Job Number: 150527
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CROFT STRUCTURAL ENGINEERS

Croft Structural Engineers
Clock Shop Mews
Rear of 60 Saxon Road
London SE25 5EH

T: 020 8684 4744
E: enquiries@croftse.co.uk
W:www.croftse.co.uk

Basement Impact Assessment

Property Details
54 Shirlock Road
London
NW3 2HS

Client Information
Alex & Grace Key

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

Hydrology Report	**Geology Report**
Frances Bennett CGeol Separate report	Frances Bennett CGeol Separate Report

Revision	Date	Comment
-	13.07.2015	First Issue for comment

LABC Regional winner 2013 awards constructionline TRADA ice The Institution of Structural Engineers

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

No. 54 Shirlock Road comprises four storeys and an unconverted cellar. The property is Victorian mid-terrace house with a front yard and a rear garden.

Proposed Works

The proposed works require the construction of:

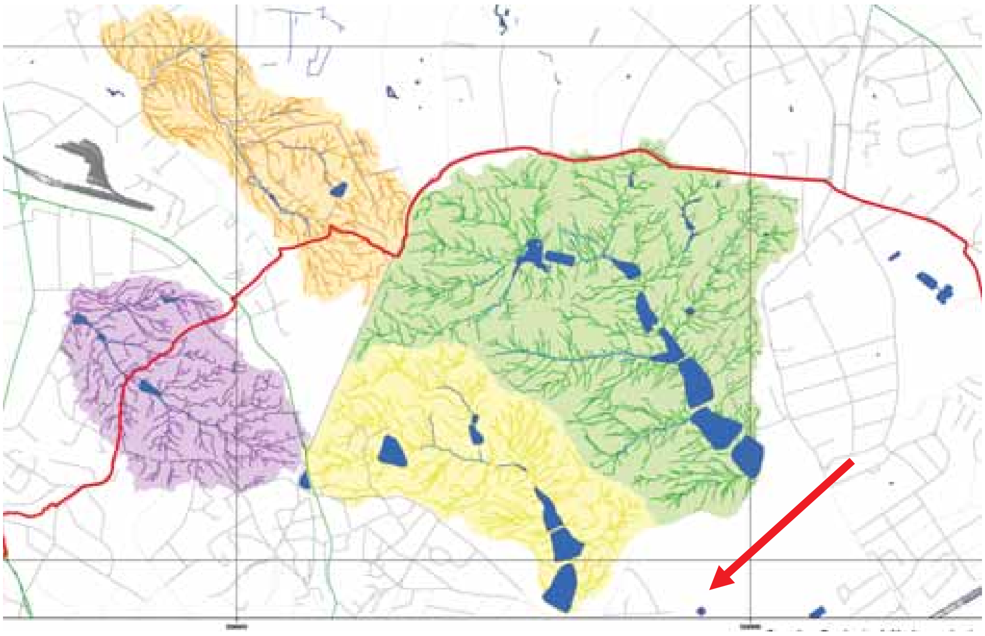
- A new basement under the foot print of property, proposed side extension and part of the front and rear gardens.
- Lightwells to the front and rear
- Garden basement
  - Roof slab to the garden
  - SUDS (Storm water storage above the garden area)
  - Covering garden slab with new top soil
- Superstructure works above the basement
  - Ground floor side return extension
  - Alterations to existing ground floor
    The superstructure works has been considered but is not required to be detailed at planning so has not been included in the Basement Impact Assessment.

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 54 SHIRLOCK ROAD is:

	1. Excavate front to allow for conveyor to be erected. 2. Safely and securely support the existing building above 3. Form lightwell with cantilevered retaining walls 4. Slowly work from the front to the rear inserting narrow cantilevered retaining walls sequentially using well developed and understood underpinning methods. 5. Prop retaining walls in temporary condition back to the central soil "dumpling". 6. Prop across the width of the basement, excavate central soil "dumpling" & cast basement slab 7. Waterproof internal space with a drained cavity system.
Stage 1 – Screening	Screening identified areas of concern and concluded a requirement to proceed to a scoping stable for the Land stability, Hydrology, Surface Water and flooding.
Stage 2 – Scoping	The Scoping stage identified the potential impacts and set the parameters required for further study of the areas of concern highlighted in the Screening phase. 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.
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. The ground comprised of a superficial covering of topsoil and concrete overlying made ground down to 0.45m to 1.00mbgl. The made ground was

	everywhere underlain by low strength orange brown grey silty clay becoming grey brown very silty clay with blue veins proven to a depth of 4.45m bgl. Laboratory testing was undertaken on the soil samples. Ground water has been measured over repeat visits to determine water levels and flows. • No ground water was encountered in boreholes. However, groundwater was encountered during monitoring at depths of 2.8m bgl within the London Clay. It is expected that limited perched groundwated may be encountered within the made ground and London Clay during construction.
Stage 4 – Impact assessment	**Land stability** See Geology Report **Hydrogeology** See hydrology report not within this report. **Drainage & Surface Water Flow** The risk of flooding from excess of surface water is not considered significant. There is a risk of flooding due to the failure of the pumping system, although this risk is inherent in all subterranean structures, which have an incoming supply of water. The risk can be reduced to acceptable levels with appropriate design measures.

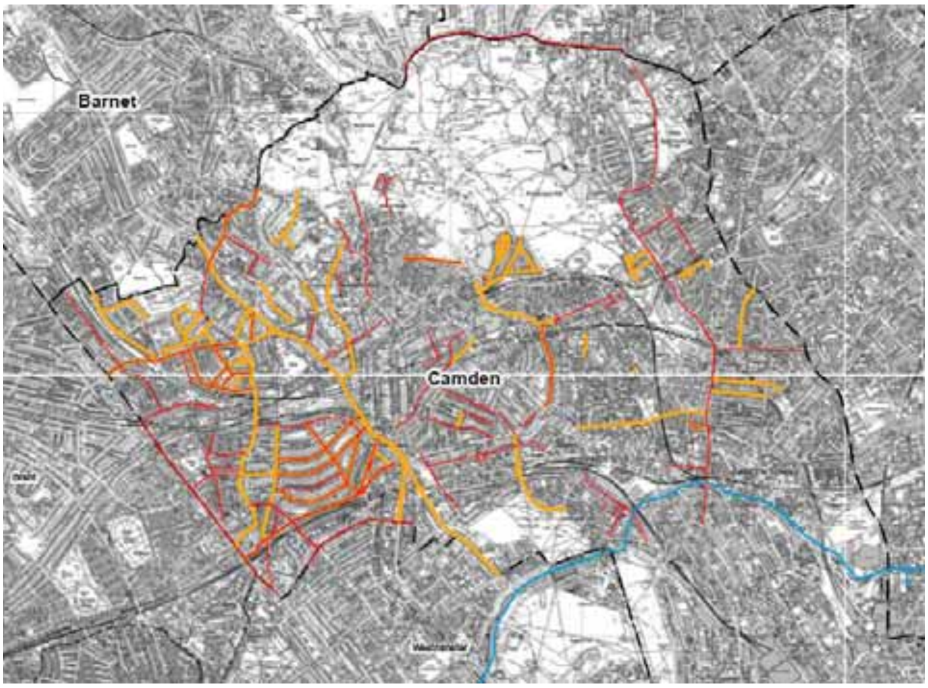
1. Screening Stage

	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?**  **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. Due to the construction of the side extension and the rear light well the flow of the water into the ground and the existing surface water drainage system may change. Carry forward to scoping.
	Question 3. Will the proposed basement development result in a change to the hard surfaced / paved external areas? **Unknown** – The light wells may reduce the impermeable areas. Carry forward to scoping **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 Garden basement may reduce the impermeable areas. Carry forward to scoping **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.

Surface water (pluvial) flooding	No	The 54 SHIRLOCK ROAD is NOT 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

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 54 SHIRLOCK ROAD require in insertion of a basement. The basement is under the footing print of existing property, proposed side extension and part of the front and rear gardens with lightwells. The basement enlarges the existing single dwelling and is not an additional unit. Lightwells have hardstanding slightly which may increase flow. 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.
	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 light wells may reduce the impermeable areas. Carry forward to Site Investigation & desk Study **Unknown** –The Garden basement may reduce the impermeable areas. Carry forward to Site Investigation & desk Study
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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 Eleni Pappa, MSc, BEng(Hons), an Engineer from Croft Structural Engineers visited 54 SHIRLOCK ROAD. Date of inspection was on the 12th June 2015. The data collected from this survey corroborates and adds to information obtained from the desk study.

Proposed Development	The proposed works require the construction of:
	
	• A new basement under the foot print of property, proposed side extension and part of the front and rear gardens.
	• 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
	
	• Superstructure works above the basement
	  o Ground floor side return extension
	  o Alterations to existing ground floor
	  The superstructure works has been considered but is not required to be detailed at planning so has not been included in the Basement Impact Assessment.
	
	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. The slope is less than a 7 degres from the horizontal.

Figure 1: Front of the property

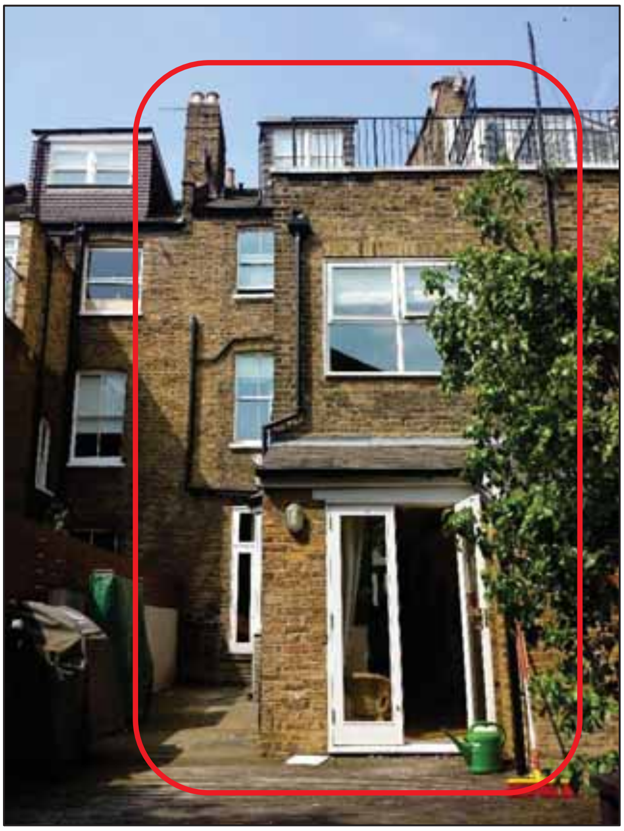

Figure 2: Rear of the property

Site History

The history of the site has been analysed using OS County Series Maps.

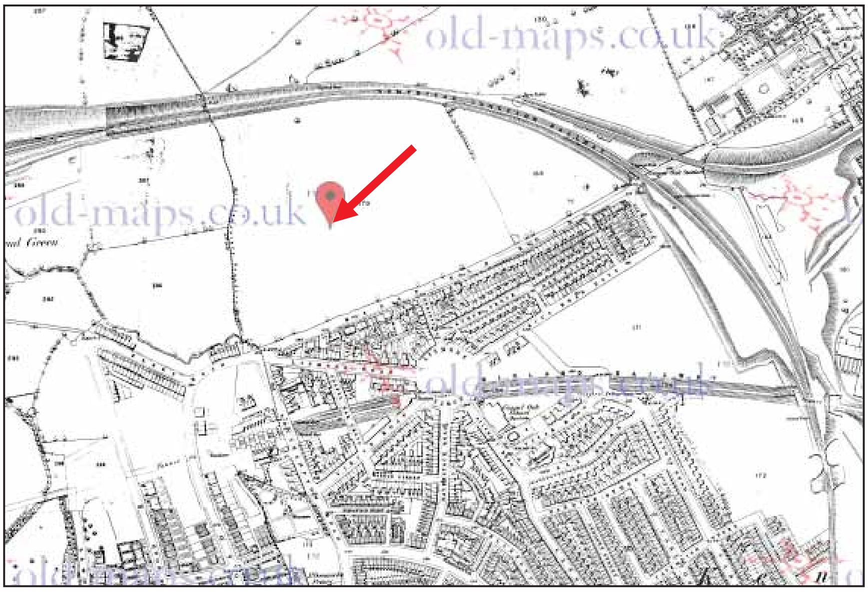

Figure 3: OS County Series 1873-1879, 1:2500

The 1873-1879 map indicates that the site at this time comprises an open field with railway tracks to the north,. Approximately 200m south from the site the area is developed with residential houses.

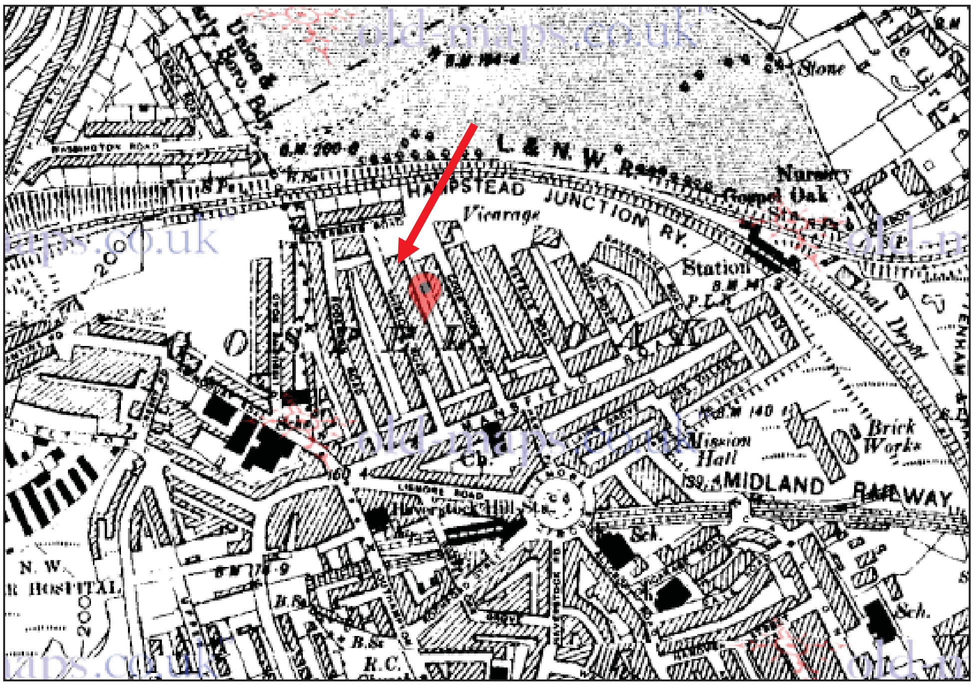

Figure 4: OS County Series, London 1896, 1:10560

The site and surrounding area has changed considerably. The site is occupied by a terraced house.

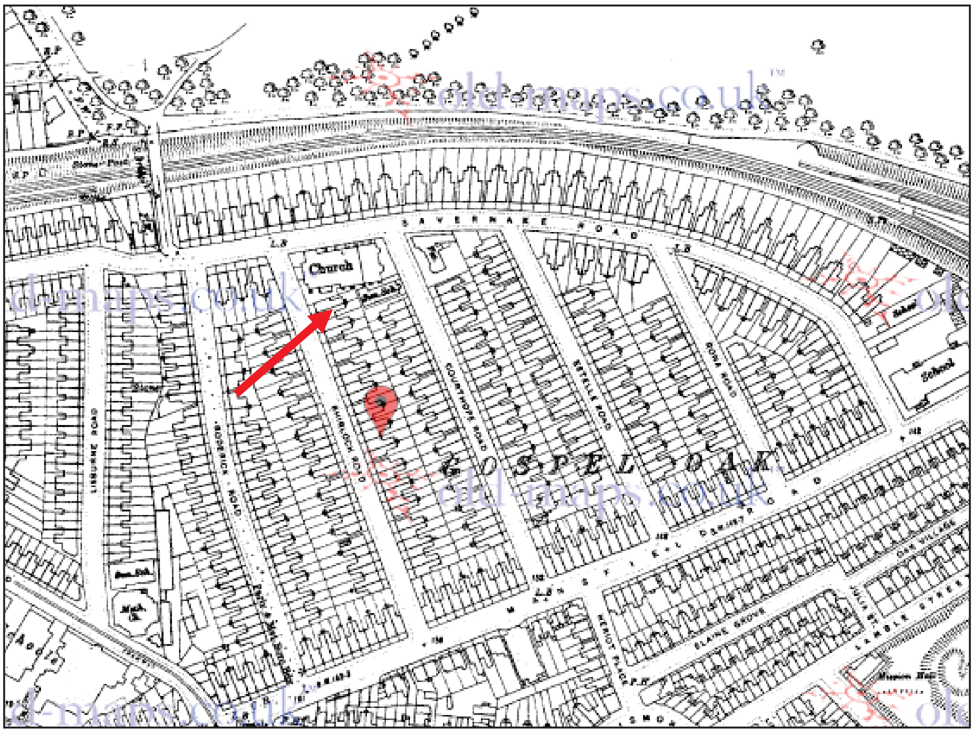

Figure 5: OS County Series, London 1916, 1:2500

The 1916 map shows that the area has undergone a general increase in residential properties. All Hallows' Church has been built.

Local Bombing

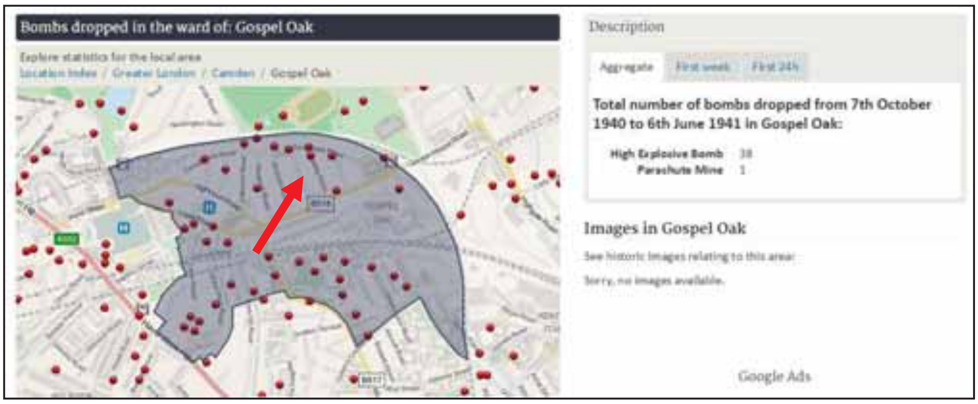

Figure 6: Gospel Oak bomb survey map

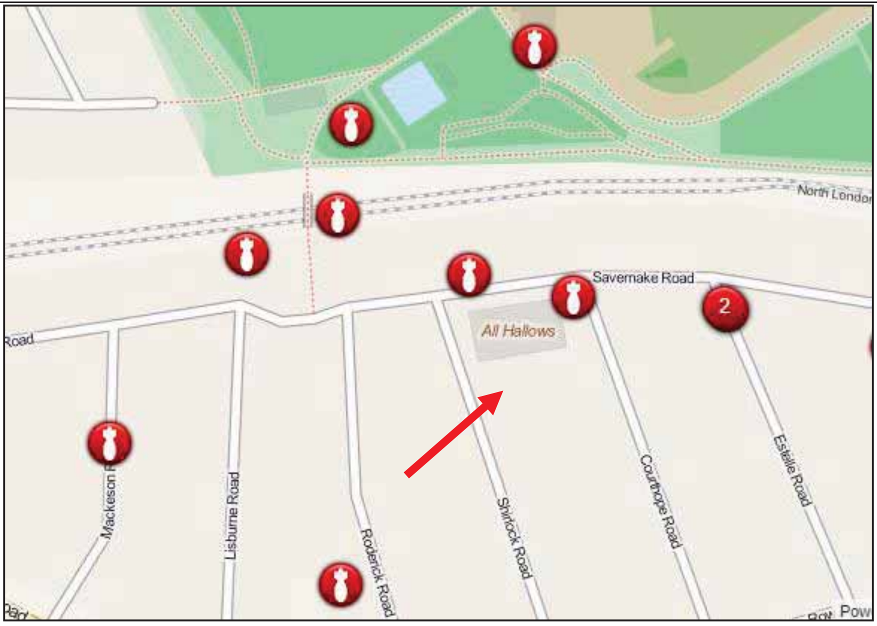

Figure 7: Extract from bomb survey map

From the bombsight website it is shown that there are reported bomb sites in the near vicinity of the site.

Listed Buildings

Is the building or Adjacent buildings listed

No.

Figure 8: Historic England Map - The National Heritage List of England

However, 54 Shirlock Road and adjacent building does fall within the Mansfield Conversation Area.

Highways, Rail and London Underground

Yes. Site is within 5m of the footpath and the road surface is less than 5m from the proposed 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, +/- 90m from site.

The nearest Overground station is Gospel Oak approximately 420m from the property.

Croft Structural Engineers are unaware of any other tunnels or infrastructure located near to the proposed development. Given the distance between the site and the nearest overground line, it is reasonable to expect that the basement development proposed will have no adverse effects on any ovrerground infrastructure in the area.

Figure 9: Open Street map showing proximity to rail lines

UK Power Networks

Will the basement works affect any UK Power Network Assets?

(Substations etc)

No. There are no significant items of electrical infrastructure (such as pylons or substations) in the immediate vicinity.

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?

Yes. The pear tree 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: • A number of minor cracks were noted. However, given the narrow width of the cracks, this is considered a non-structural defect which can be amended with standard decorative works. • Major dampness was noted at the existing kitchen area where the side bay window is positioned. This is a result of failed drainage in the past, however, the issue has now been solved as the wall was dry. Furthermore, this wall will be demolished and extended to the side in later date as part of a ground floor extension. Pictures of these are shown below. There is no sign of any ongoing movement to the existing property.  *Figure 10: Wall damage to wall in breakfast area*


Figure 11: Example of minor cracking


Figure 12: Vertical crack in the wall - dining area

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 external facades have been undertaken of the properties.

Figure 13: Location map

No 56 Shirlock Road – Property to Left

Property Age : Approximately 100.

Property use : Residential
Number of storeys : 5

Is a basement present? : Cellar is present adjacent to No54 Shirlock Road.

Structural Defects Noted: No structural defects were noted on the external face of the property to the front, rear and sides.

Structural Assessment of ongoing movement: No signs of cracking was noted to the external face of No.56 Shirlock Road. No movement noted.

Figure 14: Front elevation of 56 Shirlock Road

Figure 15: Rear elevation of 56 Shirlock Road

	 Figure 16: Rear roof terrace party wall  Figure 17: Boundary garden wall between 54and 56 Shirlock Road
No 52 Shirlock Road – Property to Right	Property Age : Approximately 100. Property use : Residential Number of storeys : 5 Is a basement present? : Cellar is present adjacent to No 50 Shirlock Road. Structural Defects Noted: No structural defects were noted on the external face

of the property to the front, rear and sides.

Structural Assessment of ongoing movement: No signs of cracking was noted to the external face of No.52 Shirlock Road. No movement noted.

Figure 18: Front elevation of 52 Shirlock Road

Figure 19: Rear elevation of 52 Shirlock Road

Figure 20: Side elevation of No 52 Shirlock Road

Figure 21: Rear wall of No 52 Shirlock Road

No 53 Courthope	Property Age : Approximately 100

Road– Property to Rear	Property use : Church Hall Number of storeys : 2 Is a basement present? : Unknown. Structural Defects Noted: No structural defects were noted on the external face of the property to the front, rear and sides. Structural Assessment of ongoing movement: No signs of cracking was noted to the external face of No.53 Shirlock Road. No movement noted.  *Figure 22: Front elevation of No. 53 Courthope Road*

Figure 23: Left hand side wall view of No 53 Courthope Road

Figure 24: Right hand side wall of No 53 Courthope Road

Figure 25: Rear views of 53 Courthope Road

Local Topography

The area surrounding the property has a general slope, downwards from north-west to south-east. The slope is gradual and less than 7 degrees. There are brick retaining walls between adjacent gardens.

Ground Investigation	Refer to the ground investigation report , which is submitted as a separate document.
Geology	See Ground investigation report and Geology report.

Surface Flow & Flooding	
Areas of Hard Standing present on site	Existing Area of hardstanding outside is ; Area = 65m^2  *Figure 26: Rear garden hardstanding - Timber decking over concrete slab*  *Figure 27: Front yard hardstanding*

Rainwater down pipes, Drains, Manholes and Gulleys	According to the Architects plans, there are two manholes in the rear garden and two manhole in the front yard.  *Figure 28: Front yard manholes*  *Figure 29: manhole in the rear garden*


Figure 30: Rwp and svp pipes

Local Water Sources	**Are there any ponds lakes or water courses on the site or adjacent sites?** No.

Field Investigation

Ground investigation specialists visited the site and subsequently produced are report for the existing ground and groundwater conditions.

	Monitoring, Reporting and Investigation
	The ground investigation report, which has data from initial site investigations and data from subsequent monitoring, is available as a separate report. Data relevant to land stability and subterranean flow is examined separate documents as described below
Land Stability	Refer to Chartered Geologist Report for land stability issues addressed to Stage 3. Features and items of concern relating to data from Stage 3 are included in this report.
Subterranean Flow	Refer to Chartered Hydrogeologist report (Basement Impact Assessment: Groundwater). This is completed by aHydrogeologist with the "CGeol" (CharteredGeologist) qualification from the Geological Society ofLondon. Features and items of concern relating to data from Stage 3 are included in this report.

Site Investigation	
Soil investigation Brief	The Soil investigation brief was completed by Aston Bennett. Soil Report is provided under a separate cover.

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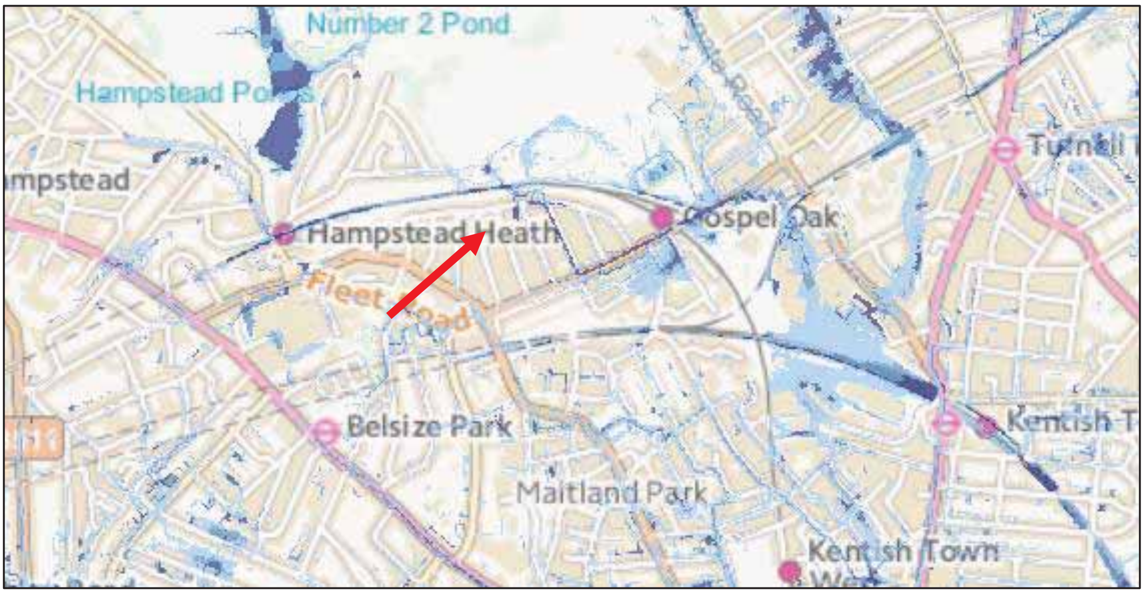
4. Basement Impact Assessment

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

The site is not on the list of flooded Camden streets therefore no flood risk assessment is necessary.

The environmental agency maps show that the area was not flooded by surface water.



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SUDS Assessment

Hard standing

The main design change resulting in the reduction of hardstanding is the removal of the concrete slab (approximately $18m^2$ in plan area) in the rear courtyard. The proposed landscaping for the rear yard has not been designed in detail. .

Figure 31: Existing hardstanding in the rear garden

Existing Hard Standing = 65 m^2

Proposed Hardstanding = 65 m^2

Percentage Increase in Hard standing = 0 %

SUDS Assessment

From review of the existing and proposed hardstanding the increase will be?

No Increase in the hardstanding.

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

Where garden basements are present then a soil band of a minimum of 1m should be provided.

Where 1m of soil is not present then SUDs is required

SUDS Calculations

The calculations below refer to the rear yard. The area of hardstanding is $40m_2$. This is equivalent to 0.004 hectares (due to rounding presentations within the calculations, this is misleadingly presented as 0.00ha in the table below).

ATTENUATION DESIGN

In accordance with CIRIA publication C697 - The SUDS Manual

Tedds calculation version 1.0.01

EA_Defra method

Site characteristics

Location	London	
Hydrological region	6	Soil type (W.R.A.P map)
Standard percentage runoff	SPR = **0.47**	Average annual rainfall
5yr rainfall of 60min duration	M5_60min = **20.0** mm	Rainfall ratio
Global warming rainfall factor	$p_{climate}$ = **0**%	
Imperv. area req. att storage	α = **100.0** %	

Catchment details

Subcatchment	Name	Area (ha)	PIMP (%)	Impermeable. area (ha)
1	**rear yard**	**0.00**	**50.0**	**0.00**
0.8	Total	**0.00**	**50.0**	**0.00**

Greenfield runoff rates

Catchment area	AREA = **50.00** hectare	Greenfield runoff rate (50
ha)	$\bar{Q}_{rural}$ = **201.6** l / s	
Greenfield runoff rate	$\bar{Q}$ = **0.0** l / s	G'field runoff rate (unit
area)	$\bar{Q}_A$ = **4.0** l / s / hectare	

Estimated site discharges

FSR growth rate (1 year)	FSR_{1yr} = **0.85**	Discharge (1 year)
FSR growth rate (30 year)	FSR_{30yr} = **2.30**	Discharge (30 year)
FSR growth rate (100 year)	FSR_{100yr} = **3.19**	Discharge (100 year)

Estimated attenuation volume - 1 year

Attenuation storage vol	$Uvol_{1yr}$ = **54.8** m^3 / hectare	Basic storage volume
FEH rainfall factor	FF_{1yr} = **0.90**	Storage volume ratio
Adjusted storage volume	ASV_{1yr} = **0.25** m^3	Hydrological regional vol
ratio	HR_{1yr} = **1.01**	
Final est. attenutation storage	Vol_{1yr} = **0.25** m^3	

Estimated attenuation volume - 30 year

Attenuation storage vol	$Uvol_{30yr}$ = **134.8** m^3 / hectare	Basic storage volume
FEH rainfall factor	FF_{30yr} = **0.85**	Storage volume ratio
Adjusted storage volume	ASV_{30yr} = **0.66** m^3	Hydrological regional vol
ratio	HR_{30yr} = **1.02**	
Final est. attenutation storage	Vol_{30yr} = **0.67** m^3	

Estimated attenuation volume - 100 year

Attenuation storage vol	$Uvol_{100yr}$ = **174.7** m^3 / hectare	Basic storage volume
FEH rainfall factor	FF_{100yr} = **0.80**	Storage volume ratio
Adjusted storage volume	ASV_{100yr} = **0.94** m^3	Hydrological regional vol
ratio	HR_{100yr} = **1.03**	
Final est. attenutation storage	Vol_{100yr} = **0.96** m^3	

Attenuation storage required

Max attenuation storage reqd V_{req_max} = **0.8** m^3

Interception storage

Interception rainfall depth	d_{int} = **5** mm	Interception storage reqd

Long term storage

Prop of paved area draining	α = **1.0**	Prop of pervious area
draining	β = **0.5**	
Rainfall 100years, 6 hour	RD = **60.1** mm	Extra runoff over g'field
runoff	Vol_{xs} = **0.11** m^3	

Treatment volume

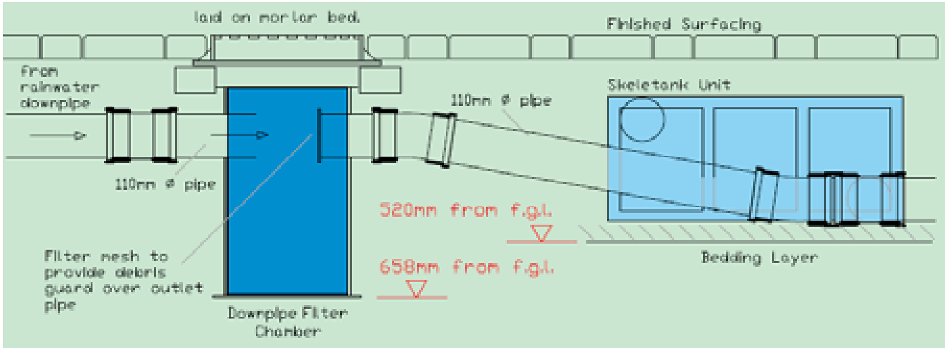
Treatment volume (assume 80% runoff)	T_{vol} = **0.24** m^3

Library item: Treatment volume summary

Mitigation Measures

From the SUDS calculations an volume of 0.8m^3 is required for storage we would recommend that onsite storage if used.

We recommend that a system similar to Skeletanks or similar are used to reduce the flow from the site.



Section through typical SEL Skeletank® Installation

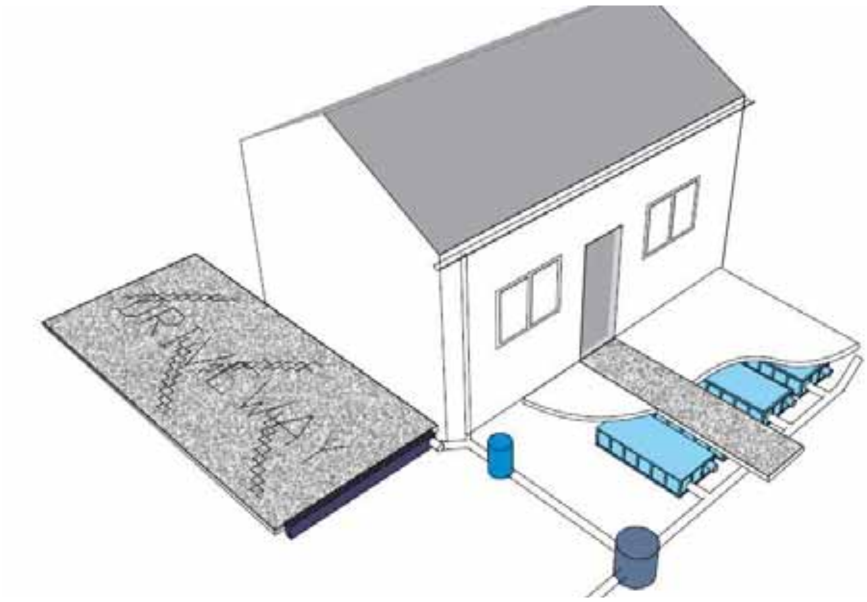


Figure 3-32 Diagrammatic Representation Only

Drainage

Not build over agreements known of.

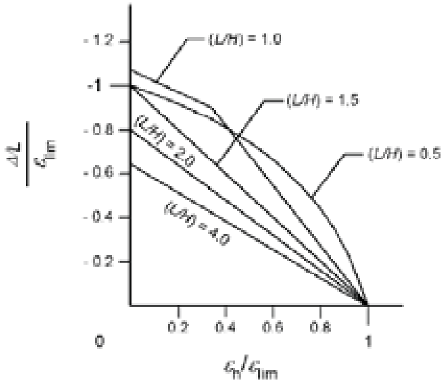
effects on Structure	Flooding. The site is not in an area of high risk flooding.

Trees	
Root Protection Zone	RPA = 1.5 x Crown diameter. A pear tree in the rear garden to be removed.
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.

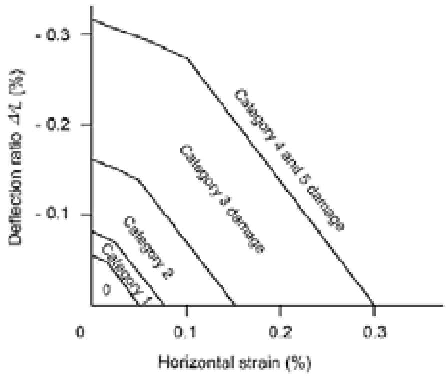
Ground Movement Assessment & Predicted Damage Category

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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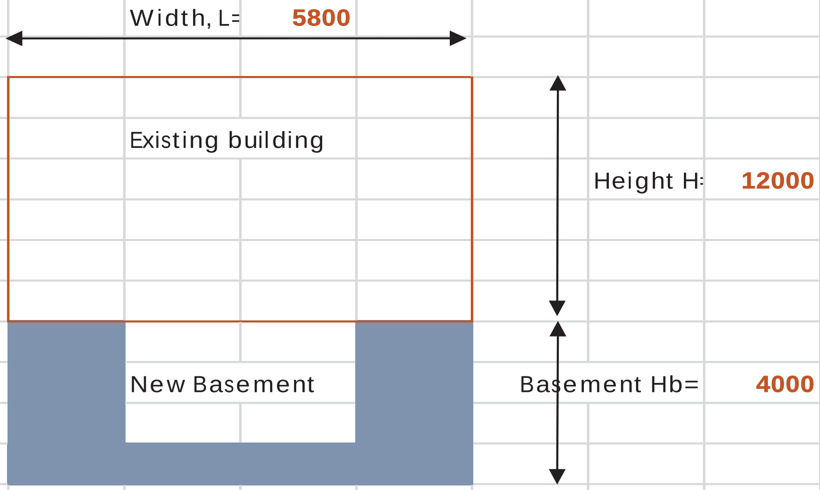
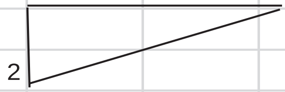


L/H = 0.48333

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

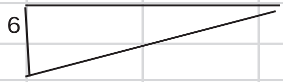
Potential Movement Due to wall installation

Horizontal surface movement =				0.05%			
DeltaH =	0.05%	x	4000	=	2	mm	
Vertical Surface Movement =				0.05%			
Delta V =	0.05%	x	4000	=	2	mm	= 0.33333 mm/m
Distance behind wall wall to neglibible movement							
lh =	4000	x	1.5	=	6000	mm	

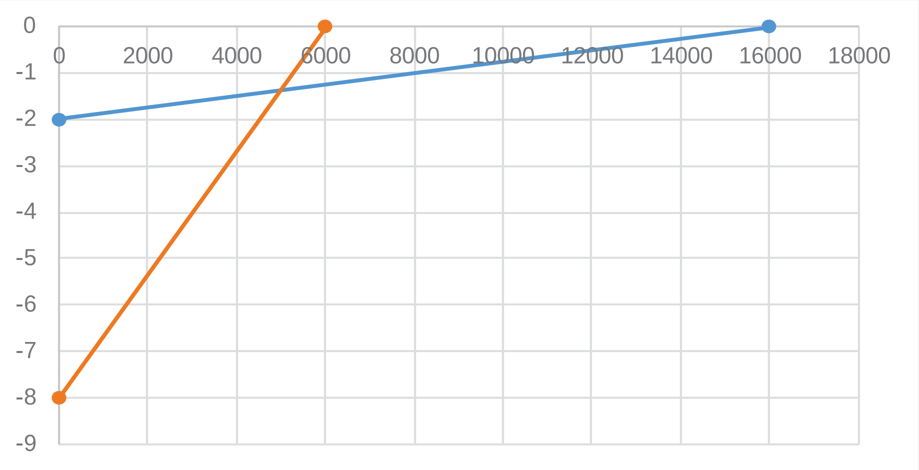


Potential Movement Due to wall Excavation

Horizontal surface movement =				0.15%			
DeltaH =	0.15%	x	4000	=	6	mm	
							= 0.375 mm/m
Vertical Surface Movement =				0.10%			
Delta V =	0.10%	x	4000	=	4	mm	
Distance behind wall wall to neglibible movement							
lh =	4000	x	4	=	16000	mm	



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



Determine Horizontal Movement

delta l =	8	mm	=	0.05%	
	16000	mm			

Table 2.4 CIRIA C580

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

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

	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**

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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 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
Monitoring Conclusion	The level of Monitoring Croft recommend on 54 SHIRLOCK 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	

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- 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 usesK_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 road surface is less than 5m from the front lightwell. Highways loading allow: 10kN/m2 if within 45° of road 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)	See table below
Part A3 Progressive	Number of Storeys 5

Loading Requirements (EC1-1):

	UDL kN/m²	Concentrated LoadskN
Domestic Single Dwellings	1.5	2.0

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

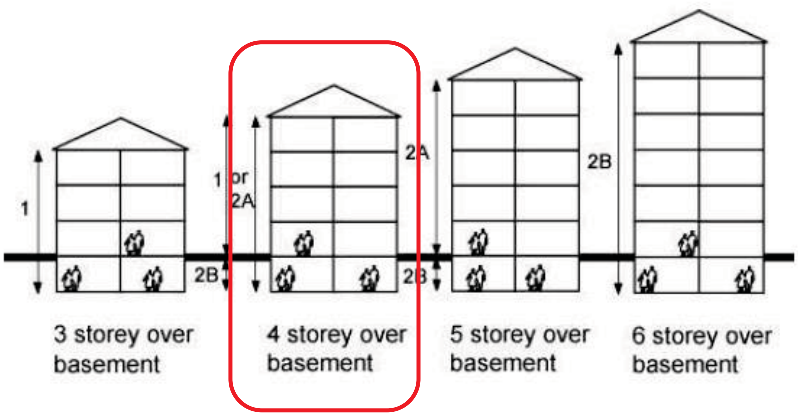
collapse

Is the Building Multi Occupancy? No

Class 2A	5 storey single occupancy houses Hotels not exceeding 4 storeys Flats, apartments and other residential buildings not exceeding 4 storeys Offices not exceeding 4 storeys Industrial buildings not exceeding 3 storeys Retailing premises not exceeding 3 storeys of less than2000m^2 floor area in each storey Single storey educational buildings All buildings not exceeding 2 storeys to which membersof the public are admitted and which contain floor areasnot exceeding 2000m^2 at each storey

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

Initial Building Class	2A
Proposed Building Class	2A
If class has changed material change has occurred	No



Lateral Stability

Exposure and wind loading conditions

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

Stability Design

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

Lateral Actions

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

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

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

Retaining Wall Calculation

Retaining wall Under Party Wall

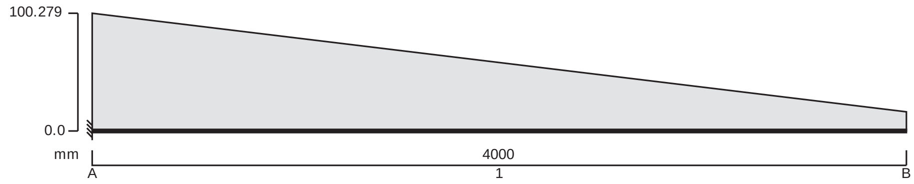
BASEMENT WALLS

RC BEAM ANALYSIS & DESIGN (EN1992-1)

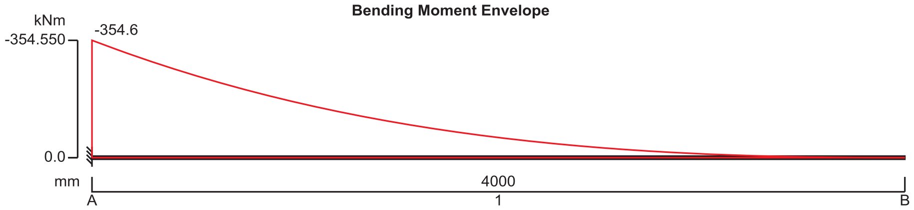
In accordance with UK national annex

TEDDS calculation version 2.1.15

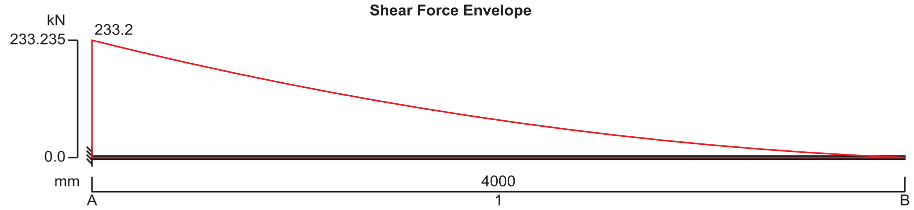
Load Envelope - Combination 1



Bending Moment Envelope



Shear Force Envelope



Support conditions

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

Applied loading

Permanent self weight of beam × 1
Permanent full UDL 2.1 kN/m
Permanent trapezoidal load 16.4 kN/m from 0 mm to 0 mm
Variable trapezoidal load 41.2 kN/m from 0 mm to 0 mm

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

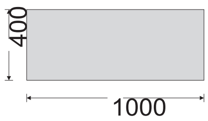
Analysis results

Maximum moment support A	M_{A_max} = **-355** kNm	M_{A_red} = **-355** kNm
Maximum moment span 1 at support	M_{s1_max} = **0** kNm	M_{s1_red} = **0** kNm
Maximum moment support B	M_{B_max} = **0** kNm	M_{B_red} = **0** kNm
Maximum shear support A	V_{A_max} = **233** kN	V_{A_red} = **233** kN
Maximum shear support A span 1	$V_{A\_s1\_max}$ = **233** kN	$V_{A\_s1\_red}$ = **233** kN

Maximum shear support B	V_{B_max} = **0** kN	V_{B_red} = **0** kN
Maximum shear support B span 1	$V_{B\_s1\_max}$ = **0** kN	$V_{B\_s1\_red}$ = **0** kN
Maximum reaction at support A	R_A = **233** kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **81** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **82** kN	
Maximum reaction at support B	R_B = **0** kN	
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **0** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **0** kN	

Rectangular section details

Section width	b = **1000** mm
Section depth	h = **400** mm



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

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

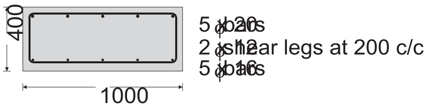
Reinforcement details

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

Nominal cover to reinforcement

Nominal cover to top reinforcement	c_{nom_t} = **35** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **50** mm
Nominal cover to side reinforcement	c_{nom_s} = **35** mm

Mid span 1



Rectangular section in flexure (Section 6.1) - Negative span moment

Design bending moment	$M = abs(M_{s1\_neg})$ = **168** kNm
Depth to tension reinforcement	$d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2$ = **343** mm
Percentage redistribution	m_{rs1} = **0** %
Redistribution ratio	$\delta = \min(1 - m_{rs1}, 1)$ = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.051**
	$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **326** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **43** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **1186** mm^2
Tension reinforcement provided	5 × 20φ bars
Area of tension reinforcement provided	$A_{s,prov}$ = **1571** mm^2
Minimum area of reinforcement (exp.9.1N)	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **493** mm^2
Maximum area of reinforcement (cl.9.2.1.1(3))	$A_{s,max} = 0.04 \times b \times h$ = **16000** 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 × 12φ legs at 200 c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **1131** mm^2/m
Minimum area of shear reinforcement (exp.9.5N)	$A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk}$ = **847** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N)	$s_{vl,max} = 0.75 \times d$ = **257** mm

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

Design shear resistance (assuming cot(θ) is 2.5)	$V_{prov} = 2.5 \times A_{sv,prov} \times z \times f_{yd}$ = **400.6** kN

Shear links provided valid between 0 mm and 4000 mm with tension reinforcement of 1571 mm^2

Crack control (Section 7.3)

Maximum crack width	w_k = **0.3** mm
Design value modulus of elasticity reinf (3.2.7(4))	E_s = **200000** N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm}$ = **2.8** N/mm^2
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300\ mm - \min(h, b)) \times 0.35 / 500\ mm, 0.65), 1)$ = **0.93**
Actual tension bar spacing	$s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{top}) / (N_{top} - 1)$ = **222** mm
Maximum stress permitted (Table 7.3N)	σ_s = **223** N/mm^2
Concrete to steel modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm}$ = **6.19**
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **197** mm
Area of concrete in the tensile zone	$A_{ct} = b \times y$ = **197144** mm^2

Minimum area of reinforcement required (exp.7.1) $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **911** mm^2

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

Quasi-permanent value of variable action ψ_2 = **0.30**

Quasi-permanent limit state moment $M_{QP} = abs(M_{s1\_c21}) + \psi_2 \times abs(M_{s1\_c22})$ = **0** kNm

Permanent load ratio $R_{PL} = M_{QP} / M$ = **0.00**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **0** N/mm^2

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

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

Minimum bar spacing

Minimum bottom bar spacing $s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1)$ = **222** mm

Minimum allowable bottom bar spacing $s_{bar\_bot,min} = max(\phi_{bot}, h_{agg} + 5\text{ mm}, 20\text{ mm}) + \phi_{bot}$ = **41** mm

Minimum top bar spacing $s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1)$ = **221** mm

Minimum allowable top bar spacing $s_{bar\_top,min} = max(\phi_{top}, h_{agg} + 5\text{ mm}, 20\text{ mm}) + \phi_{top}$ = **45** mm

PASS - Actual bar spacing exceeds minimum allowable

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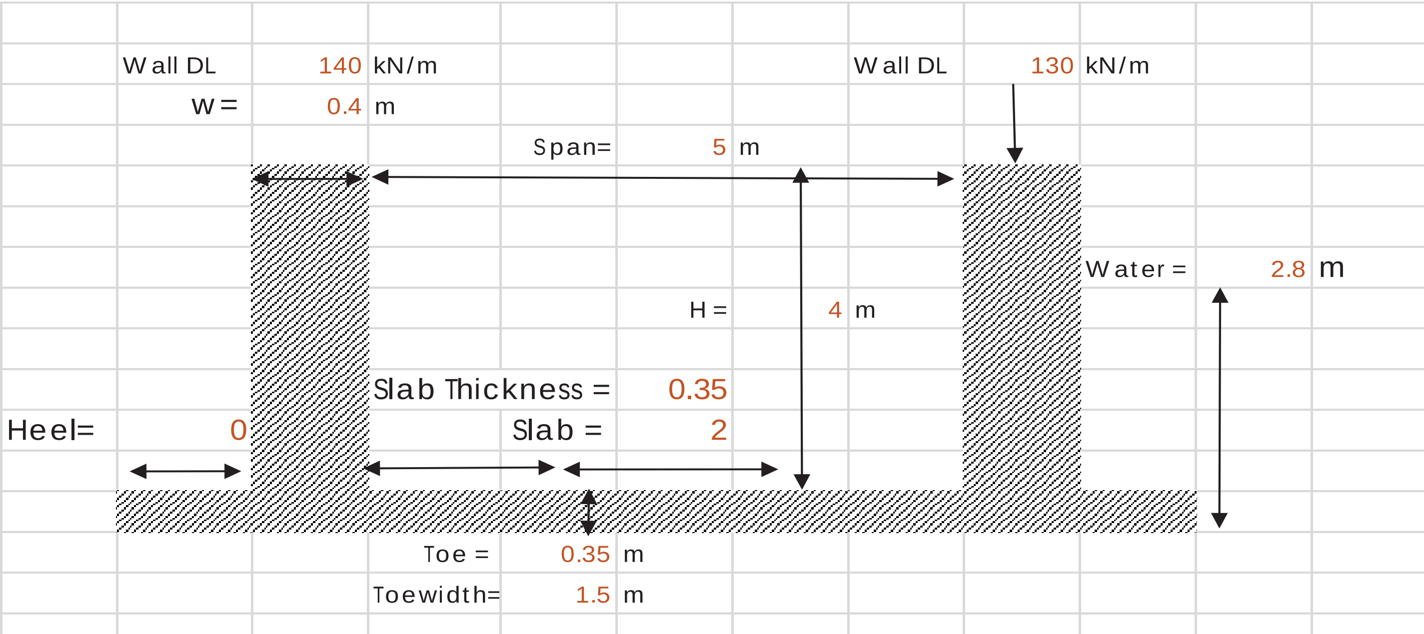
Excel Sheet Uplift Calcs



Uplift Calc

Total Dead Load =	Slab=	17.5	kN/m		
	Toe and heel =	33.25	kN/m		
	Wall =	80			
	Soil=(	0		) x 2=	0
	Total Dead load =	400.75	kN/m		
Total Uplift Force=		162.4	kN/m	f.o.s.=	**2.46767 No Global Uplift**

Slab Uplift

Slab =	8.75	kN/m	Uplift =	28
Service Moment =	-60.156	kNm/m		
Factored Design moment=	-71.641	kNm/m		
Factored Design shear =	-57.313	kN/m		

Global Heave

Weight of building =	400.75	kN/m			
Weight of soil removed =	302.4				
% change	-33%		place	-33%	of Slab area as heave protection
Wide of Heave protection =	-1.8863	m	place	-1.89	m of Slab area as heave protection

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Wall DL 140 kN/m
w = 0.4 m
Sur l = 5.5

ϕ 26
δ 20
Span= 5 m

Wall DL 130 kN/m
Sur2 = 5.5

H 2= 4 m
Water = 3 m
H 1= 2 m

Slab Thickness = 0.35
Heel= 0
Slab = 2

Toe = 0.35 m
Toewidth= 1.5 m

		57.2958
	kp =	2.56107
	ka =	0.39046

Thrust from left =	12.10431	Thrust from Right	12.1043
Resitance from Left =	102.4428	Resitance from Right =	102.443

Equilibrioum check Kp from Right Adequate
Kp from left Adequate

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

Temporary works design

54 Shirlock 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 Shirlock 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 cantilever retaining walls in underpin sections at the new basement level.

2.0

1.6. The cantilever pins are designed to be inherently stable during the construction stage <u>without</u> temporary propping to the head. The base benefits from propping, this is provided in the final condition by the ground slab. In the temporary condition the edge of the slab is buttressed against the soil in the middle of the property, also the skin friction between the concrete base and the soil provides further resistance. The central slab is to be poured in a maximum of a 1/3 of the floor area.

1.7. A soil investigation has been undertaken. The soil conditions are made ground on silty clay. Refer to Soil Investigation report for details.

1.8. The bearing pressures have been limited to 40kN/m^2. .

1.9. No water table is encountered during soil investigation (SI). See SI for details.

3.0

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

4.0

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 sit mixing to correct specification, contractor must undertake tool box talks with staff to ensure site quality is maintained.

2. Enabling Works

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

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

2.3. Provide protection to public where conveyor extends over footpath. Depending on the requirements of the local authority, construct a plywood bulkhead onto the pavement. Hoarding to have a plywood roof covering, night-lights and safety notices.
5.0
2.4. Dewater: No significant dewatering is expected. Localised removal of water may be required to deal with rain from perched water or localised water. This is to be dealt with by localised pumping. Typically achieved by a small sump pump in bucket.

2.5. On commencement of construction the contractor will determine the foundation type, width and depth. Any discrepancies will be reported to the structural engineer in order that the detailed design may be modified as necessary.

3. Basement Sequencing

3.1. Excavate Light well to front of property down to 600mm below external ground level

3.2. Excavate first front corner of light well. (Follow methodology in section 4)

3.3. Excavate second front corner of light well. (Follow methodology in section 4)
6.0
3.4. Place cantilevered walls 1, 2 and 3 noted on plans. (Cantilevered walls to be placed in accordance with section 4.).

3.5. Needle the bay/front wall above.

3.6. Insert steel over and sit on cantilevered walls.
7.0
3.6.1. Beams over 6m to be jacked on site to reduce deflections of floors.

3.6.2. Dry pack to steelwork. Ensure a minimum of 24 hours from casting cantilevered walls to dry packing.

3.7. Continue excavating section pins to form front light well. (Follow methodology in section 4)

3.8. Place cantilevered retaining wall to the left side of front opening. After 48 hours place cantilevered retaining wall to the right side of front opening.
8.0
3.9. Excavate out first 1.2m around front opening prop floor and erect conveyor.
9.0
3.10. Continue cantilevered wall formation around perimeter of basement following the numbering sequence on the drawings.
10.0
3.10.1. Excavation for the next numbered sequential sections of underpinning shall not commence until at least 8 hours after drypacking of previous works. Excavation of adjacent pin to not commence until 48 hours after drypacking. (24hours possible due to inclusion of Conbextra 100 cement accelerator to dry pack mix).

3.10.2. Floor over to be propped as excavations progress. Steelwork to support Floor to be inserted as works progress.

3.11. Cast base to internal wall. Construct wall to provide support to floor and steels as works progress.

3.12. Excavate a maximum of a 1/3 of the middle section of basement floor. Place reinforcement to central section of ground bearing slab and pour concrete. Excavate next third and cast slab. Excavate and cast final third and cast.

3.13. Provide structure to ground floor and water proofing to retaining walls as required.

4. Underpinning and Cantilevered Walls

4.1. Prior to installation of new structural beams in the superstructure, the contractor may undertake the local exploration of specific areas in the superstructure. This will confirm the exact form and location of the temporary works that are required. The permanent structural work can then be undertaken whilst ensuring that the full integrity of the structure above is maintained.

4.2. Provide propping to floor where necessary.

4.3. Excavate first section of retaining wall (no more than 1200mm wide). Where excavation is greater than 1.2m deep provide temporary propping to sides of excavation to prevent earth collapse (Health and Safety). A 1200mm width wall has a lower risk of collapse to the heel face.

11.0

4.4. Excavation of pins deeper than 3m comes under confirmed working space and operators must wear harness and there must be a winch above the excavation.

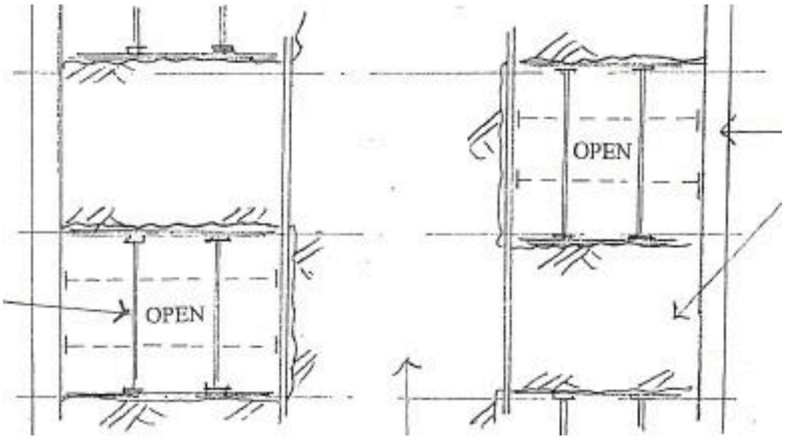


Figure 33 – Schematic Plan view of Soil Propping

Figure 34 Propping

Figure 35 Excavation of Pin

Figure 36 Completed Wall

12.0

4.5. Backpropping of rear face. Rear face to be propped in the temporary conditions with a minimum of 2 Trench sheets. Trench sheets are to extend over entire height of excavation. Trench sheets can be placed in short sections are the excavation progresses.

4.5.1. If the ground is stable, trench sheets can be removed as the wall reinforcement is placed and the shuttering is constructed.

4.5.2. Where soft spots are encountered leave in trench sheets or alternatively back prop with Precast lintels or trench sheeting. (If the soil support to the ends of the lintels is insufficient then brace the ends of the PC lintels with 150x150 C24 Timbers and prop with Acros diagonally back to the floor.)

4.5.3. Where voids are present behind the lintels or trench sheeting. Grout voids behind sacrificial propping; Grout to be 3:1 sand cement packed into voids.

4.5.4. Prior to casting place layer of DPM between trench sheeting (or PC lintels) and new concrete. The lintels are to be cut into the soil by 150mm either side of the pin. A site stock of a minimum of 10 lintels to be present for to prevent delays due to ordering.

4.6. If cut face is not straight, or sacrificial boards noted have been used, place a 15mm cement particle board between sacrificial sheets and or soil prior to casting. Cement particle board is to line up with the adjacent owners face of wall. The method adopted to prevent localised collapse of the soil is to install these progressively one at a time. Cement particle board must be used to in any condition where overspill onto the adjacent owners land is possible.

4.7. Underpinns can be completed in Segmental lifts (eg top section of wall followed by bottom section of wall).

13.0

Crofts recommendation is that walls with high vertical loads or susceptible to settlement, and all party walls, should be completed as first pin top first pin bottom, next pin top next pin bottom. **We do not recommend** for such conditions that all the top sections for every pin followed by all the lower pins are completed; such a sequencing can result in the existing wall being left on a narrower section than the original footing for too long resulting in settlement.

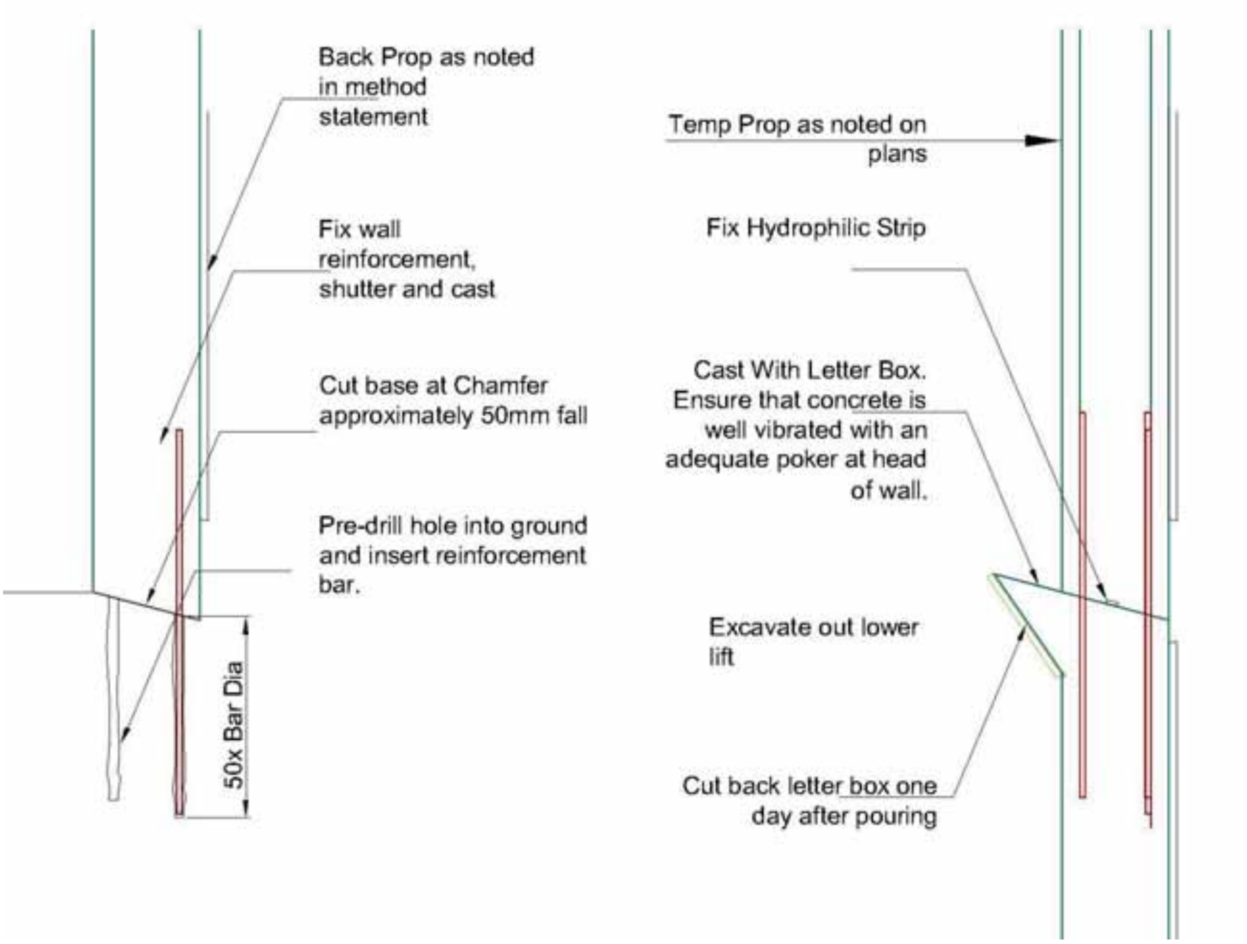
14.0

4.7.1. Place reinforcement for retaining wall segmental lift

15.0

4.7.1.1. At lift sections reinforcement needs to be driven in. This is to be completed by pre drilling holes and inserting the reinforcement into the predrilled hole.

4.7.1.2. Underside of the wall to be cast with chamfer to allow concrete for lower lift to be cast and no packing to be required.



16.0

4.8. Excavate base. Mass concrete heels to be excavated. If soil over unstable prop top with PC lintel and sacrificial prop.

17.0

4.9. Visually inspect the footings and provide propping to local brickwork, if necessary sacrificial acrow, or pit props, to be sacrificial and cast into the retaining wall.

4.10. Clear underside of existing footing.

4.11. Local authority inspection to be carried for approval of excavation base.

4.12. Place blinding.

4.13. Place reinforcement for retaining wall base, heel & toe. Site supervisor to inspect and sign off works for proceeding to next stage.

18.0

4.14. Cast base. (on short stems it is possible to cast base and wall at same time). It is essential that pokers/vibrators are used.

19.0

4.15. Take 2 cubes of concrete and store for testing. Test one at 28 days, if result is low test second cube. Provide results to client and design team on request or if values are below those required.

20.0

4.16. Horizontal temporary prop to base of wall to be inserted. Alternatively cast base against soil.

21.0

4.17. Place reinforcement for retaining wall stem. Site supervisor to inspect and sign off works for proceeding to next stage.

4.18. Drive H16 Bars UBars into soil along centre line of stem to act as shear ties to adjacent wall.

4.19. Place shuttering & pour concrete for retaining wall. Stop a minimum of 75mm from the underside of existing footing.). It is essential that pokers/vibrators are used, hitting shutters is not considered adequate.

4.20. 24 hours after pouring the concrete pin the gap shall be filled using a dry pack mortar. Ram in drypack between retaining wall and existing masonry.

22.0

4.21. After 24 hours the temporary wall shutters are removed.

4.22. Trim back existing masonry corbel and concrete on internal face.

4.23. Site supervisor to inspect and sign off for proceeding to the next stage. A record will be kept of the sequence of construction, which will be in strict accordance with recognised industry procedures.

5. Floor Support

Timber Floor

5.1. The timber floor will remain in situ, and be supported by a series of steel beams that will support the floors, to provide the open areas in the basement.

5.2. Position 100 x 100mm temporary timber beam lightly packed to underside of joists either side of existing sleeper wall and support with vertical acrow props @ 750 centres. Remove sleeper walls and insert steel beam as a replacement. Beams to bear onto concrete padstones built into the masonry walls (refer to Structural Engineer's details for padstone & beam sizes)

5.3. Dismantle props and remove timber plates on completion of installation of permanent steel beams.

Concrete Ground bearing slabs

5.4. The support of the existing concrete floor will be undertaken in conjunction with the underpinning process. Two opposite pins are constructed and allowed to cure as described elsewhere.

5.5. Locally prop concrete floors with Acros at 2m centres with timbers between. If the underside is found be in poor condition then temporary boarding and props are to be introduced.

5.6. Insert Steelwork and dry pack to underside of floor

23.0

5.7. Between steelwork place 215wide x 65dp PC lintels at a maximum spacing of 600mm
24.0
5.8. If necessary Brick up to the 50mm below underside of floor
25.0
5.9. Dry pack between lintel/brickwork to underside of slab.
26.0
5.10. Remove props

5.11. This process is to continue one pin width at a time.

6. Supporting existing walls above basement excavation

6.1. Where steel beams need to be installed directly under load bearing walls, temporary works will be required to enable this work. Support comprises the installation of steel needle beams at high level, supported on vertical props, to enable safe removal of brickwork below, and installation of the new beams and columns.

6.1.1. The condition of the brickworks must be inspected by the foreman to determine its condition and to assess the centres of needles. The foreman must inspect upstairs to consider where loads are greatest. Point loads and between windows should be given greater consideration.

6.1.2. Needles are to be spaced to prevent the brickwork above “saw-toothing”. Where brickwork is good needles must be placed at a maximum of 1100mmcenters. Lighter needles or strong boys should be placed at tighter centres under door thresholds

6.2. Props are to be placed on Sleepers of firm ground or if necessary temporary footings will be cast.

6.3. Once the props are fully tightened, the brickwork will be broken out carefully by hand. All necessary platforms and crash decks will be provided during this operation.

6.4. Decking and support platforms to enable handling of steel beams and columns will be provided as required.

6.5. Once full structural bearing is provided via beams and columns down to the new basement floor level. The temporary works will be redundant and can be safely removed.

6.6. Any voids between the top of the permanent steel beams and the underside of the existing walls will be packed out as necessary. Voids will be drypacked with a 1:3 (cement: sharp sand) drypack layer, between the top of the steel and underside of brickwork above.

6.7. Any voids in the brickwork left after removal of needle beams can at this point be repaired by bricking up and/or drypacking, to ensure continuity of the structural fabric.

7. Approval

7.1. Building control officer/approved inspector to inspect pin bases and reinforcement prior to casting concrete.

7.2. Contractor to keep list of dates pins inspected & cast

7.3. One month after work completed the contractor is to contact adjacent party wall surveyor to attend site and complete final condition survey and to sign off works.

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8. 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

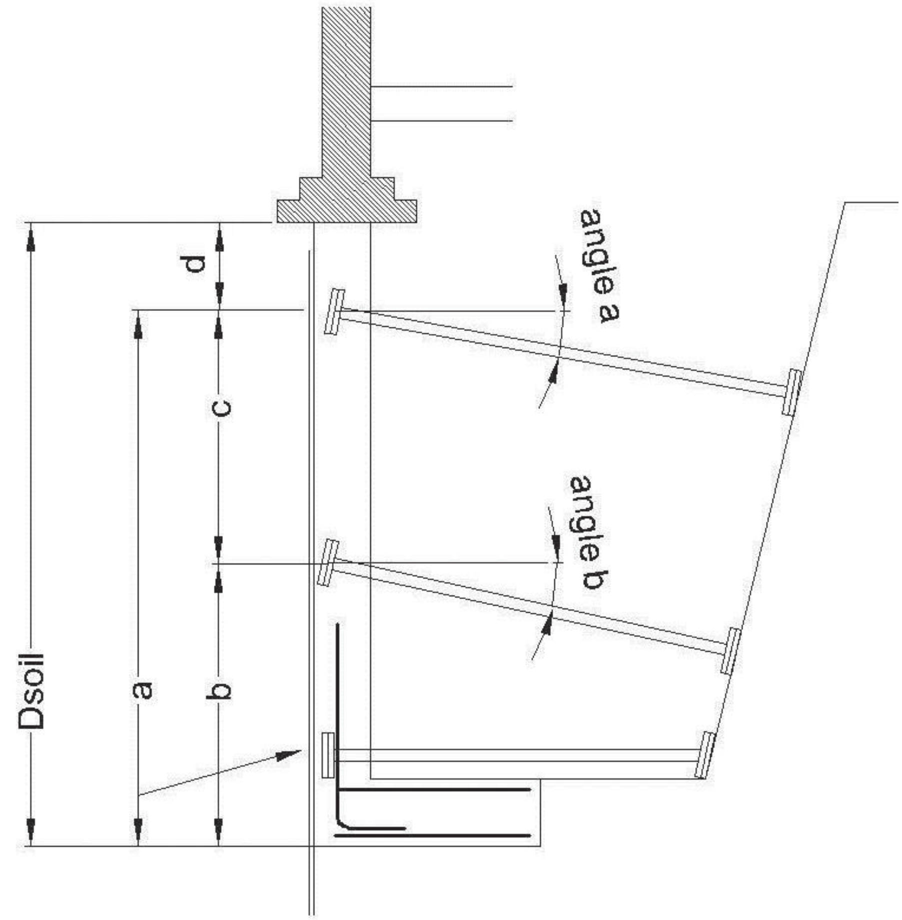
STANDARD LAP

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

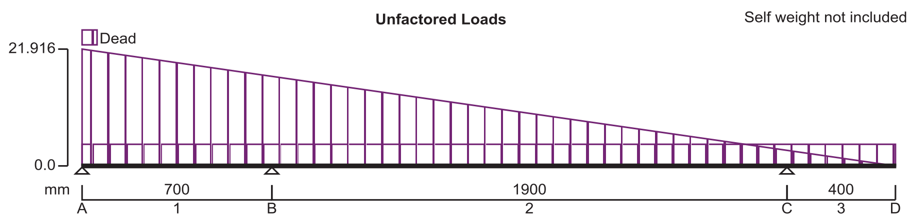
Technical Information

Effective width per sheet (mm)	330
Thickness (mm)	3.4
Depth (mm)	35
Weight per linear metre (kg/m)	10.8
Weight per m^2 (kg)	32.9
Section modulus per metre width (cm^3)	48.3
Section modulus per sheet (cm^3)	15.9
I value per metre width (cm^4)	81.7
I value per sheet (cm^4)	26.9
Total rolled metres per tonne	92.1

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



Length a	a = **2.600** m
Length b bottom	b = **0.700** m
Length c Middle	c = a – b = **1.900**m
Length d top	d = Dsoil – a = **0.400**m



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **3**

Material Properties:

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

Support Conditions:

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

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

Span Definitions:

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

LOADING DETAILS

Beam Loads:

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

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

LOAD COMBINATIONS

Load combination 1

Span 1 1×Dead

Span 2 1×Dead

Span 3 1×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Unfactored support reactions

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

Support Reactions - Combination Summary

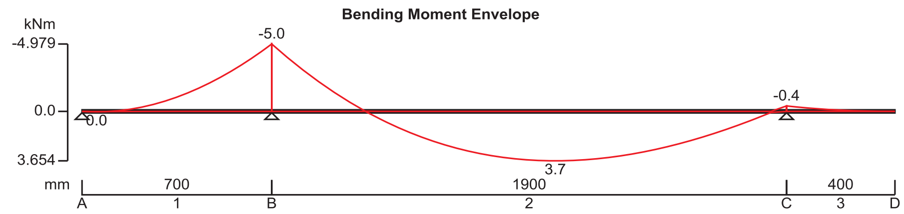
Support A	Max react = **-1.4** kN	Min react = **-1.4** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-32.8** kN	Min react = **-32.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-10.8** kN	Min react = **-10.8** kN	Max mom = **0.0** kNm	Min mom = **0.0**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

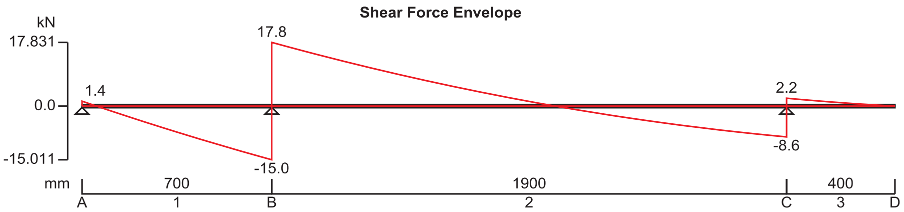
Beam Max/Min results - Combination Summary

Maximum shear = **17.8** kN — Minimum shearF_{min} = **-15.0** kN

Maximum moment = **3.7** kNm — Minimum moment = **-5.0** kNm

Maximum deflection = **21.0** mm — Minimum deflection = **-14.3** mm





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Number of sheets Nos = 2

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

SRU 4.0

Safe working loads for Acrow Props — loads given in kN

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

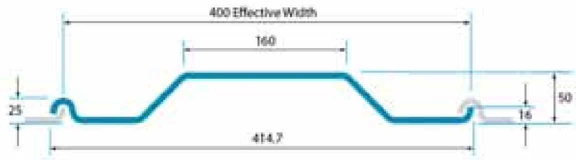
Shear V = (14.6kN + 13.4kN) /2 = 14.000kN

Any Acro Prop is accetpable

KD4 SHEETS

KD4

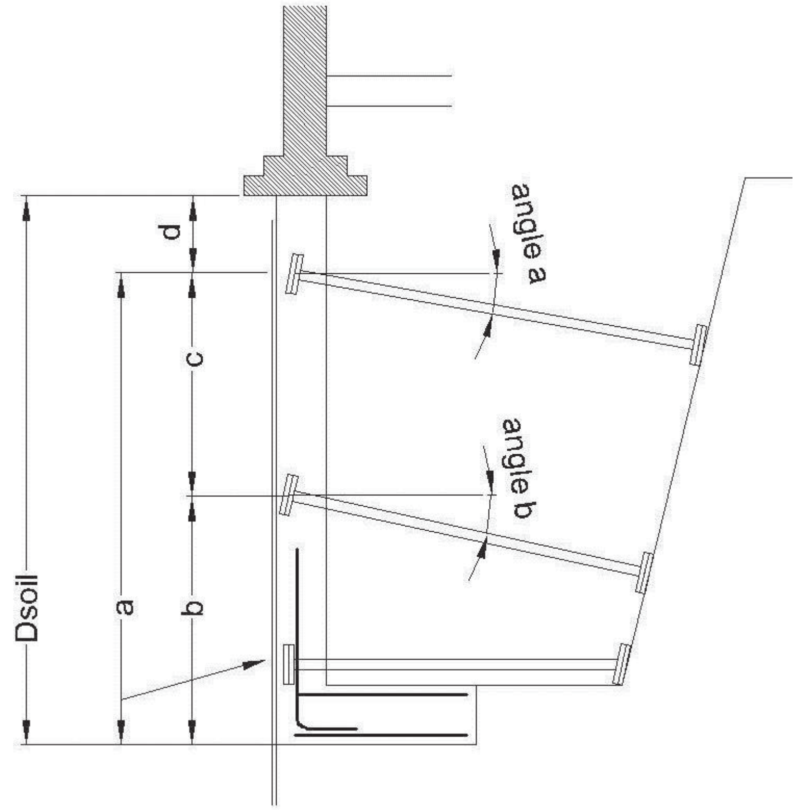
The overlapping trench sheeting profile is a heavier version of the Standard Lap, with a wider gauge and width coverage, designed in large for construction work.



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 m² (kg)	55.2
Section modulus per metre width (cm³)	101
Section modulus per sheet (cm³)	40.34
I value per metre width (cm⁴)	250
I value per sheet (cm⁴)	101
Total rolled metres per tonne	45.659

$S_{xx} = 48.3cm^3$
$p_y = 275N/mm^2$
$I_{xx} = 26.9cm^4$
$A = (1m^2 * 55.2kg/m^2) / (400mm * 7750kg/m^3) = \mathbf{17806.452}mm^2$

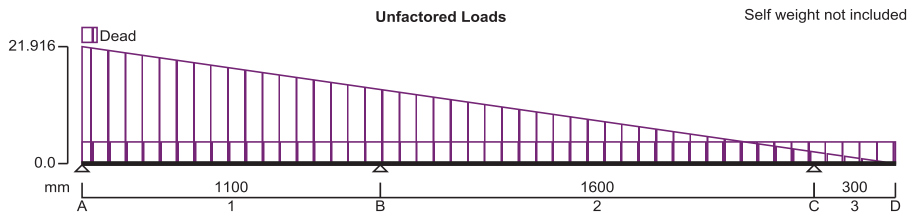


Length a a = **2.700** m

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.×10^3** mm^4

LOADING DETAILS

Beam Loads:

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

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

LOAD COMBINATIONS

Load combination 1

Span 1 1×Dead

Span 2 1×Dead

Span 3 1×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Support Reactions - Combination Summary

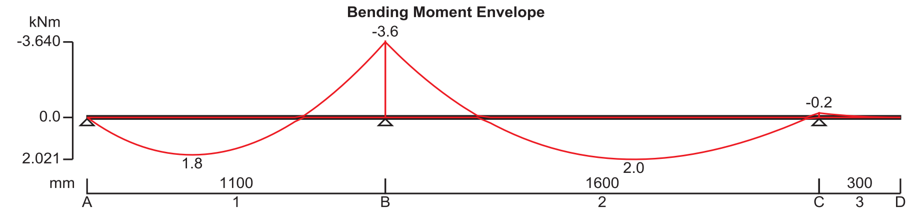
Support A	Max react = **-9.5** kN	Min react = **-9.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-28.0** kN	Min react = **-28.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-7.5** kN	Min react = **-7.5** kN	Max mom = **0.0** kNm	Min mom = **0.0**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

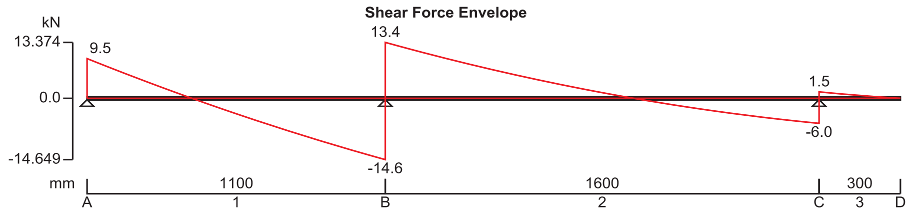
Beam Max/Min results - Combination Summary

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

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

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





Number of sheets Nos = 2

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

CROFT STRUCTURAL ENGINEERS

SRU4·0

Safe working loads for Acrow Props — loads given in kN

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

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

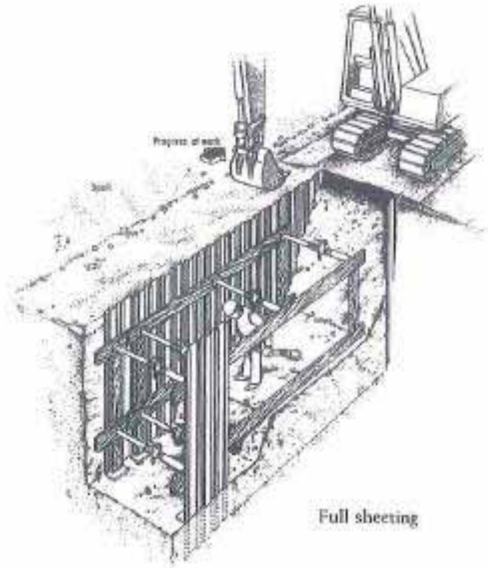
Any Acro Prop is accetpable

Sheeting requirements

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

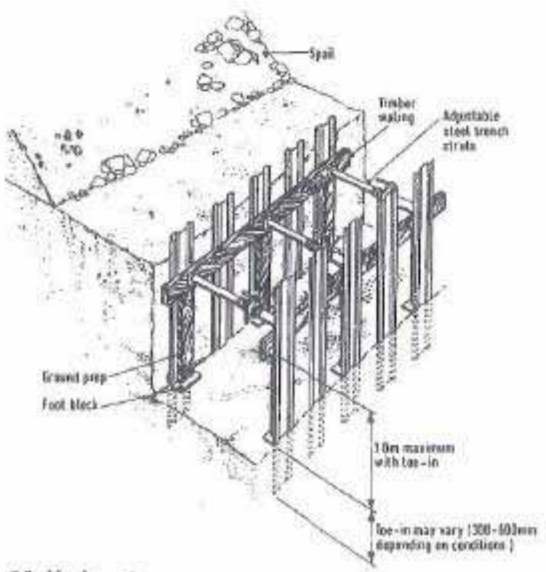
CROFT STRUCTURAL ENGINEERS

Sheeting requirements



Full sheeting

Sheeting requirements



Half sheeting
shown for 1.5 m deep trench

11/04/2012

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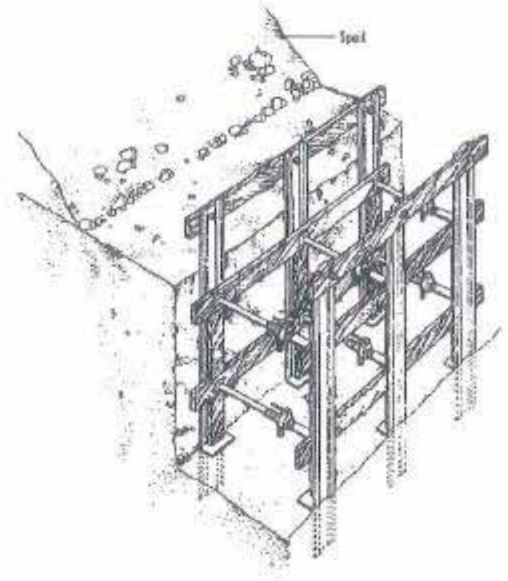
Sheeting requirements



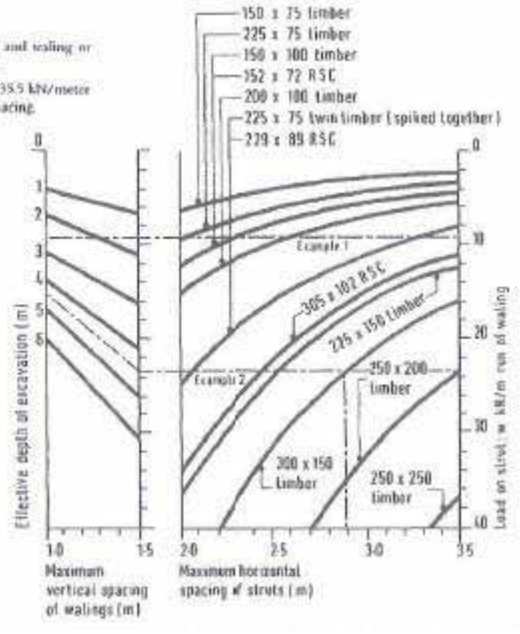
11/0 Quarter sheeting

Design to CIRIA 97

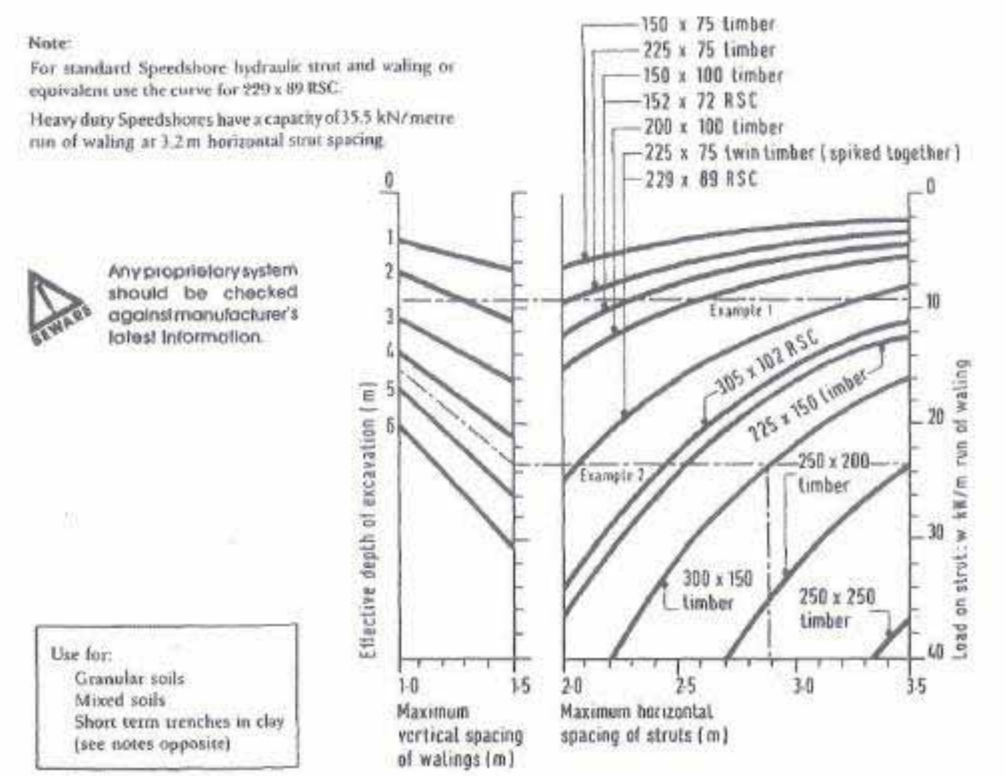
Note:
For standard Speedshore hydraulic strut and waling or equivalent use the curve for 229 x 89 RSC.
Heavy duty Speedshores have a capacity of 35.5 kN/meter run of waling at 3.2 m horizontal strut spacing.

BEWARE

Any proprietary system should be checked against manufacturer's latest information.



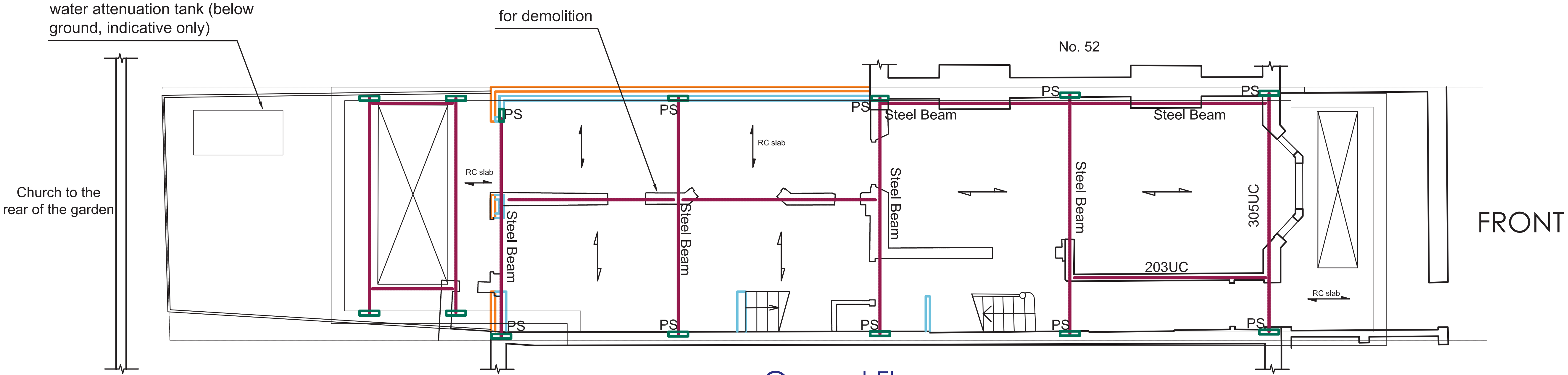
Use for:
Granular soils
Mixed soils
Short term trenches in clay (see notes opposite)


Note:
For standard Speedshore hydraulic strut and waling or equivalent use the curve for 229 x 89 RSC.
Heavy duty Speedshores have a capacity of 35.5 kN/metre run of waling at 3.2 m horizontal strut spacing.
Any proprietary system should be checked against manufacturer's latest information.
BEWARE
Use for:
Granular soils
Mixed soils
Short term trenches in clay
(see notes opposite)
150 x 75 timber
225 x 75 timber
150 x 100 timber
152 x 72 RSC
200 x 100 timber
225 x 75 twin timber (spiked together)
229 x 89 RSC
305 x 102 RSC
225 x 150 timber
250 x 200 timber
300 x 150 timber
250 x 250 timber
Example 1
Example 2
Effective depth of excavation (m)
Load on strut: w kN/m run of waling
Maximum vertical spacing of walings (m)
Maximum horizontal spacing of struts (m)
0
1
2
3
4
5
6
1·0
1·5
2·0
2·5
3·0
3·5
10
20
30
40

Appendix B : Structural Drawings

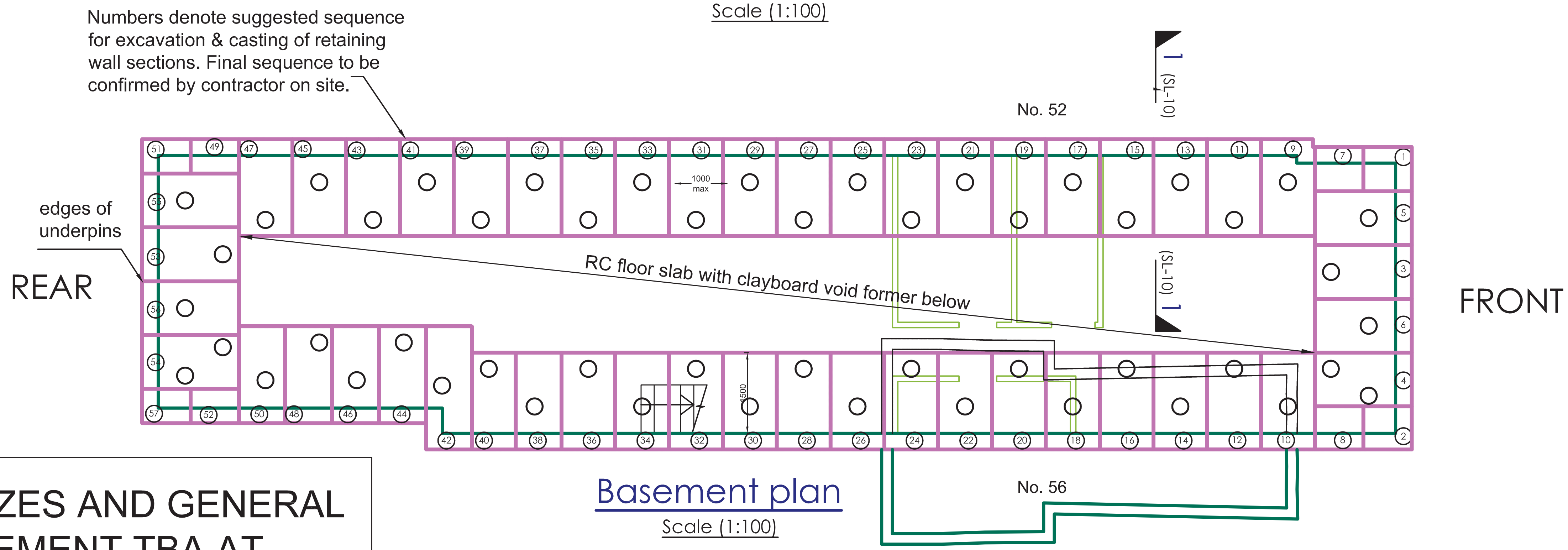
1:100 Basement Plan on A3Showing Neighbouring basements if present
1:100 Ground Floor plan on A3 Showing Neighbouring property
1:20 Section on A3 Including section through Neighbouring Footings

The general construction is load bearing external walls with timber/masonry internal walls. Timber floors are on the ground and upper floors.
Structural steel supporting timber floor joists is assumed to exist within the building



Ground Floor plan

Scale (1:100)



Basement plan

Scale (1:100)

STEEL SIZES AND GENERAL ARRANGEMENT TBA AT DETAILED DESIGN STAGE

NOT FOR CONSTRUCTION

Rev	Date	Amendments
-	10/07/15	First issue for comment

Job No.s 150527	Client: Alex Kay
Dwg Nos SL-10	Project: 54 Shirlock Road
Date June15	Title : Basement and Ground Floor plan
Drawn EP / Chk'd CT	
Scale As shown @ A3 / Rev -	

Croft Structural Engineers
Clockshop Mews, r/o 60 Saxon Rd, London, SE25 5EH.
020 8684 4744
www.croftse.co.uk

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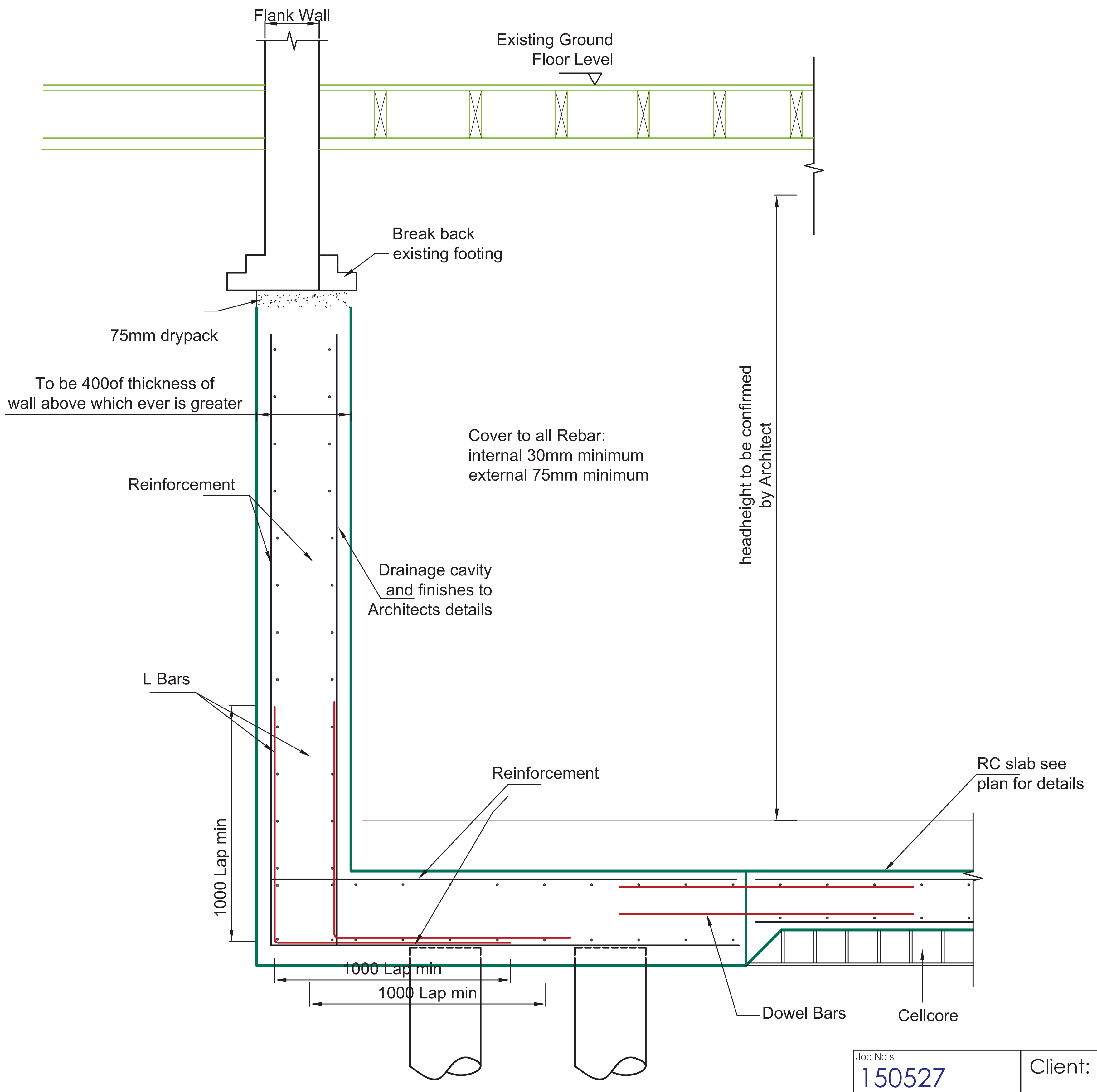

Typical Section
Scale 1:20

Rev	Date	Amendments
-	10/07/15	First issue for comment

Job No.s: 150527	Client: Alex Kay
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