6.0	
1st fl partitions	3.0	1.0	3.0	g_k		2.60	7.8			
ground fl DL	4.0	1.0	4.0	g_k		4.62	18.5			
ground fl LL				q_k		1.50			6.0	
grd fl partitions	3.0	1.0	3.0	g_k		2.60	7.8			
walls	9.0	1.0	9.0	g_k		10.00	90.0			
							175.6	kN/m	27.0	kN/m
37 greville road										
roof DL	2.0	1.0	2.0	g_k		1.05	2.1			
roof LL				q_k		0.75			1.5	
loft DL	2.0	1.0	2.0	g_k		0.63	1.3			
loft DL				q_k		1.50			3.0	
2nd fl DL	2.0	1.0	2.0	g_k		4.62	9.2			
2nd fl LL				q_k		1.50			3.0	
2nd fl partitions	3.0	1.0	3.0	g_k		2.60	7.8			
1st fl DL	2.0	1.0	2.0	g_k		4.62	9.2			
1st fl LL				q_k		1.50			3.0	
1st fl partitions	3.0	1.0	3.0	g_k		2.60	7.8			
ground fl DL	2.0	1.0	2.0	g_k		4.62	9.2			
ground fl LL				q_k		1.50			3.0	
grd fl partitions	3.0	1.0	3.0	g_k		2.60	7.8			
walls	9.0	1.0	9.0	g_k		10.00	90.0			
							144.5	kN/m	13.5	kN/m



<u>37 Greville Road</u>

Total service load = 144 kN/m + 14 kN/m

= 158 kN/m

Surcharge on 35 Greville Road

$$= \frac{158 \text{ kN/m}}{2.75 \text{ m}}$$

$$= 57 \text{ kN/m}^2$$

RETAINING WALL ANALYSIS & DESIGN (BS8002)

Loadings

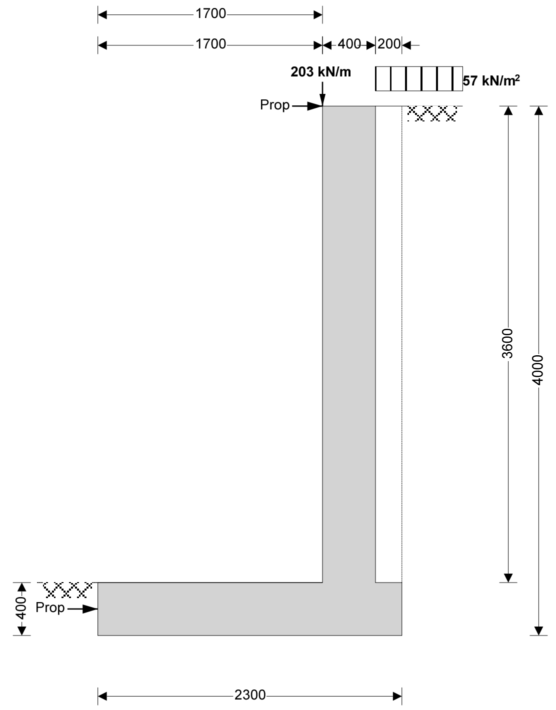
Dead loadDL=176kN/m

Live loadLL=27kN/m

Surcharge from 37 Greville Road

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3600** mm	Wall stem thickness	t_{wall} = **400** mm
Length of toe	l_{toe} = **1700** mm	Length of heel	l_{heel} = **200** mm
Overall length of base	l_{base} = **2300** mm	Base thickness	t_{base} = **400** mm
Height of retaining wall	h_{wall} = **4000** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **400** mm
Position of downstand	l_{ds} = **1250** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **0** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **4000** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg	Angle of wall friction	δ = **0.0** deg
Design shear strength	ϕ'_b = **24.2** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **18.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **130** kN/m^2

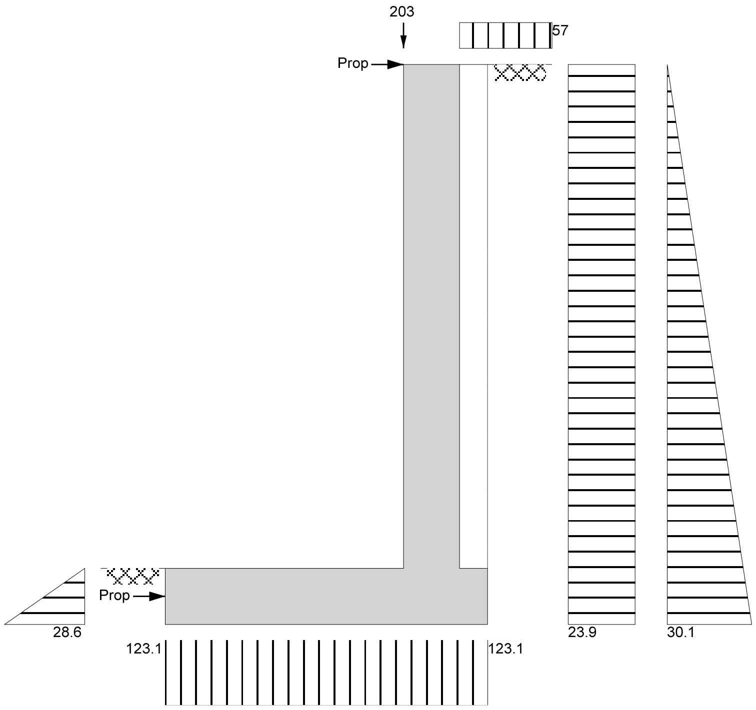
Using Coulomb theory

Active pressure	K_a =**0.419**	Passive pressure	K_p = **4.187**
At-rest pressure	K_0 = **0.590**		

Loading details

Surcharge load	Surcharge = **57.0** kN/m^2		
Vertical dead load	W_{dead} = **176.0** kN/m	Vertical live load	W_{live} = **27.0** kN/m
Horizontal dead load	F_{dead} = **0.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m

Position of vertical load l_{load} = **1700** mm | Height of horizontal load h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m²

Calculate propping force

Propping force F_{prop} = **67.6** kN/m

Check bearing pressure

Total vertical reaction R = **283.1** kN/m | Distance to reaction x_{bar} = **1150** mm

Eccentricity of reaction e = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe p_{toe} = **123.1** kN/m^2 | Bearing pressure at heel p_{heel} = **123.1** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall F_{prop_top} = **43.667** kN/m | Propping force to base of wall F_{prop_base} = **23.972** kN/m

CROFT STRUCTURAL ENGINEERS

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4** Live load factor γ_{f_l} = **1.6**

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

Calculate propping force

Propping force F_{prop} = **67.6** kN/m

Calculate propping forces to top and base of wall

Propping force to top of wall $F_{prop\_top\_f}$ = **82.573** kN/m Propping force to base of wall $F_{prop\_base\_f}$ = **128.378** kN/m

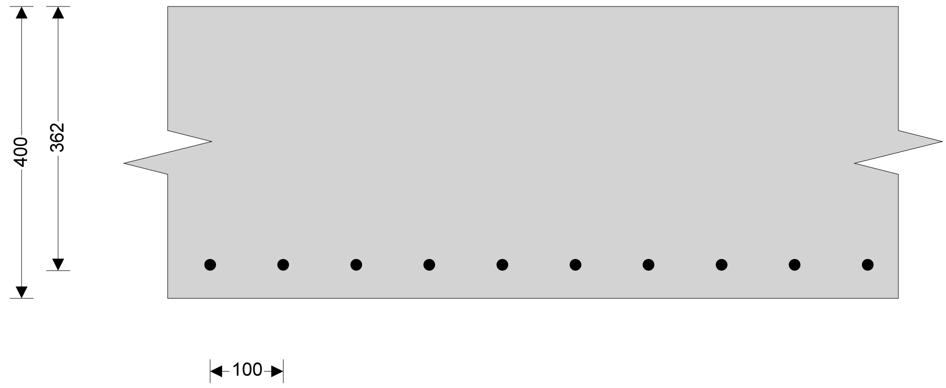
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **35** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** % Cover in toe c_{toe} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **276.1** kN/m Moment at heel M_{toe} = **293.2** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **16 mm dia.bars @ 100 mm centres**

Area required $A_{s\_toe\_req}$ = **2003.8** mm^2/m Area provided $A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.759** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.490** N/mm^2

v_{toe} > v_{c_toe} - Shear reinforcement required

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

Material properties

Strength of concrete f_{cu} = **35** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** % Cover in heel c_{heel} = **30** mm

As the moment is negative the design of the retaining wall heel is beyond the scope of this calculation

CROFT STRUCTURAL ENGINEERS

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

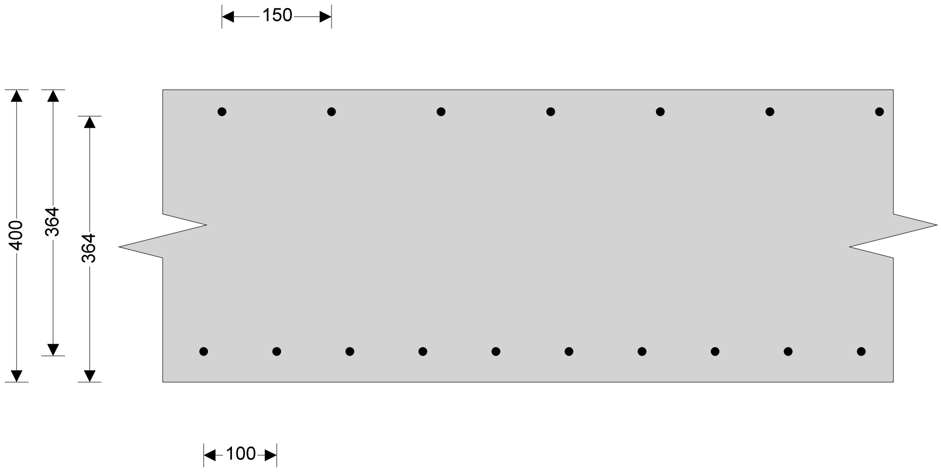
Material properties

Strength of concrete f_{cu} = **35** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

Cover in stem c_{stem} = **30** mm Cover in wall c_{wall} = **30** mm



Design of retaining wall stem

Shear at base of stem V_{stem} = **196.0** kN/m Moment at base of stem M_{stem} = **145.4** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **966.5** mm^2/m Area provided $A_{s\_stem\_prov}$ = **1131** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.538** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.490** N/mm^2

v_{stem} > v_{c_stem} - Shear reinforcement required

Design of retaining wall at mid height

Moment at mid height M_{wall} = **74.1** kNm/m

Compression reinforcement is not required

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

Area required $A_{s\_wall\_req}$ = **520.0** mm^2/m Area provided $A_{s\_wall\_prov}$ = **754** mm^2/m

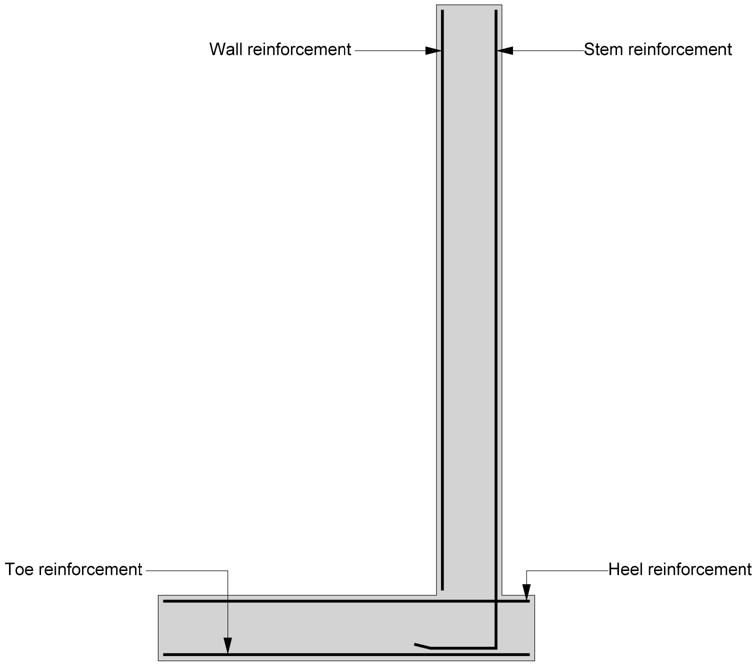
PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Max span/depth ratio $ratio_{max}$ = **27.03** Actual span/depth ratio $ratio_{act}$ = **9.89**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)

The design of the retaining wall heel is beyond the scope of this calculation!

Wall bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)

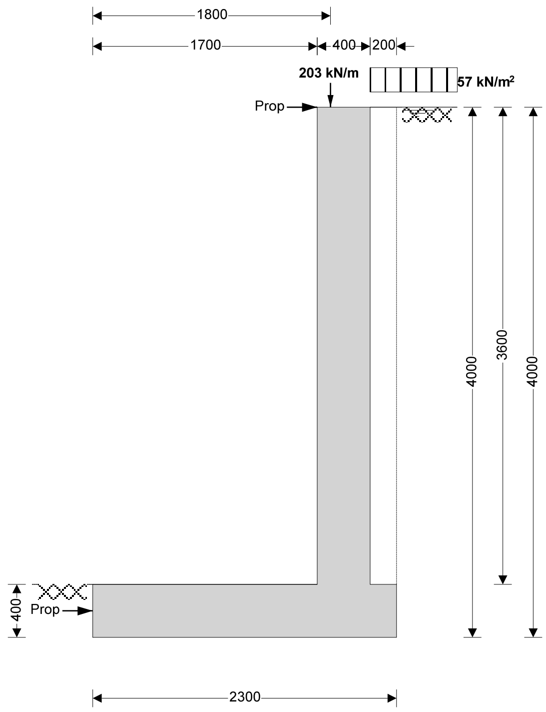
Stem bars - 12 mm dia.@ 100 mm centres - (1131 mm^2/m)

PILED WALL 2 (PERMANENT CASE)

RETAINING WALL ANALYSIS & DESIGN (BS8002)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details			
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3600** mm	Wall stem thickness	t_{wall} = **400** mm
Length of toe	l_{toe} = **1700** mm	Length of heel	l_{heel} = **200** mm
Overall length of base	l_{base} = **2300** mm	Base thickness	t_{base} = **400** mm
Height of retaining wall	h_{wall} = **4000** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **400** mm
Position of downstand	l_{ds} = **1200** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **4000** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **4000** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg	Angle of wall friction	δ = **0.0** deg
Design shear strength	ϕ'_b = **24.2** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **18.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **130** kN/m^2
Using Coulomb theory			
Active pressure	K_a =**0.419**	Passive pressure	K_p = **4.187**
At-rest pressure	K_0 = **0.590**		

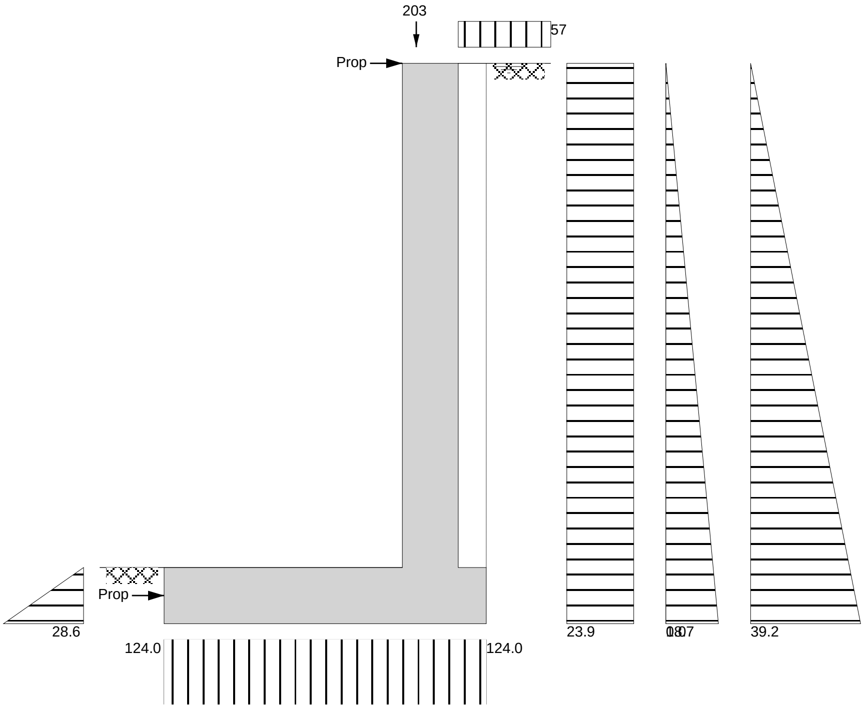
Loading details

Surcharge load | Surcharge = **57.0** kN/m^2

Vertical dead load | W_{dead} = **176.0** kN/m | Vertical live load | W_{live} = **27.0** kN/m

Horizontal dead load | F_{dead} = **0.0** kN/m | Horizontal live load | F_{live} = **0.0** kN/m

Position of vertical load | l_{load} = **1800** mm | Height of horizontal load | h_{load} = **0** mm



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

Calculate propping force

Propping force | F_{prop} = **122.6** kN/m

Check bearing pressure

Total vertical reaction | R = **285.2** kN/m | Distance to reaction | x_{bar} = **1150** mm

Eccentricity of reaction | e = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe | p_{toe} = **124.0** kN/m^2 | Bearing pressure at heel | p_{heel} = **124.0** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall | F_{prop_top} = **55.083** kN/m | Propping force to base of wall | F_{prop_base} = **67.508** kN/m

CROFT STRUCTURAL ENGINEERS

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4** Live load factor γ_{f_l} = **1.6**

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

Calculate propping force

Propping force F_{prop} = **122.6** kN/m

Calculate propping forces to top and base of wall

Propping force to top of wall $F_{prop\_top\_f}$ = **93.516** kN/m Propping force to base of wall $F_{prop\_base\_f}$ = **181.283** kN/m

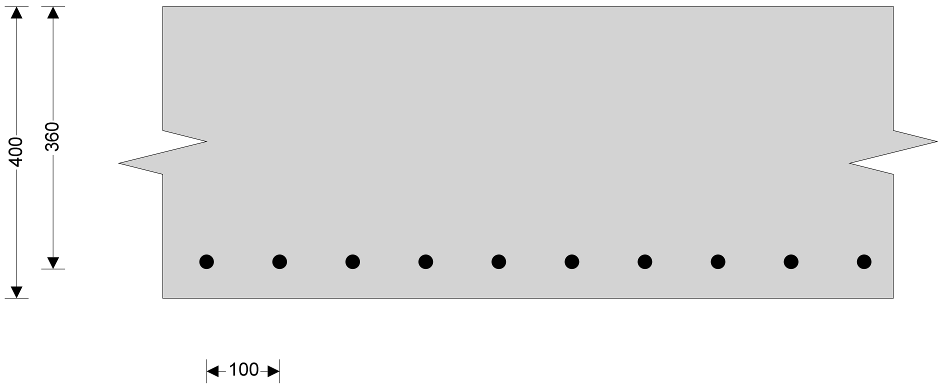
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **35** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** % Cover in toe c_{toe} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **278.3** kN/m Moment at heel M_{toe} = **295.5** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **20 mm dia.bars @ 100 mm centres**

Area required $A_{s\_toe\_req}$ = **2034.7** mm^2/m Area provided $A_{s\_toe\_prov}$ = **3142** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.769** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.521** N/mm^2

v_{toe} > v_{c_toe} - Shear reinforcement required

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

Material properties

Strength of concrete f_{cu} = **35** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** % Cover in heel c_{heel} = **30** mm

As the moment is negative the design of the retaining wall heel is beyond the scope of this calculation

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

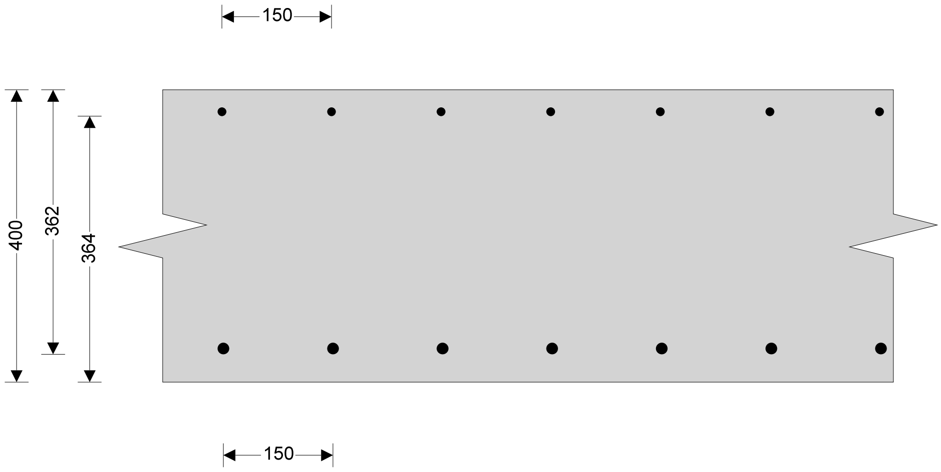
Material properties

Strength of concrete f_{cu} = **35** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

Cover in stem c_{stem} = **30** mm Cover in wall c_{wall} = **30** mm



Design of retaining wall stem

Shear at base of stem V_{stem} = **240.2** kN/m Moment at base of stem M_{stem} = **167.5** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **1119.4** mm^2/m Area provided $A_{s\_stem\_prov}$ = **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.664** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.521** N/mm^2

v_{stem} > v_{c_stem} - Shear reinforcement required

Design of retaining wall at mid height

Moment at mid height M_{wall} = **85.5** kNm/m

Compression reinforcement is not required

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

Area required $A_{s\_wall\_req}$ = **568.4** mm^2/m Area provided $A_{s\_wall\_prov}$ = **754** mm^2/m

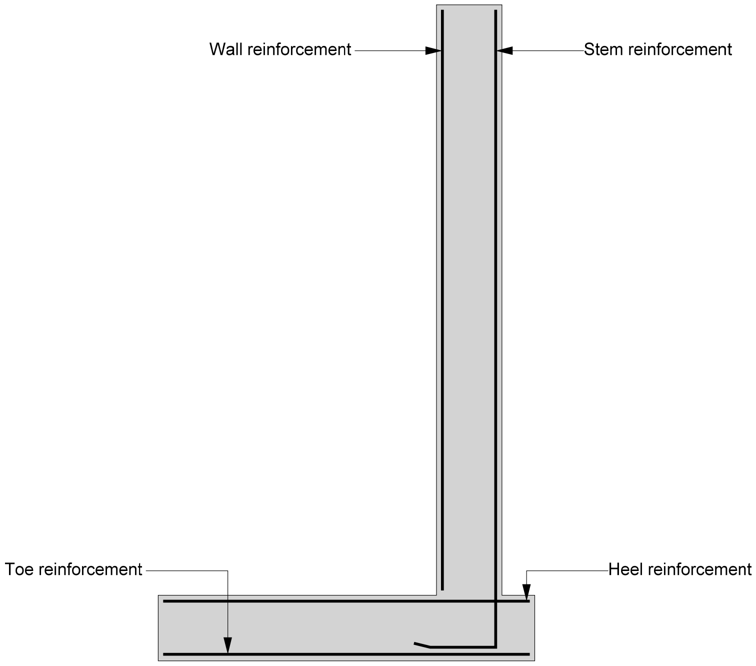
PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Max span/depth ratio $ratio_{max}$ = **26.20** Actual span/depth ratio $ratio_{act}$ = **9.94**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 20 mm dia.@ 100 mm centres - (3142 mm^2/m)

The design of the retaining wall heel is beyond the scope of this calculation!

Wall bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)

Stem bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)

CAPPING BEAM FOR PILED WALL 2

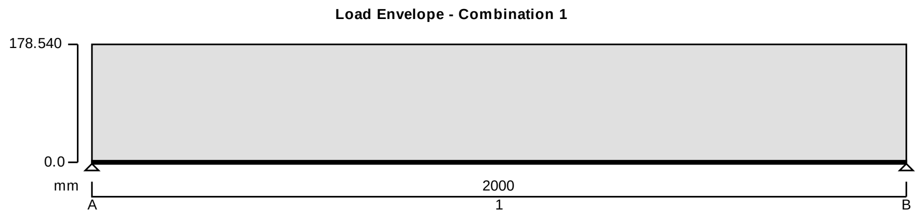
RC BEAM ANALYSIS & DESIGN (EN1992)

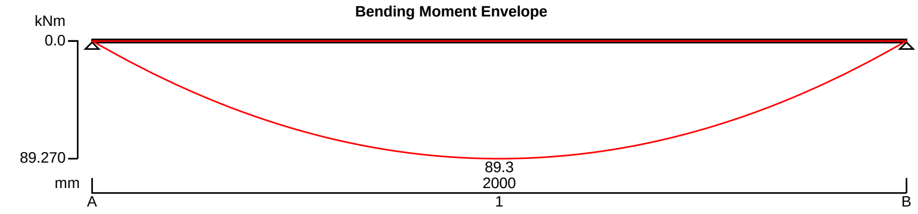
Prop forceDL=122.6kN/m

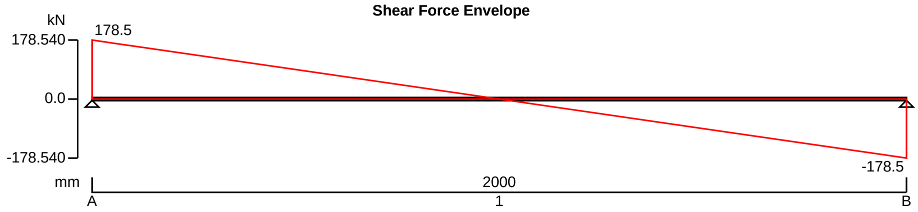
RC BEAM ANALYSIS & DESIGN (EN1992-1)

In accordance with UK national annex

TEDDS calculation version 2.1.15





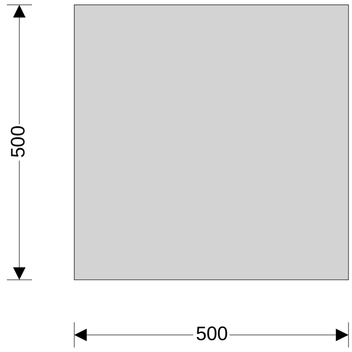


Support conditions		
Support A	Vertically restrained	
	Rotationally free	
Support B	Vertically restrained	
	Rotationally free	
Applied loading		
	Permanent self weight of beam × 1	
	Permanent full UDL 126 kN/m	
Load combinations		
Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 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} = **89** kNm	M_{s1_red} = **89** kNm
Maximum moment support B	M_{B_max} = **0** kNm	M_{B_red} = **0** kNm
Maximum shear support A	V_{A_max} = **179** kN	V_{A_red} = **179** kN
Maximum shear support A span 1 at 449 mm	$V_{A\_s1\_max}$ = **98** kN	$V_{A\_s1\_red}$ = **98** kN
Maximum shear support B	V_{B_max} = **-179** kN	V_{B_red} = **-179** kN
Maximum shear support B span 1 at 1551 mm	$V_{B\_s1\_max}$ = **-98** kN	$V_{B\_s1\_red}$ = **-98** kN
Maximum reaction at support A	R_A = **179** kN	
Maximum reaction at support B	R_B = **179** kN	

Rectangular section details

Section width	b = **500** mm	Section depth	h = **500** mm



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	Char.comp.cube strength	$f_{ck,cube}$ = **35** N/mm^2
Mean comp.cylinder strength	f_{cm} = **36** 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
Partial factor for concrete	γ_C = **1.50**	Comp.strength coefficient	α_{cc} = **0.85**
Design compressive strength	f_{cd} = **15.9** N/mm^2		

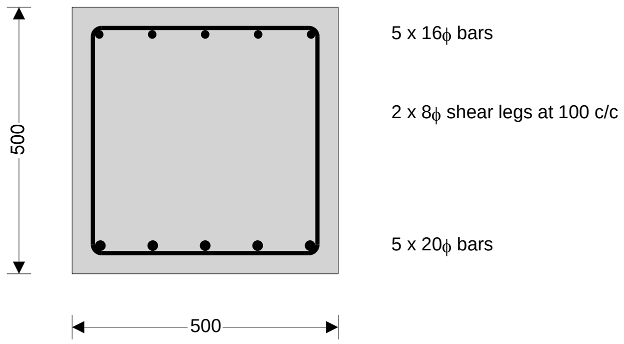
Reinforcement details

Characteristic yield strength	f_{yk} = **500** N/mm^2	Partial factor for reinforcment	γ_S = **1.15**
Design yield strength	f_{yd} = **435** N/mm^2		

Nominal cover to reinforcement

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

<u>Support A</u>



Rectangular section in flexure (Section 6.1) -

Design bending moment M = **22** kNm K = **0.008** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{s,req}$ = **120** mm^2

Tens.reinforcement provided 5 × 16ϕ bars Tens.reinforcement provided $A_{s,prov}$ = **1005** mm^2

Min area of reinforcement $A_{s,min}$ = **323** mm^2 Max area of reinforcement $A_{s,max}$ = **10000** mm^2

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

Minimum bottom reinforcement at supports (cl.9.2.1.4(1))

Adj span reinforcement $A_{s,span}$ = **1571** mm^2 Min btm reinforcement rqd $A_{s2,min}$ = **393** mm^2

Btm reinforcement provided 5 × 20ϕ bars Btm reinforcement provided $A_{s2,prov}$ = **1571** mm^2

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

Rectangular section in shear (Section 6.2)

Des.shear force at support support $V_{Ed,max}$ = **179** kN Max.design shear force $V_{Rd,max}$ = **901** kN

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

Des.shear span 1 at 449 mm V_{Ed} = **98** kN

Shear reinforcement required $A_{sv,req}$ = **212** mm^2/m Min shear reinforcement $A_{sv,min}$ = **423** mm^2/m

Shear reinforcement provided 2 × 8ϕ legs at 100 c/c Shear reinforcement provided $A_{sv,prov}$ = **1005** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing $s_{vl,max}$ = **337** 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 Modulus of elasticity reinf E_s = **200000** N/mm^2

Mean conc. tensile strength $f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2 Stress distribution coefficient k_c = **0.4**

Self-equilibrating stress coef k = **0.86** Actual tension bar spacing s_{bar} = **99** mm

Max stress permitted (T.7.3N) σ_s = **320** N/mm^2 Conc/steel mod of elast. ratio α_{cr} = **6.19**

Distance of the ENA y = **246** mm Area of conc in tensile zone A_{ct} = **122966** mm^2

Min area of reinf reqd (exp.7.1) $A_{sc,min}$ = **365** mm^2

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

Quasi-perm value ψ_2 = **0.30** Quasi-perm limit state moment M_{QP} = **0** kNm

Permanent load ratio R_{PL} = **0.00** Service stress in reinf σ_{sr} = **0** N/mm^2

Max bar spacing (Table 7.3N) $s_{bar,max}$ = **300** mm

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

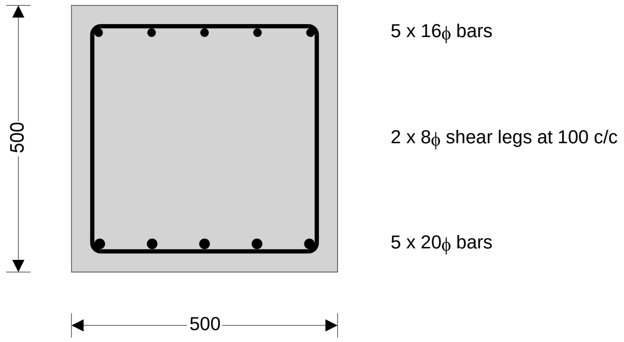
Minimum bar spacing

Minimum bottom bar spacing $s_{bot,min}$ = **98** mm Min allowable bottom spacing $s_{bar\_bot,min}$ = **45** mm

Minimum top bar spacing $s_{top,min}$ = **99** mm Min allowable top spacing $s_{bar\_bot,min}$ = **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 = **89** kNm K = **0.032** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{s,req}$ = **484** mm^2

Tens.reinforcement provided 5 × 20ϕ bars Tens.reinforcement provided $A_{s,prov}$ = **1571** mm^2

Min area of reinforcement $A_{s,min}$ = **321** mm^2 Max area of reinforcement $A_{s,max}$ = **10000** 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 Shear reinforcement provided $A_{sv,prov}$ = **1005** mm^2/m

Min shear reinforcement $A_{sv,min}$ = **423** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing $s_{vl,max}$ = **335** 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 Modulus of elasticity reinf E_s = **200000** N/mm^2

Mean conc. tensile strength $f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2 Stress distribution coefficient k_c = **0.4**

Self-equilibrating stress coef k = **0.86** Actual tension bar spacing s_{bar} = **98** mm

Max stress permitted (T.7.3N) σ_s = **321** N/mm^2 Conc/steel mod of elast. ratio α_{cr} = **6.19**

Distance of the ENA y = **244** mm Area of conc in tensile zone A_{ct} = **121889** mm^2

Min area of reinf reqd (exp.7.1) $A_{sc,min}$ = **361** mm^2

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

Quasi-perm value ψ_2 = **0.30** Quasi-perm limit state moment M_{QP} = **66** kNm

Permanent load ratio R_{PL} = **0.74** Service stress in reinf σ_{sr} = **99** N/mm^2

Max bar spacing (Table 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 $s_{bot,min}$ = **98** mm Min allowable bottom spacing $s_{bar\_bot,min}$ = **45** mm

Minimum top bar spacing $s_{top,min}$ = **99** mm Min allowable top spacing $s_{bar\_bot,min}$ = **45** mm

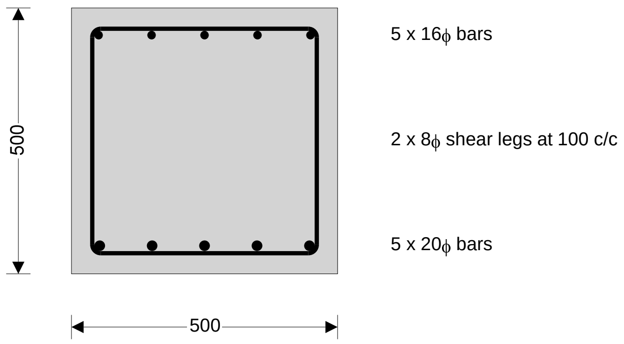
PASS - Actual bar spacing exceeds minimum allowable

Deflection control (Section 7.4)

Allowable span to depth ratio span_to_depth$_{allow}$ = **40.0** Actual span to depth ratio span_to_depth$_{actual}$ = **4.5**

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

<u>Support B</u>



Rectangular section in flexure (Section 6.1) -

Design bending moment M = **22** kNm K = **0.008** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{s,req}$ = **120** mm^2

Tens.reinforcement provided 5 × 16ϕ bars Tens.reinforcement provided $A_{s,prov}$ = **1005** mm^2

Min area of reinforcement	$A_{s,min}$ = **323** mm^2	Max area of reinforcement	$A_{s,max}$ = **10000** mm^2

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

Minimum bottom reinforcement at supports (cl.9.2.1.4(1))

Adj span reinforcement	$A_{s,span}$ = **1571** mm^2	Min btm reinforcement rqd	$A_{s2,min}$ = **393** mm^2
Btm reinforcement provided	5 × 20ϕ bars	Btm reinforcement provided	$A_{s2,prov}$ = **1571** mm^2

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

Rectangular section in shear (Section 6.2)

Des.shear force at support support $V_{Ed,max}$ = **179** kN Max.design shear force $V_{Rd,max}$ = **901** kN

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

Des.shear span 1 at 1551 mm	V_{Ed} = **98** kN		
Shear reinforcement required	$A_{sv,req}$ = **212** mm^2/m	Min shear reinforcement	$A_{sv,min}$ = **423** mm^2/m
Shear reinforcement provided	2 × 8ϕ legs at 100 c/c	Shear reinforcement provided	$A_{sv,prov}$ = **1005** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing $s_{vl,max}$ = **337** 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	Modulus of elasticity reinf	E_s = **200000** N/mm^2
Mean conc. tensile strength	$f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2	Stress distribution coefficient	k_c = **0.4**
Self-equilibrating stress coef	k = **0.86**	Actual tension bar spacing	s_{bar} = **99** mm
Max stress permitted (T.7.3N)	σ_s = **320** N/mm^2	Conc/steel mod of elast. ratio	α_{cr} = **6.19**
Distance of the ENA	y = **246** mm	Area of conc in tensile zone	A_{ct} = **122966** mm^2
Min area of reinf reqd (exp.7.1)	$A_{sc,min}$ = **365** mm^2		

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

Quasi-perm value	ψ_2 = **0.30**	Quasi-perm limit state moment	M_{QP} = **0** kNm
Permanent load ratio	R_{PL} = **0.00**	Service stress in reinf	σ_{sr} = **0** N/mm^2
Max bar spacing (Table 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	$s_{bot,min}$ = **98** mm	Min allowable bottom spacing	$s_{bar\_bot,min}$ = **45** mm
Minimum top bar spacing	$s_{top,min}$ = **99** mm	Min allowable top spacing	$s_{bar\_bot,min}$ = **45** mm

PASS - Actual bar spacing exceeds minimum allowable

	Wall DL	203	kN/m	ϕ	30		Wall DL	203	kN/m		
	w =	0.3	m	δ	18						
Sur l =	10			Span=	14	m			Sur2 =	10	
					H 2=	3	m		Water =	3.6	m
			H 1=	3	m						
			Slab Thickness =		0.4						
Heel=	0			Slab =	11.2						
			Toe =	0.35	m						
			Toewidth=	1.4	m						
		57.2958									
	kp =	3									
	ka =	0.33333									
			Thrust from left =	19				Thrust from Right	19		
			Resitance from Left =	162				Resitance from Right =	162		
		Equilibrioum check		Kp from Right Adequate							
				Kp from left Adequate							

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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

35 Greville Road
London
NW6 5JB

Client Information
Igor Gokhberg
55 Peiantho CRT
Vaughan
ON
LGA 1G1
Canada

Contents

CROFT STRUCTURAL ENGINEERS

35 Greville Road

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 35 Greville Road 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.

1.0

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

1.7. The bearing pressures have been limited to 130-150kN/m^2. This is standard loadings for local ground conditions and acceptable to building control and their approvals.

1.8. The water table is expected to encountered at 0.8m BGL

2.0

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

3.0

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.

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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.

4.0

2.4. Dewater: Water is expected at 0.5 depths

5.0

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.

6.0

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

7.0

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

8.0

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.

9.0

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.

10.0

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.

11.0

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.

12.0

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.

13.0

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.

14.0

15.0

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.

16.0

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

17.0

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.

18.0

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

19.0

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

20.0

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.

21.0

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

22.0

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

23.0

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

24.0

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

25.0

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

26.0

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.

27.0

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

28.0

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.
29.0

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
30.0

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

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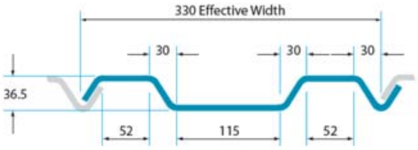
STANDARD LAP TRENCH SHEETING

STANDARD LAP

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



Technical Information

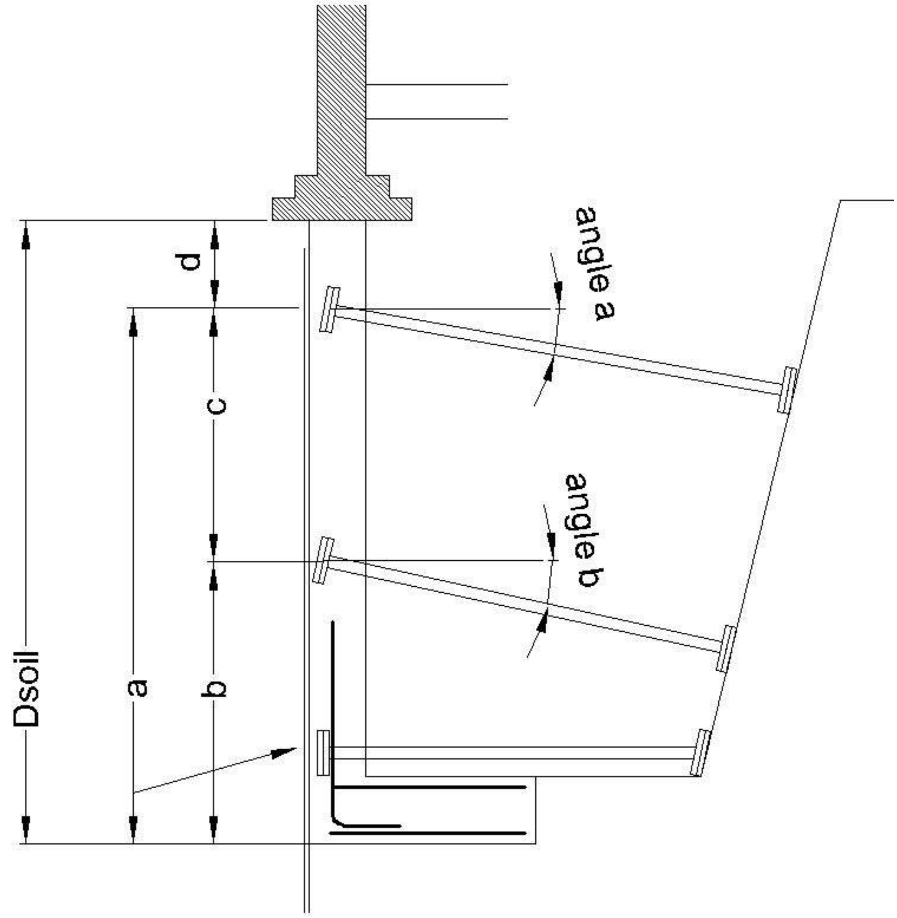
Effective width per sheet (mm)	330
Thickness (mm)	3.4
Depth (mm)	35
Weight per linear metre (kg/m)	10.8
Weight per m² (kg)	32.9
Section modulus per metre width (cm³)	48.3
Section modulus per sheet (cm³)	15.9
I value per metre width (cm⁴)	81.7
I value per sheet (cm⁴)	26.9
Total rolled metres per tonne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

$S_{xx} = 15.9\ cm^3$

$p_y = 275 N/mm^2$

$I_{xx} = 26.9 cm^4$

$A = (1m^2 * 32.9kg/m^2) / (330mm * 7750kg/m^3) =$ **12864.125**mm^2

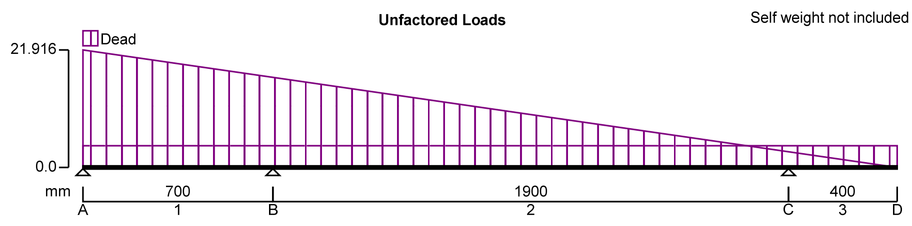


Length a a = **2.600** m

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"**

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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

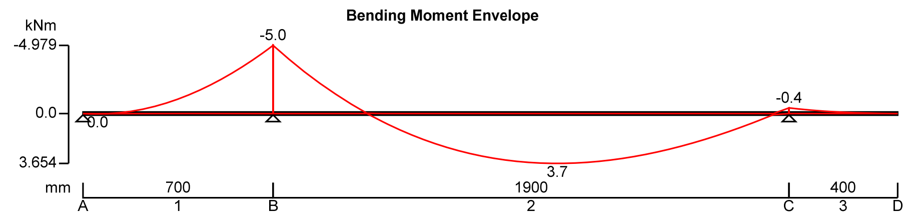
Support A	Max react = **-1.4** kN	Min react = **-1.4** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-32.8** kN	Min react = **-32.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-10.8** kN	Min react = **-10.8** kN	Max mom = **0.0** kNm	Min mom = **0.0**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

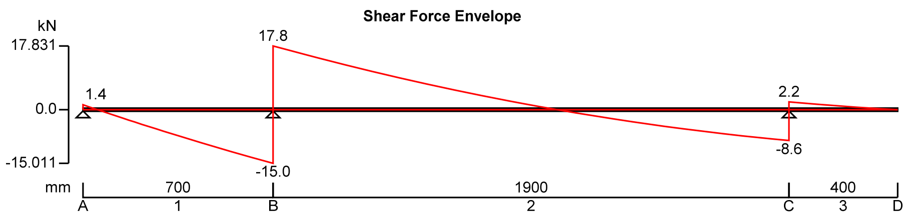
Beam Max/Min results - Combination Summary

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

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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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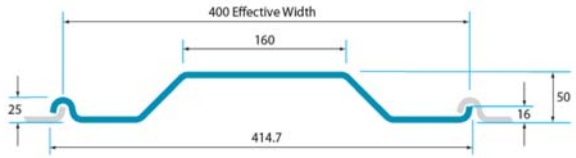
KD4 SHEETS

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

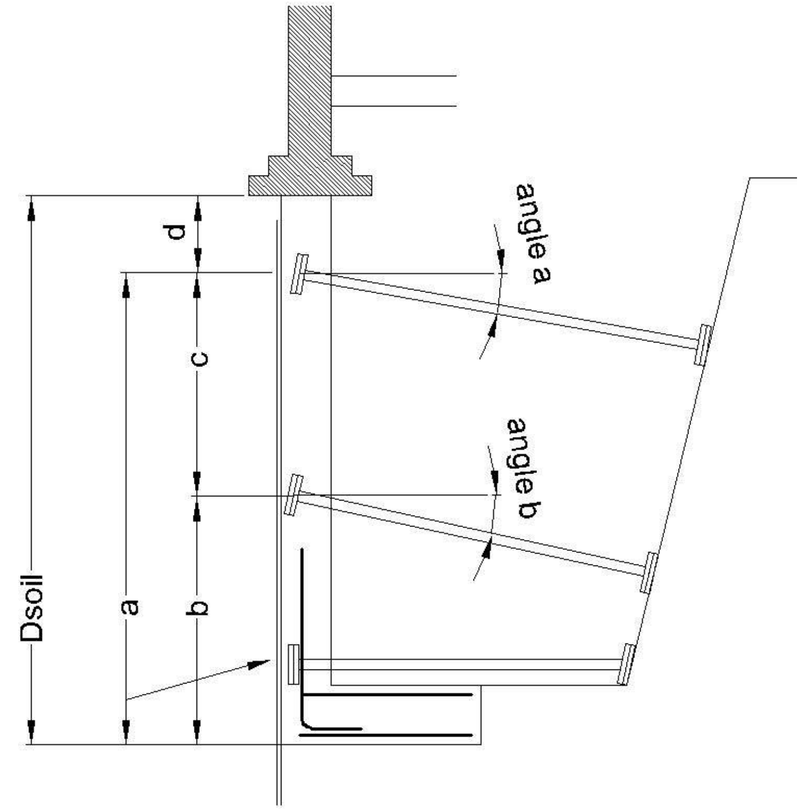
Effective width per sheet (mm)	400
Thickness (mm)	6.0
Depth (mm)	50
Weight per linear metre (kg/m)	21.90
Weight per m^2 (kg)	55.2
Section modulus per metre width (cm^3)	101
Section modulus per sheet (cm^3)	40.34
I value per metre width (cm^4)	250
I value per sheet (cm^4)	101
Total rolled metres per tonne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

Sxx = $48.3cm^3$

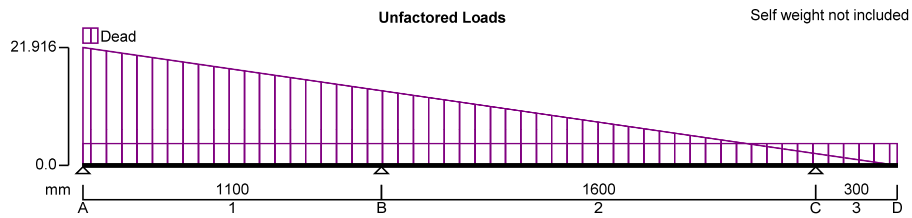
py = $275N/mm^2$

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.×10³** 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

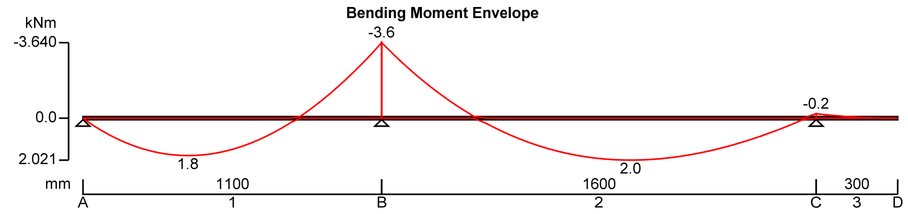
Support A	Max react = **-9.5** kN	Min react = **-9.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-28.0** kN	Min react = **-28.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-7.5** kN	Min react = **-7.5** kN	Max mom = **0.0** kNm	Min mom = **0.0**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

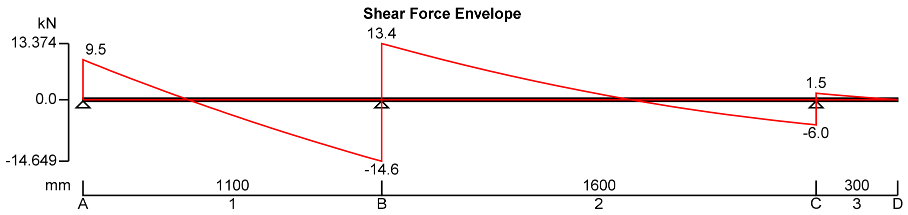
Beam Max/Min results - Combination Summary

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

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SRU 4.0

Safe working loads for Acrow Props — loads given in kN

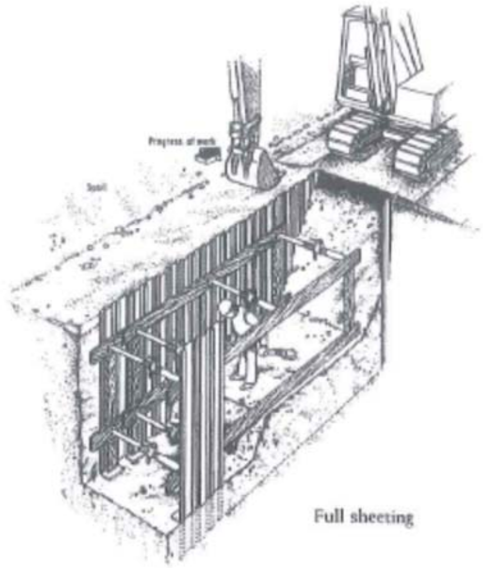
For normal purposes 1 kilo Newton (kN) = 100 kg	Height	m / ft	2.0 / 6.6	2.25 / 7.4	2.5 / 8.2	2.75 / 9.0	3.0 / 9.8	3.25 / 10.7	3.5 / 11.5	3.75 / 12.3	4.0 / 13.1	4.25 / 13.9	4.5 / 14.8	4.75 / 15.6
TABLE A Props loaded concentrically and erected vertically	Prop size 1 or 2		35	35	35	34	27	23						-
	Prop 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

Mabey 25 acros and piles to be used

Sheeting requirements

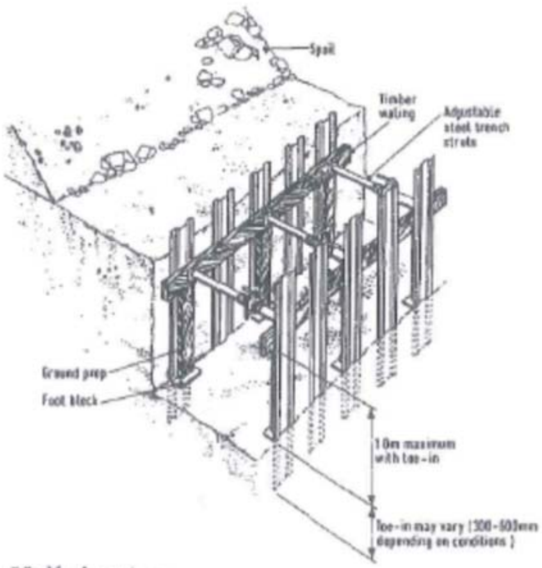
Ground Type	Trench Depth, D: ~~less than 1.2m(1)~~	1.2 to 3m	3 to 4.5m	4.5 to 6m
Sands and gravels Silt Soft Clay High compressibility Peat	~~Close, ½, ¼, ⅛ or nil~~	Close	Close	Close
Firm/stiff Clay Low compressibility Peat	~~¼, ⅛ or nil~~	½ or ¼	½ or ¼	Close or ½
Rock(2)	From ½ for incompetent rock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

Sheeting requirements



Full sheeting

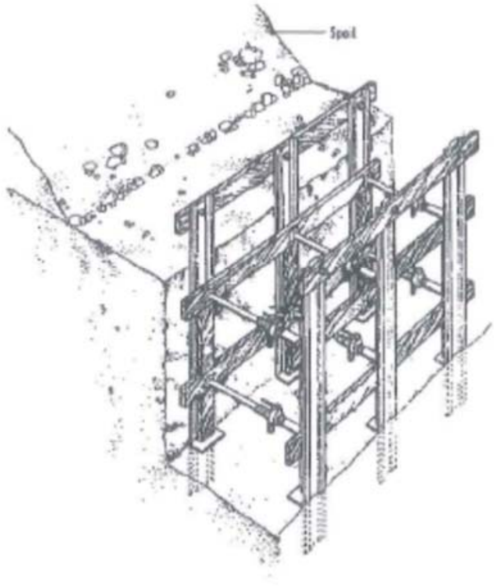
Sheeting requirements



Half sheeting
shown for 1.5 m deep trench

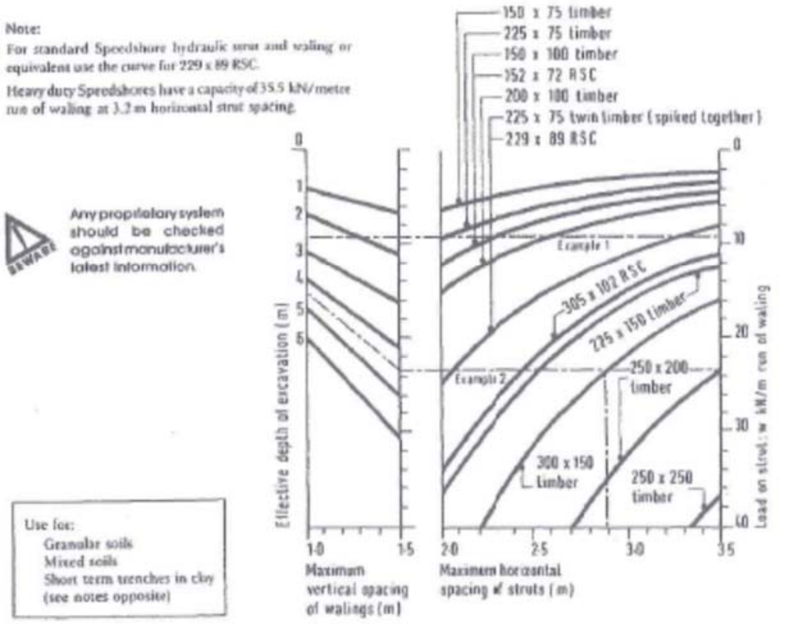
11/04/2012

Sheeting requirements



Quarter sheeting

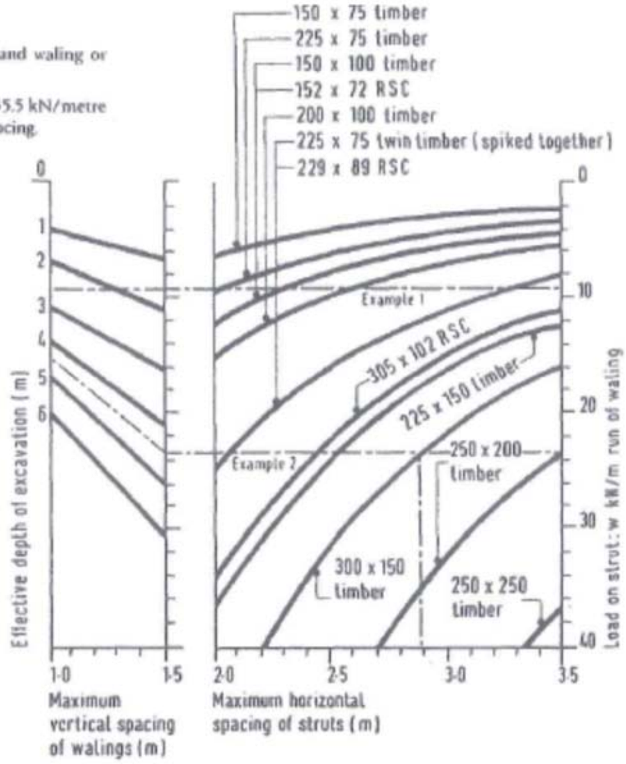
Design to CIRIA 97



CROFT STRUCTURAL ENGINEERS

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.



Use for:
Granular soils
Mixed soils
Short term trenches in clay (see notes opposite)

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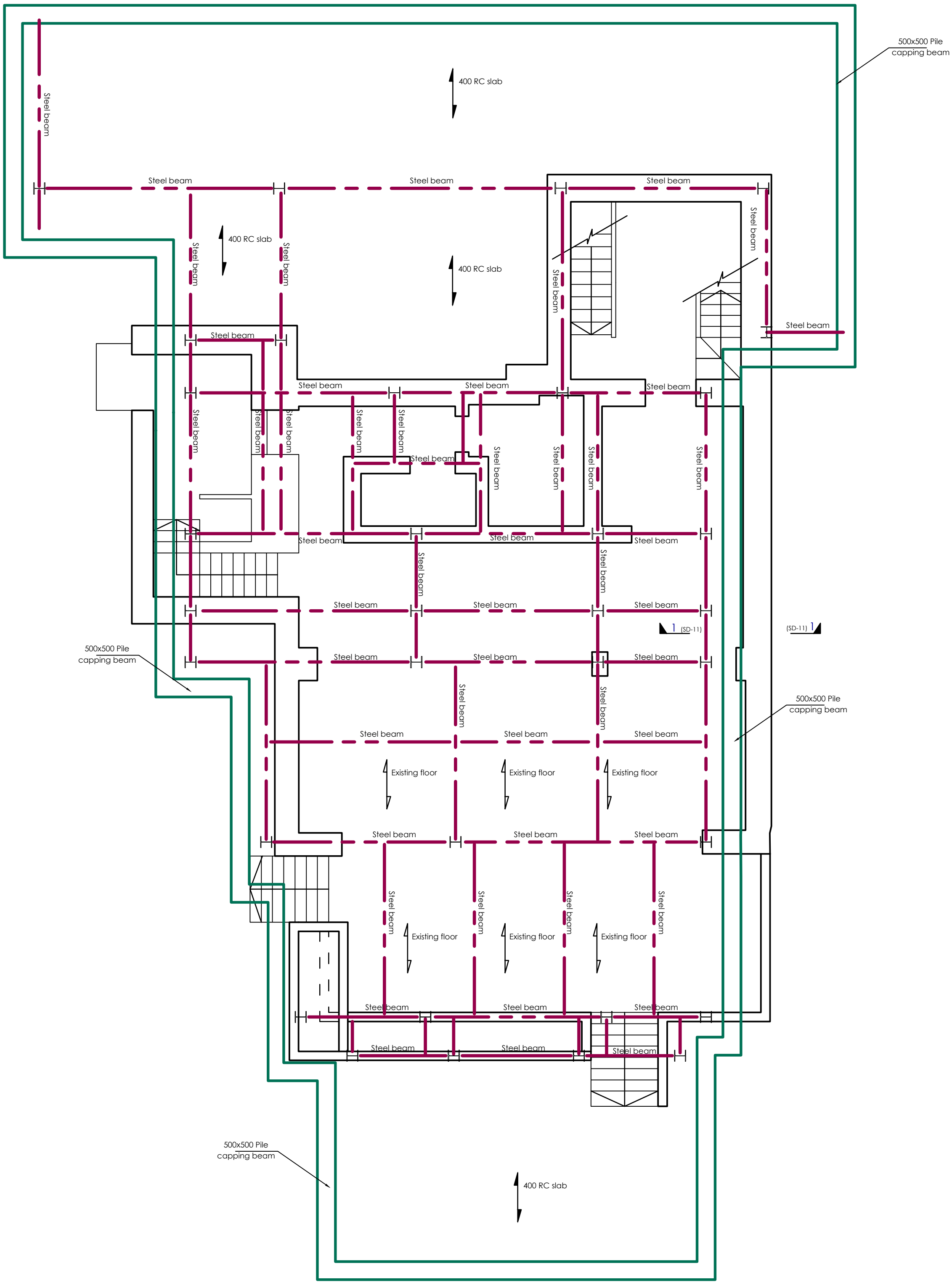
Appendix B : Structural Drawings

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



Lower Ground Floor plan

Scale (1:100)

Rev	Date	Amendments
-	27/08/15	First issue for comment

Client: Igor Gokhberg

Project: 35 Greville Road

Title : Lower Ground Floor plan BIA

Job No.s	Drawn	Date
150525	NM	Aug 15
Dwg Nos	**Rev**	**Scale**
SL-20	-	As Shown @ A3

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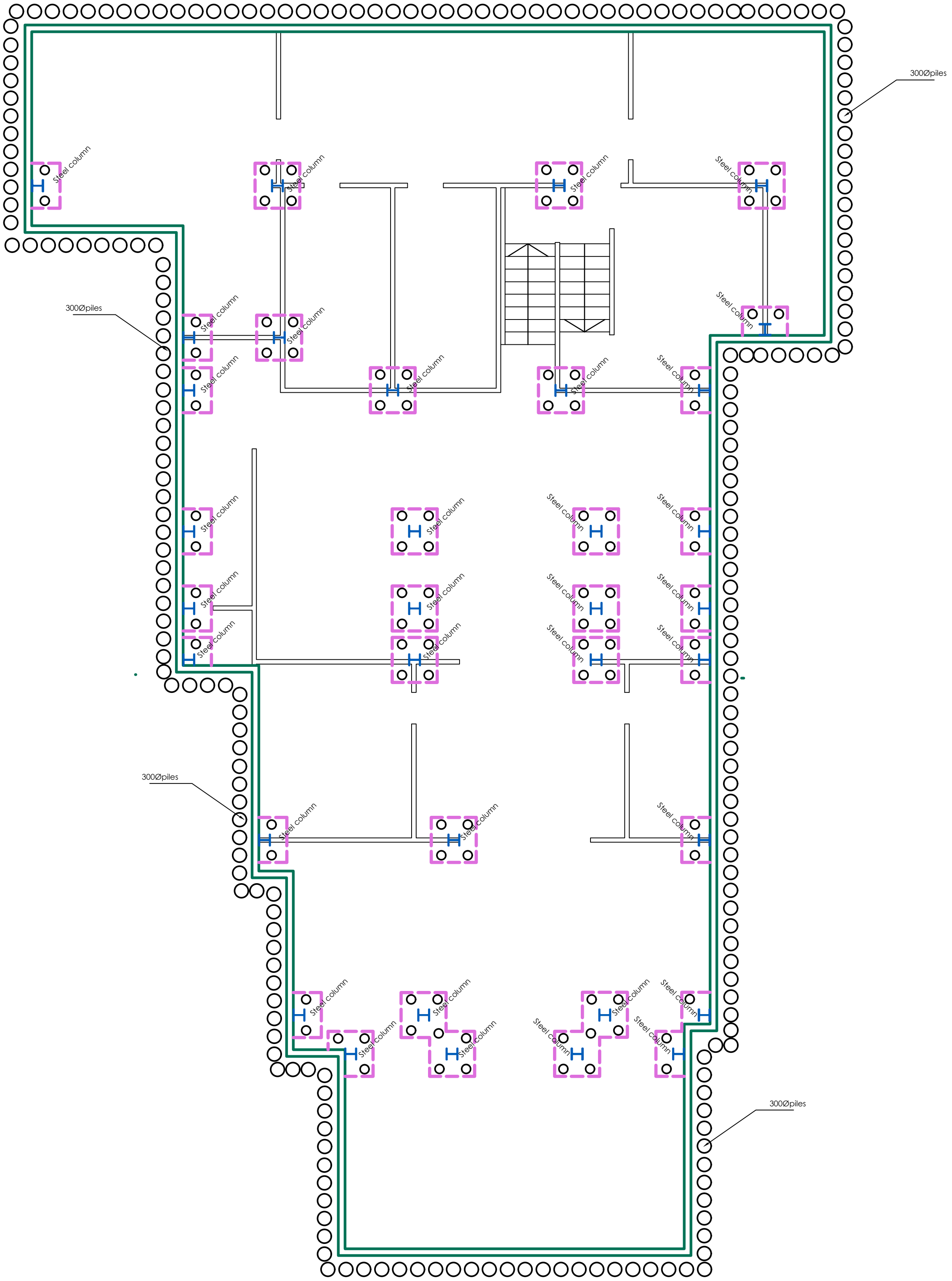

Basement plan

Scale (1:100)

Rev	Date	Amendments
-	27/08/15	First issue for comment

Client: Igor Gokhberg

Project: 35 Greville Road

Title : Basement Floor plan BIA

Job No.s	Drawn	Date
150525	NM	Aug 15
Dwg Nos	**Rev**	**Scale**
SL-10	-	As Shown @ A3

Croft Structural Engineers

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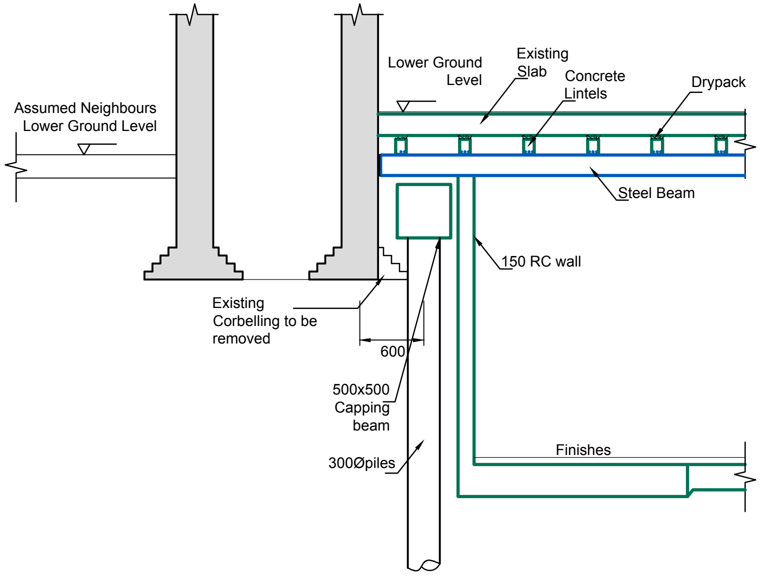

Section 1-1

Scale (1:50)

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.

12. 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.

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

14. 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.

15. 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.

16. 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.

17. 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.

18. 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.

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

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

21. 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.

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

23. 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
-	27/08/15	First issue for comment

Job No.s: 150525

Dwg Nos: SD-11

Date: Aug 15

Drawn: NM | Chk'd: CT

Scale: As shown @ A3 | Rev: -

Client: Igor Gokhberg

Project: 35 Greville Road

Title : Lower Ground Floor plan BIA

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