Job Number: 150912

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:

6 Stukeley Street
London
WC2B 5LQ

Client:

Derek Savage
c/o Konrad Romaniuk
Milan Babic Architects Ltd,
151B Bermondsey Street,
London,
SE1 3UW

Structural Design Reviewed by Chris Tomlin MEng CEng MIStructE	**Above Ground Drainage Reviewed by** Phil Henry BEng MEng MICE
Hydrology Report Julian Maund BSc PhD CEng MiMMM CGeol FGS Separate report	**Geology Report** Julian Maund BSc PhD CEng MiMMM CGeol FGS Separate report
Report compiled by Eleni Pappa MSc BEng	

Revision	Date	Comment
-	28-04-16	Draft Issue
1	06-06-17	Final Issue for planning

CROFT STRUCTURAL ENGINEERS

Contents

CROFT STRUCTURAL ENGINEERS

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

6 Stukeley Street is a 1 storey, 3 bedroom property and it is part of an amalgamation of properties that have evolved on the site since mid 1800's. No. 6 is a terraced property of original Victorian construction.

Proposed Works

The proposed works require the construction of a new basement with a new superstructure on top. The existing building will be demolished following construction of the new basement to allow for better temporary works details.

The new superstructure works will consist of a new first and second floor levels with new mansard roof.

Figure 1: Front of 6 Stukeley Street

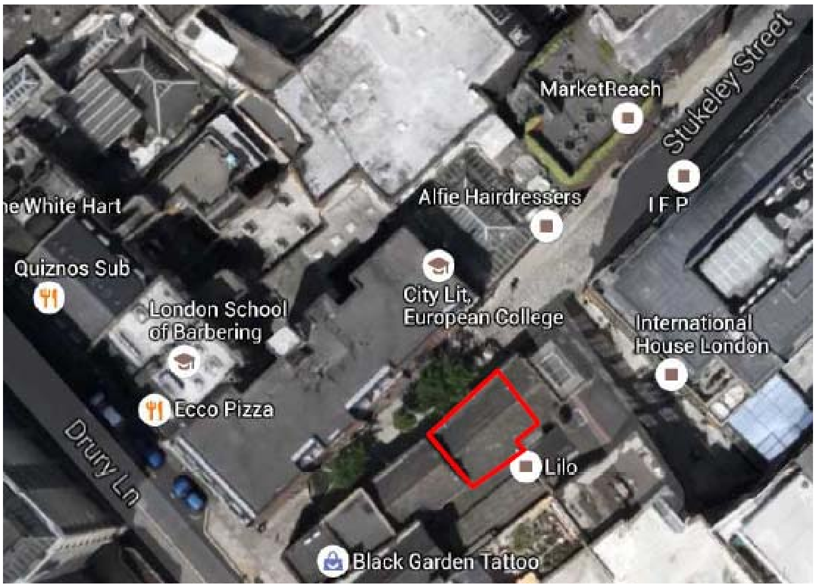


Figure 2: Plan location of 6 Stukeley Street

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 6 Stukeley Street is:

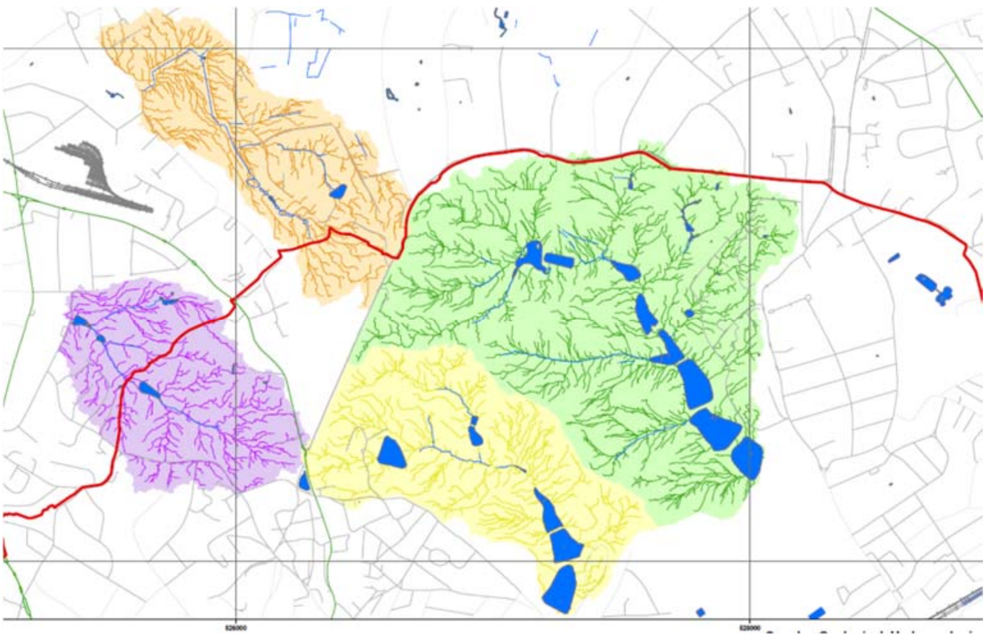
1. Excavate front to allow for start of underpinning
2. Safely and securely support the existing building above
3. Slowly work from the front to the rear inserting narrow cantilevered retaining walls sequentially using well developed and understood underpinning methods.
4. Form side lightwell with cantilevered retaining walls
5. Prop across the width of the basement, excavate central soil "dumpling"
6. Place reinforcement and 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, Hydrogeology, 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. A ground investigation was also carried out. The most relevant findings are: • Fill material above lynch Hill Gravels overlying London Clay • Ground water was encountered below the level of the basement
Stage 4 – Impact assessment	**Land stability** The Geologist has concluded that the basement will not make the area unstable. See below the summary from the Hydrogeology and Land stability Basement Impact Assessment **6 Stukeley Street Basement Impact Assessment Hydrogeology and Land Stability – Summary** A basement impact assessment (BIA) has been undertaken for hydrogeology and land stability in general accordance with CPG4 (2105) for the site at 6 Stukeley Street, WC2B 5LQ in the London Borough of Camden. A basement is proposed to a formation depth of approximately 3.70 m below ground level within the existing building foot print. The existing building was constructed prior to 1873.The BIA report considered relevant information from existing sources included in the 'Guidance for subterranean development' produced for the London Borough of Camden' (November 2010) and a Groundsure Enviro / Geoinsight Report with historical maps and BGS records. A ground investigation at the site was undertaken by Ground and Water Ltd in November 2015 which comprised a borehole to 8 m depth below ground level, and two hand dug trial pits to expose existing foundations. The ground investigation confirmed the ground conditions as a predominantly loose granular made ground to a depth of 3.2 m which overlies the dense to very dense Lynch Hill Gravel Member to a depth of 5.0 m which in turn overlies the stiff to very stiff London Clay Formation. Groundwater was recorded at 5.60 m below ground level. An assessment of hydrogeology has shown that the site is located on a secondary A aquifer', which has been confirmed as 'unproductive strata'. It is not anticipated that the development will have any significant impact on groundwater, which is currently 1.33 m below the basement formation. As a precaution it is recommended that groundwater monitoring is undertaken to confirm if seasonal fluctuations impact on the basement construction, so that appropriate mitigation measures can be designed and implemented.

	An assessment of land stability has been made from the excavation and construction of the basement. It has been calculated that heave is not expected to exceed 15 mm resulting from the excavation. A ground movement assessment has been undertaken to evaluate the impact of the basement excavation and construction on adjacent properties 4, 8 and 10 Stukeley Street. The assessment has shown that the impact will be Category 0 to 1 or negligible to very slight. **Hydrogeology** Refer to Hydrogeologist report **Drainage & Surface Water Flow** Refer to Geologist report

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 Julian Mound, BSc, PhD, FGS, CGeol, MIMMM,CEng, 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 3: Figure 14 of Arup's Hydro-geological map* **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?

No – The surface water that flows from the proposed development will be routed the same way as before: water is and will be collected from hard surfaced areas and enter the existing drainage system.

The proposed light well will be constructed at side of the property. And hence the impermeable areas may change.

Question 3. Will the proposed basement development result in a change to the hard surfaced / paved external areas?

No – Currently the site is fully occupied by buildings and hard-surfaced areas. This will remain the case with the proposed development.

Question 4. Will the proposed basement result in changes to the inflows (instantaneous and long term) of surface water being received by adjacent properties or downstream watercourses?

No. The proposed development will enter the current drainage system.

Question 5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?

No. The quality of water is unlikely to be altered.

Question 6. Is the site in an area identified to have surface water flood risk according to either the Local Flood Risk Management Strategy or the Strategic Flood Risk Assessment or is it at risk from flooding, for example because the proposed basement is below the static water level of nearby surface water feature?

The potential sources of flooding are summarised below:

Potential Source	**Potential Flood Risk at Site?**	**Justification**
Fluvial flooding	No	EA Flood Mapping shows Flood Zone 1. Distance from nearest surface watercourse >1km
Tidal flooding	No	Site location is 'inland' and topography > 40mAOD.
Flooding from rising / high groundwater	No	Site is located on low permeability London Clay.
Surface water (pluvial) flooding	No	The 8-10 Stukeley Street, London, WC2B 5LB is not noted on the flood street list

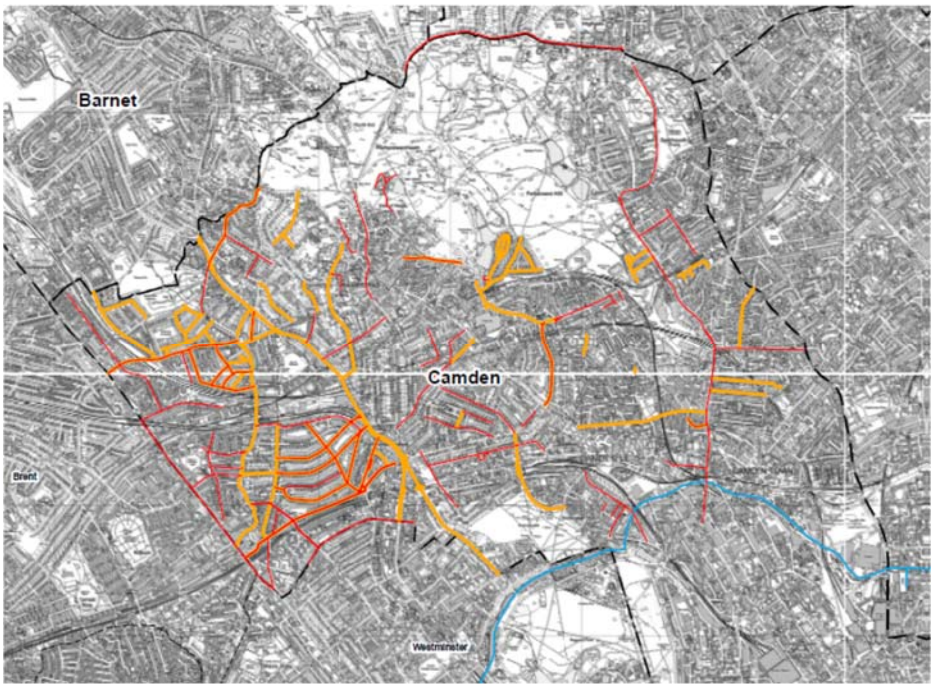


Figure 4: Surface flooding

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 8-10 Stukeley Street, London, WC2B 5LB require an insertion of a basement. The basement is under the footprint of the property which will not affect the overall flow. The basement enlarges the existing dwelling and is not an additional unit. There will be a lightwell to the side. This will increase the hardstanding slightly which may increase flow.
	Question 1: **Is the site within the catchment of the pond chains on Hampstead Heath?** **No.** Further info required from Scoping stage
	Question 2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route? **No**. There will be a side lightwell at basement level which will increase the hardstanding slightly which may increase flow. **Unknown** – The light wells may reduce the impermeable areas. Carry forward to Site Investigation & desk Study
	Question 3. Will the proposed basement development result in a change to the hard surfaced /paved external areas? **No**. The area of hard standing remains unchanged by the development.
	Question 4. Will the proposed basement result in changes to the inflows (instantaneous and long term) of surface water being received by adjacent properties or downstream watercourses?

	No. The area of hard standing will change by the development. **Unknown** – The light well may reduce the impermeable areas. Carry forward to Site Investigation & desk Study Will this increase the hardstanding? There is already an internal side lightwell at ground floor
Flooding	As described at the screening stage the site was not noted on the list of streets flooded in 1995 or 2002. A flood risk assessment is therefore not required.

3.

3. Site Investigation and Study

This section identifies the relevant features of the site and its immediate surroundings, providing further scoping where required.

Desk Study and Walkover Survey

Miss Eleni Pappa, MSc, BEng, an Engineer from Croft Structural Engineers, visited 6 Stukeley Street, London, WC2B 5LQ.

Date of inspection was on the on the 21st September 2015.

Proposed Development

Figure 5: Front of 6 Stukeley Street

6 Stukeley Street is an existing one storey, three bedroom property and it is part of an amalgamation of properties that have evolved on the site since mid 1800's. No. 6 is a terraced property of original Victorian construction: solid load bearing masonry walls and concrete ground floor. The external walls appear to be 225mm thick solid masonry with a metal stud internal lining wall on all sides. Internally all partitions are studs. The property is arranged over one floor which is raised from street level.

Proposed Works

The proposed works require the construction of:

- A new basement under the property.
- Light well to the side of the property
- Superstructure works above the basement

The superstructure works have been considered but is not required to be detailed at planning so has not been included in the Basement Impact Assessment.

CROFT STRUCTURAL ENGINEERS

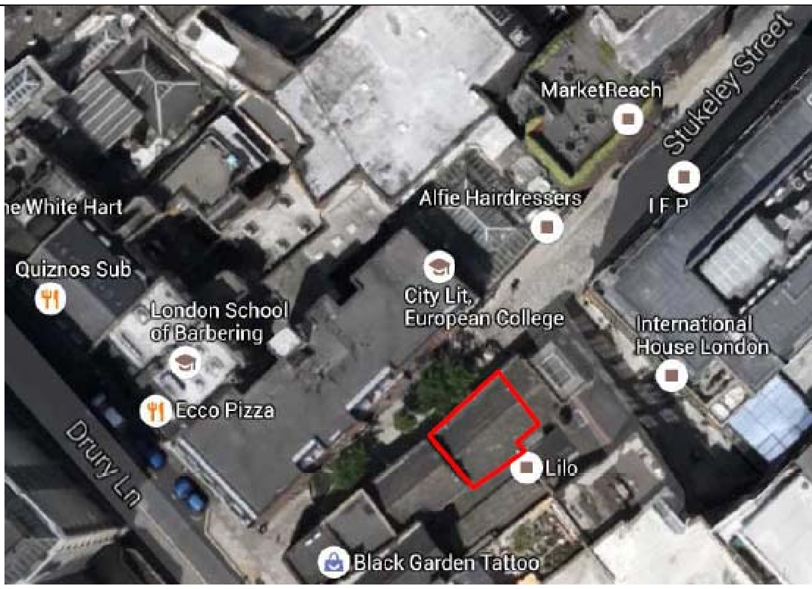


Figure 6: Plan location of 6 Stukeley Street

Location

Stukeley Street is located off Drury Lane and includes a cul-de-sac (court) in front of 6 Stukeley Street, as well as a pedestrianised passage that links Stukeley Street with Drury Lane. 6 Stukeley Street is a terraced property and plays a vital role in negotiating the change in scale from International House (former City Literary Institute) opposite and the other properties on Stukeley Street.

Site History

In the late 18th century, the site was known as the Coal Yard. In the early 20th century many of the original buildings in the street were small tenements and semi-industrial buildings, which were used for the storage of goods. Today Stukeley Street lies just off Drury Lane, and is a pedestrianised way leading to Smart's Place. Since the 1970s, only internal alterations to No. 6 Stukeley Street have been proposed.

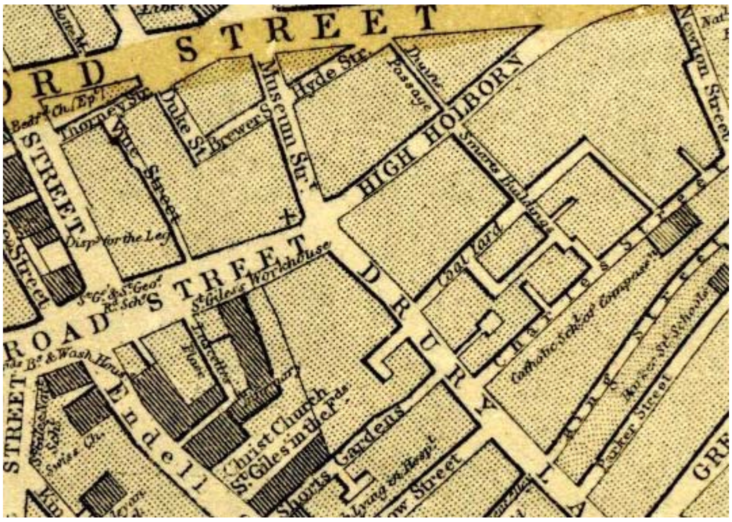


Figure 7: Map of London, 1868, by Edward Weller

<table>
<tr><td>Local Bombing</td><td>It can be seen from the map below that no properties on the street have been bombed in the past.


Figure 8: The London Blitz map</td></tr>
<tr><td>Listed Buildings</td><td>**Is the building or Adjacent buildings listed**
No. The building and the adjacent buildings are not listed.


Figure 9: English heritage map</td></tr>
<tr><td></td><td>Highways, Rail and London Underground

Yes. Site is within 5m of the footpath/alleyway and the road surface</td></tr>
<tr><td>London Underground, Network Rail & Crossrail</td><td>**Is the site over (or within the exclusion zone) of any tunnels, e.g. railway lines?**

The site is more than 20m away from the nearest national rail line. The proposed basement is unlikely to significantly affect this. The nearest London</td></tr>
</table>

Underground Line is the Circle Line. This is more than 30m away and is therefore unlikely to be affected by the works.

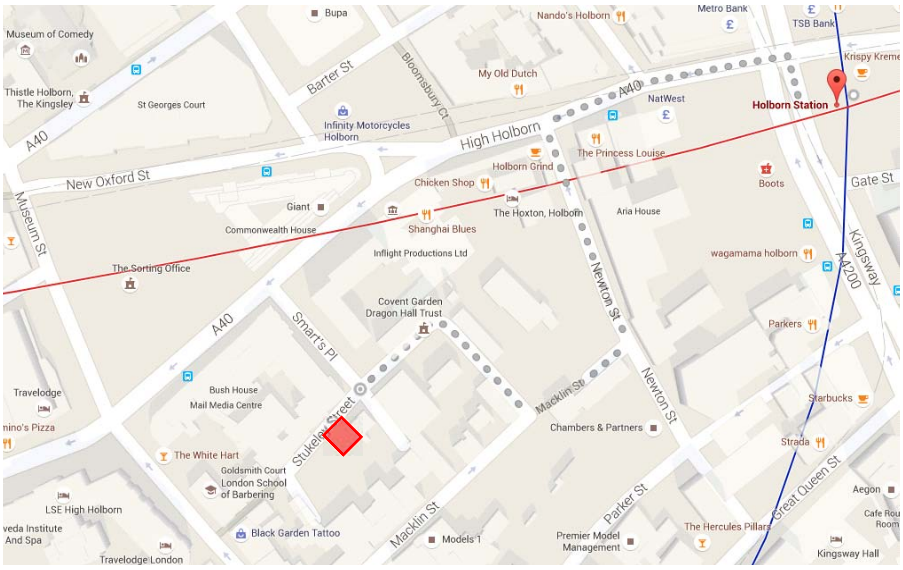


Figure 10: Location Plan

The Elizabeth Line runs more than 50m away. A search on Crossrail's website has confirmed that the development is not in the Safeguarding Zone for this line.

Figure 11: Location Plan

UK Power Networks

Will the basement works affect any UK Power Network Assets?
(Substations etc)
No, UK Power Network Assets will not get affected

Vicinity of Trees

No trees are affected by the basement.

Are any trees to be removed due to the basement?
No, no trees are to be removed.

CROFT STRUCTURAL ENGINEERS

Building Defects

A visual inspection was undertaken of the existing building with particular attention given to movement of the building. The defects noted were:

6 Stukeley Street

The property at 6 Stukeley Street is of original Victorian construction: solid load bearing masonry walls and a concrete ground floor.

Externally the brickwork showed signs of deterioration. Internally the walls were lined with a metal stud partition. Signs of damp could be seen in most of the rooms.

<u>Structural assessment of ongoing movement</u>

In several locations, cracks are present, suggesting that movement has occurred.

Figure 12: Cracking noted over the main entrance

Figure 13: Front brickwork view No. 4 Stukeley Street side

Figure 14: Front brickwork view No. 4 Stukeley Street side

Figure 15: Front brickwork view No. 8 Stukeley Street side

Figure 16: Damp to the living room area at the rear

Figure 17: Cracking noted internally

Figure 18: Metal stud wall internally, solid wall externally

	The cracking and condition of the wall is significant and must be monitored. Croft recommends that underpinning is undertaken to construct the new basement and the existing walls are then rebuilt for the new superstructure.
	Adjacent Properties The condition of the adjacent buildings has 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 19: Site layout showing outline of 6 Stukeley St*
8-10 Stukeley Street – Property to Left	Property Age : mid 1800's Property use : commercial and residential Number of storeys : 1-2 storeys Is a basement present? : No Structural Defects Noted: The properties at 9 and 10 Stukeley Street form part of the building, with Nos 10 being combined with the shop on Drury Lane. No 8 is the corner plot. It is of original Victorian construction; solid load bearing masonry walls, timber floors and concrete ground floor.

	The building is constructed of solid masonry walls and has a timber roof truss. To the rear, a box frame has been inserted to open the ground floor. The rear first floor is a lightweight timber frame construction. The ground floor did not show any signs of damage. The first floor is less well decorated and is showing signs of concern. These are: 1. The first floor rear is constructed of lightweight materials. The roof is an untied truss. At the junction of the cross walls, there are cracks and open joints. The open joints run around the baths and also at the junction of walls and columns.  *Figure 20: Internal cracking at wall junction*  *Figure 21: Internal cracking at wall junction*

Figure 22: Internal cracking within bathroom

The cracking around the bath shows the mastic has failed. Clearly movement in the region of 4mm has occurred since the mastic was installed. This is not minor movement.

Cracking to the main studio room. There is cracking noted around the door of the main studio. The cracking is around 5-10mm in width. We were informed that this cracking occurred during the opening up works on the ground floor.

Figure 23: Internal cracking within the property

The cracking is significant and must be monitored.

Nos 4 Stukeley Street – Property to Right

Property Age : mid 1800's

Property use : commercial and residential

Number of storeys : 2 storey

Is a basement present? : No

	Structural Defects Noted  *Figure 24: Cracking to the top left front corner*  *Figure 25: Bottom left front wall corner view*
Nos 8-10 Stukeley Street – Property to Rear and left hand side	Property Age : mid 1800's Property use : commercial and residential Number of storeys : storeys Is a basement present? : No To the rear of 6 Stukeley Street there is an open courtyard which belongs to No. 8-10 Stukeley street. Structural Defects Noted: See property to left, above.

Local Topography	The land is level with no major falls.
Ground Investigation	A ground investigation has been undertaken see separate report.
Geology	See Ground investigation report and Geology report
Surface Flow & Flooding	**Conceptual Model** The proposed works at the property require construction of a basement. The basement is under the footprint of the property and will therefore not affect the above ground flow. The basement enlarges the existing property and is not an additional self-contained unit intended to serve as a standalone dwelling.
Rainwater down pipes, Drains, Manholes and Gulley	As described previously, there is a surface water drainage gully in the front.
Local Water Sources	**Are there any ponds lakes or water courses on the site or adjacent sites?** No ponds, lakes or water course are within the site or the adjacent sites.
	Field Investigation Ground investigation specialists visited the site and subsequently produced a 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 the Chartered Hydrogeologist's report 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 (Ground and Water ltd.). From the Scoping stage we considered that their brief should cover: • Two trial pits to the front and 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. • One bore hole to a depth of 8m below ground level. • 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.

4. Basement Impact Assessment

Subterranean Flow	Refer to Hydrogeologist report
Land Stability	Refer to Geologist Report
Conservation and Listed Buildings	If the property is in a conservation area, or it is listed then management plan for demolition and construction may be needed. This is not included with this BIA document and is not within the Croft Structural Engineers' Brief.
Surface water flow and flooding	As described in previous sections there are no significant risks of flooding. However, there are risks at present which are inherit in the construction of all subterranean structures, such as flooding due to unexpected failure of the drainage, water mains, etc. For this reason we would recommend the following measures to reduce the risks mentioned above: • A pumping mechanism will be installed for the proposed basement. There is a likelihood that this may fail and allow excess water to accumulate. If this were to occur, the build-up of water would be gradual and noticeable before it becomes a significant life-threatening hazard. • Install a dual pumping system to maintain operation in the event of a failure. This should include a battery backup and a suitable alarm system for warning purposes. • To reduce the impact of surface water flooding, sustainable drainage systems such as on site attenuation should be considered at detailed design stage. The risk of flooding from excess surface water is not considered significant. There is a risk of flooding due to the failure of the pumping system but this can be reduced to acceptable levels with appropriate design and installation measures.

SUDS Assessment

Hard standing

Existing Hard Standing	= 0 m^2
Proposed Hardstanding	= 7.25 m^2
Increase in Hard standing	= 7.25 m^2

SUDS Assessment

Given the increase of hardstanding a SUDS proposals will be required.

SUDS Calculations

ATENUATION DESIGN

ATTENUATION DESIGN

In accordance with CIRIA publication C697 - The SUDS Manual

Tedds calculation version 1.0.02

EA_Defra method

Site characteristics

Location	London
Hydrological region	6
Soil type (Wallingford Procedure W.R.A.P map)	2
Standard percentage runoff	SPR = **0.30**
Average annual rainfall	SAAR = **600** mm
5 year return period rainfall of 60 minute duration	M5_60min = **20.0** mm
Ratio 60-minute to 2 day rainfalls of 5 year return	r = **0.44**
Rainfall intensity increase due to global warming	$p_{climate}$ = **20**%
Impervious area req. attenuation storage	α = **100.0** %

Catchment details

Subcatchment	Name	Area (ha)	PIMP (%)	Impermeable. area (ha)
1	**lightwell**	**0.00**	**100.0**	**0.00**
	Total	**0.00**	**100.0**	**0.00**

Greenfield runoff rates

Catchment area	AREA = **50.00** hectare
Greenfield runoff rate (50 hectare site)	$\bar{Q}_{rural} = 0.00108 m^3/s \times (AREA/1km^2)^{0.89} \times (SAAR/1mm)^{1.17} \times SPR^{2.17}$ = **76.1** l / s
Greenfield runoff rate	$\bar{Q} = \bar{Q}_{rural} / AREA \times A$ = **0.0** l / s
Greenfield runoff rate per unit area	$\bar{Q}_A = \bar{Q} / A$ = **1.5** l / s / hectare

Estimated site discharges

FSR growth rate (1 year)	FSR_{1yr} = **0.85**
Discharge (1 year)	$Q_{1yr} = \bar{Q} \times FSR_{1yr}$ = **0.0** l/s
FSR growth rate (30 year)	FSR_{30yr} = **2.30**
Discharge (30 year)	$Q_{30yr} = \bar{Q} \times FSR_{30yr}$ = **0.0** l/s
FSR growth rate (100 year)	FSR_{100yr} = **3.19**
Discharge (100 year)	$Q_{100yr} = \bar{Q} \times FSR_{100yr}$ = **0.0** l/s

Estimated attenuation volume - 1 year

Attenuation storage vol (fig A7.1 - A7.8)	$Uvol_{1yr}$ = **205.0** m^3 / hectare
Basic storage volume	$BSV_{1yr} = Uvol_{1yr} \times \alpha \times A$ = **0.10** m^3
FEH rainfall factor (figs A11.1, A6.1.1 - A6.3.4)	FF_{1yr} = **0.90**
Storage volume ratio (fig A8.1 - A8.8)	SVR_{1yr} = **1.46**
Adjusted storage volume	$ASV_{1yr} = SVR_{1yr} \times BSV_{1yr}$ = **0.15** m^3
Hydrological regional volume ratio (fig A9.1)	HR_{1yr} = **1.01**
Final estimated attenuation storage	$Vol_{1yr} = HR_{1yr} \times ASV_{1yr}$ = **0.15** m^3

Library item: Estimated attenuation output

Estimated attenuation volume - 30 year

Attenuation storage vol (fig A7.1 - A7.8)	$Uvol_{30yr}$ = **420.0** m^3 / hectare
Basic storage volume	$BSV_{30yr} = Uvol_{30yr} \times \alpha \times A$ = **0.21** m^3
FEH rainfall factor (figs A11.1, A6.1.1 - A6.3.4)	FF_{30yr} = **0.80**
Storage volume ratio (fig A8.1 - A8.8)	SVR_{30yr} = **1.74**
Adjusted storage volume	$ASV_{30yr} = SVR_{30yr} \times BSV_{30yr}$ = **0.37** m^3
Hydrological regional volume ratio (fig A9.1)	HR_{30yr} = **1.01**
Final estimated attenuation storage	$Vol_{30yr} = HR_{30yr} \times ASV_{30yr}$ = **0.37** m^3

Library item: Estimated attenuation output

Estimated attenuation volume - 100 year

Attenuation storage vol (fig A7.1 - A7.8)	$Uvol_{100yr}$ = **525.0** m^3 / hectare
Basic storage volume	$BSV_{100yr} = Uvol_{100yr} \times \alpha \times A$ = **0.26** m^3
FEH rainfall factor (figs A11.1, A6.1.1 - A6.3.4)	FF_{100yr} = **0.75**
Storage volume ratio (fig A8.1 - A8.8)	SVR_{100yr} = **1.74**
Adjusted storage volume	$ASV_{100yr} = SVR_{100yr} \times BSV_{100yr}$ = **0.46** m^3
Hydrological regional volume ratio (fig A9.1)	HR_{100yr} = **1.02**
Final estimated attenuation storage	$Vol_{100yr} = HR_{100yr} \times ASV_{100yr}$ = **0.47** m^3

Library item: Estimated attenuation output

Attenuation storage required

Vol. increase due to head-discharge relationship	p_{hydro} = **1.25**
Maximum attenuation storage required	$V_{req\_max} = Vol_{30yr} \times p_{hydro}$ = **0.5** m^3

Interception storage

Interception rainfall depth	d_{int} = **10** mm
Volume of interception storage required	$V_{int\_req} = 0.8 \times A_{imp} \times d_{int}$ = **0.04** m^3

Long term storage

Proportion of paved area draining in to network	α = **1.0**
Proportion of pervious area draining in to network	β = **0.5**
Rainfall depth for 100years, 6 hour event	RD = M100_360 = **70.4** mm
Extra runoff vol of dev.runoff over greenfield runoff	$Vol_{xs} = \max(RD \times A \times (PIMP \times \alpha \times 0.8 + ((1 - PIMP) \times \beta \times SPR) - SPR), 0m^3)$ = **0.18** m^3

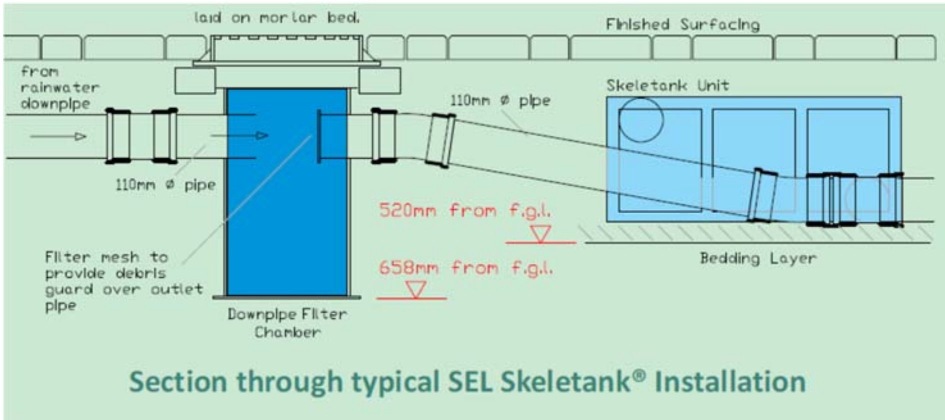
Treatment volume

Treatment volume (assume 80% runoff) $T_{vol} = 0.8 \times A \times 15mm \times PIMP =$ **0.06** m^3

Mitigation Measures

From the SUDS calculations a volume of 0.5m³ is required for storage we would recommend that onsite storage is used.

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



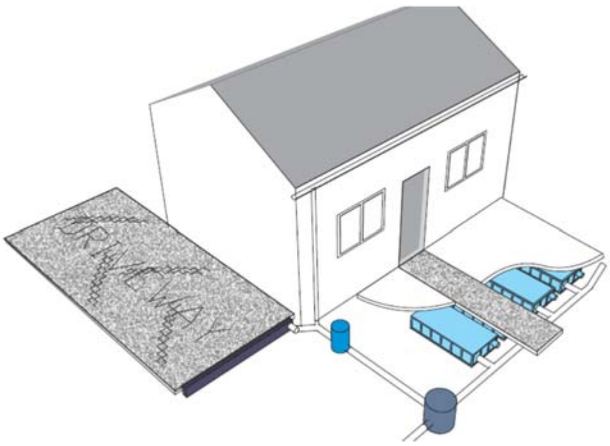


Figure 26 Diagrammatic Representation Only

A proposal for a pumped drainage mechanism, with alarm fixtures, is appended.

Drainage effects on Structure

Not build over agreements known of.

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

Trees

Root Protection Zone

RPA = 1.5 x Crown diameter.

The closest tree is positioned at the front of No 12 Stukeley street and approximately 2.5m away from the front face of the front retaining walls. The basement is not within the RPA of the trees noted below

Figure 27: Young Mimosa tree outside 12-14 Stukeley Street

Site: 6 Stukeley Street

Reference: Root protection area calculator

Date: 1/18/16 **Surveyor:** Eleni Pappa

- ► Enter field mensuration data into red boxes to commence calculation.
- ► All results to be read with reference to the recommendations set out in the ***BS5837:2005 Trees in Relation to Construction*** and corrections/variations made accordingly.

ROOT PROTECTION AREA (RPA) CALCULATOR

Single stem trees

Diameter at 1.5m above ground level: 130 millimetres

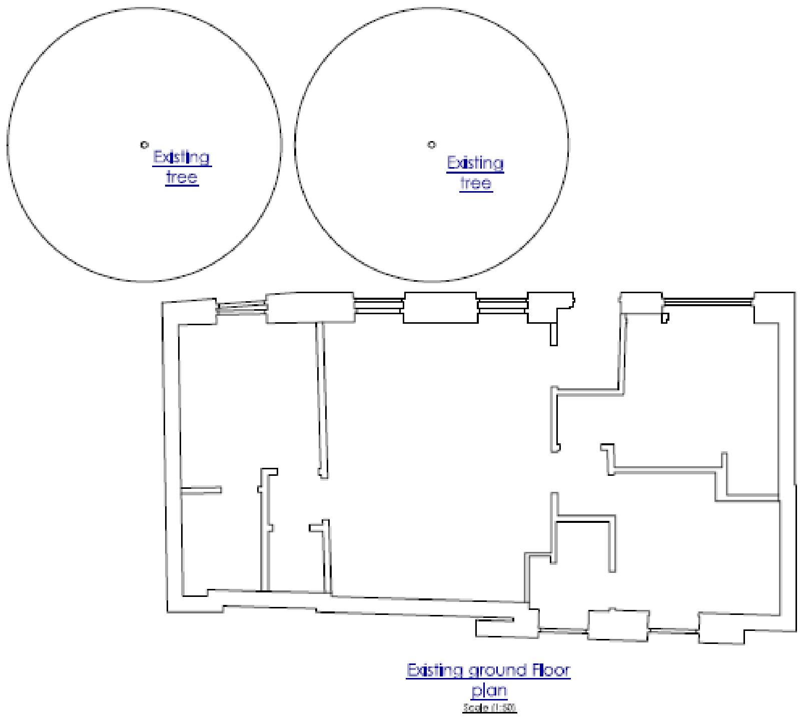
RPA: 7.6 m^2 **See Note**

equates to circle with radius of 1.6 m

equates to square with approx 2.8 m si

20% Offset 0.3 m

The basement is not within the RPA of the trees noted below



Conclusion

The basement does not cut 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	
	The design and construction methodology aims to limit damage to the existing building on the site, and to the neighbouring buildings, to Category 2 or lower as set out in Table 2.5 of CIRIA report C580. For this development, suitable temporary propping during the construction phase will limit the amount of movement due to the basement works. This is described in the Basement Method Statement (appended). The ground movement assessment is contained within the Land Stability BIA.

Mitigation Measures Ground Movement

A method statement, appended, has been formulated with Croft's experience of over 500 basements completed without error. As mentioned previously, the procedures described in this statement will mitigate the impacts that the construction of the basement will have on nearby properties.

The works must be carried out in accordance with the Party Wall Act and condition surveys will be necessary at the beginning and the end of the works. The Party Wall Approval procedure will reinforce the use of the proposed method statement and, if necessary, require it to be developed in more detail with more stringent requirements than those required at planning stage.

It is not expected that any cracking will occur in nearby structures during the works. However, Croft's experience advises that there is a risk of movement to the neighbouring property.

To reduce the risk to the development:

- Employ a reputable firm that has extensive knowledge of basement works.
- Employ suitably qualified consultants Croft Structural Engineers has completed over 500 basements in the last five years.
- Provide method statements for the contractors to follow
- Investigate the ground this has now been done.
- Record and monitor the properties close by. This is completed by a condition survey under the Party Wall Act, before and after the works are completed. Refer to the end of the appended Basement Construction Method Statement.

With the measures listed above, the maximum level of cracking anticipated is 'Hairline' cracking. This can be repaired with normal decorative works. Under the Party Wall Act, minor damage, although unwanted, can be tolerated it is permitted 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.
With the above the maximum level of cracking anticipated is 'Hairline' cracking, which can be remediated with decorative repairs. Under the Party Wall Act, minor damage is considered acceptable (although unwanted) in a neighbouring property as long as repairs are suitability undertaken to rectify this. To ensure this risk is mitigated, the Party Wall Act is to be followed and a Party Wall Surveyor will be appointed.

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

Monitoring Assessment

The level of Monitoring Croft recommend on 8-10 Stukeley Street is:

Monitoring Level proposed	Type of Works.
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 shown within the proposed monitoring statement (appended).

CROFT STRUCTURAL ENGINEERS

Basement Design & Construction Impacts

Foundation type	Reinforced concrete cantilevered retaining walls The design for the retaining wall 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 wall is designed 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 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 only considers floatation as a risk if the recorded ground water level is lower than the basement. The design accounts for 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. 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 road loading. The site has a paved road at the front. This is not a main road and regular traffic is not allowed. However, cars have been spotted parked outside the property and hence the area in front of the property is used as an access point for some of the businesses. Highways loadings would not be required however, given the possibility that vehicles may enter the front, a surcharge of 10kN/m^2 should be allowed for in the design. Highways loading allow: 10kN/m2 if within 45° of road 5kN/m2 if within 45° of Pavement Surcharge for adjacent property 1.5kN/m2 + 4kN/m2 for concrete ground bearing slab

<table>
<tr><td>Intended use of structure and user requirements</td><td>Family/domestic use</td></tr>
<tr><td>Loading Requirements (EC1-1)</td><td>
<table>
<tr><th></th><th>UDL kN/m²</th><th>Concentrated Loads kN</th></tr>
<tr><td>Domestic Single Dwellings</td><td>1.5</td><td>2.0</td></tr>
</table>
The basement does line within a 45º angle of the highway.
Therefore Highways HA loading is required to be applied.
</td></tr>
<tr><td>Part A3 Progressive collapse</td><td>
Number of Storeys 1

Is the Building Multi Occupancy? No
<table>
<tr><th></th><th></th></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>
</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>
<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 the proposed basement, then ignore
</td></tr>
</table>

	Design Permanent condition for water table level: If deeper than proposed, 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. A ground investigation showed that water was present at a level lower than the proposed formation level of the basement. Standpipe monitoring, which included readings from return visit, and no ground water was encountered
Additional loading requirements	**Surcharge Loading** The following will be applied as surcharge loads to the front retaining walls: • $10kN/m^2$ if within 45° of road • 100kN point loads if under road or within 1.5m • $5kN/m^2$ if within 45° of Pavement • Garden Surcharge $2.5kN/m^2$ + 1 m of soil (if present above basement ceiling) $20kN/m^2$ • Surcharge for adjacent property $1.5kN/m^2$ + $4kN/m^2$ for concrete ground bearing slab <u>Highways loading:</u> The basement is within 5m of the pavement but not within 5m of the public highway. However, there is a vehicle access road to the front of the property where service vehicles can access. <u>Adjacent Properties:</u> All adjacent property footings within 45° to have additional geotechnical engineers input. A line at 45° from the base of the neighbours' wall footing would be intersected by the basement retaining wall. This should be accounted for in the design. The appended calculations show the design of one of the most heavily loaded retaining wall. The most critical parameters have been used for this.
Mitigation Measures - Internal Flooding	To mitigate the risks associated with failure of infrastructure, Croft would recommend the following measures to reduce these risks: • A pumping mechanism will be installed for the proposed basement. There is a likelihood that this may fail and allow excess water to accumulate. If this were to occur, the build-up of water

	would be gradual and noticeable before it becomes a significant life-threatening hazard. • Install a dual pumping system to maintain operation in the event of a failure. This should include a battery backup and a suitable alarm system for warning purposes. • To reduce the impact of surface water flooding, sustainable drainage systems such as on site attenuation should be considered at detailed design stage.
Mitigation Measures - Drainage and Damp-proofing	The design of drainage and damp-proofing is not within the scope of this assessment and would not normally be expected to be part of the structural engineer's remit at detailed design stage. A common and anticipated detailed design stage approach is to use internal membranes (Delta or similar). These will be integral to the waterproofing of the basement. Any water from this will enter a drainage channel below the slab. This will be pumped and discharged into the exiting sewer system. It is recommended that a waterproofing specialist is employed to ensure all the water proofing requirements are met. The waterproofing specialist must name their structural waterproofer. The structural waterproofer must inspect the structural details and confirm that he is happy with the robustness. Due to the segmental construction nature of the basement, it is not possible to water proof the joints. All waterproofing must be made by the waterproofing specialist. He should review the structural engineer's design stage details and advise if water bars and stops are necessary. The waterproofing designer must not assume that the structure is watertight. To help reduce water flow through the joints in the segmental pins, the following measures should be applied: • All faces should be cleaned of all debris and detritus • Faces between pins should be needle hammered to improve key for bonding • All pipe work and other penetrations should have puddle flanges or hydrophilic strips
Mitigation Measures - Localised Dewatering	Monitor water levels 1 month prior to starting on site and throughout the construction process. Localised dewatering to pins may be necessary.

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. Critical areas where point loads are present from above include: Cross walls Door openings To demonstrate the feasibility of the works, a proposed basement construction method statement is appended.
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 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. The ground floor slab is not being removed, minimising the vibration and sound to adjacent properties. Working in the basement 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. The level of noise from basement construction works 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: Structural Calculations

CPG4 section 5 highlights that other permits and requirements will be necessary after planning. Item 5.1 highlights that Building Regulations will be required. As part of the building control pack full calculations must be undertaken and provided at detailed design stage once planning permission is granted. The calculations must be completed to a recognised Standard (BS or Euro Codes). The calculations must take into account the findings of this report and the recommendations of the auditors.

The design must resist:

- Vertical loads from the proposed works and adjacent properties
- Lateral loads from wind, soil water and adjacent properties
- Loadings in the temporary condition
- All other applied loads on the building
- Uplift forces from hydrostatic effects and soil heave

The final proposed scheme must:

- Provide stability in the temporary condition to all forces
- Provide stability to all forces in the permanent condition

As part of the planning Croft structural engineers has considered some of the pertinent parts of the basement structure to ensure that it can be constructed. The following calculations are not a full set of calculations for the final design which must be provided for building regulations. The structural calculations we consider pertinent and included in this appendix for this development are:

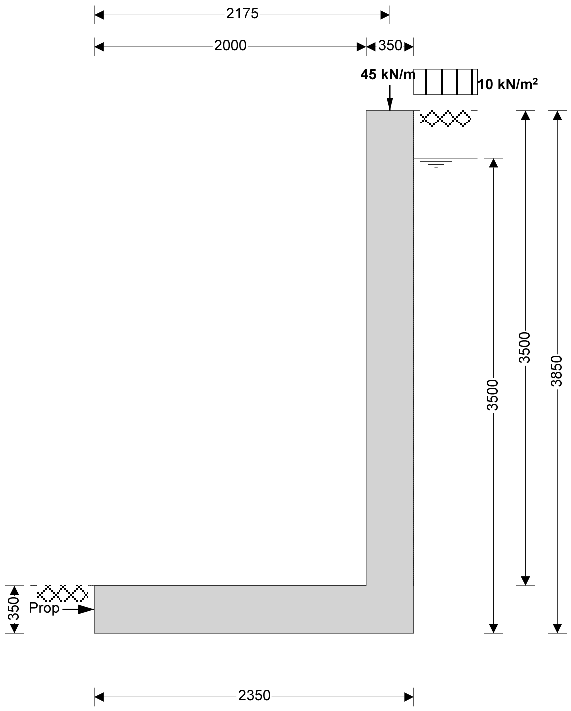
1. Front basement foundation & retaining wall with highways loading as necessary

2. Party Wall foundation and retaining wall

Retaining Wall Design

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever propped at base**
Height of retaining wall stem	h_{stem} = **3500** mm
Thickness of wall stem	t_{wall} = **350** mm
Length of toe	l_{toe} = **2000** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **2350** mm
Thickness of base	t_{base} = **350** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **1900** mm
Thickness of downstand	t_{ds} = **350** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3850** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **3500** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **3150** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3850** mm

Retained material details

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

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

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

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

Base material details

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

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

Design base friction δ_b = **18.6** deg

Allowable bearing pressure $P_{bearing}$ = **200** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

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

Passive pressure coefficient for base material

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

At-rest pressure

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

Loading details

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

Applied vertical dead load on wall W_{dead} = **30.0** kN/m

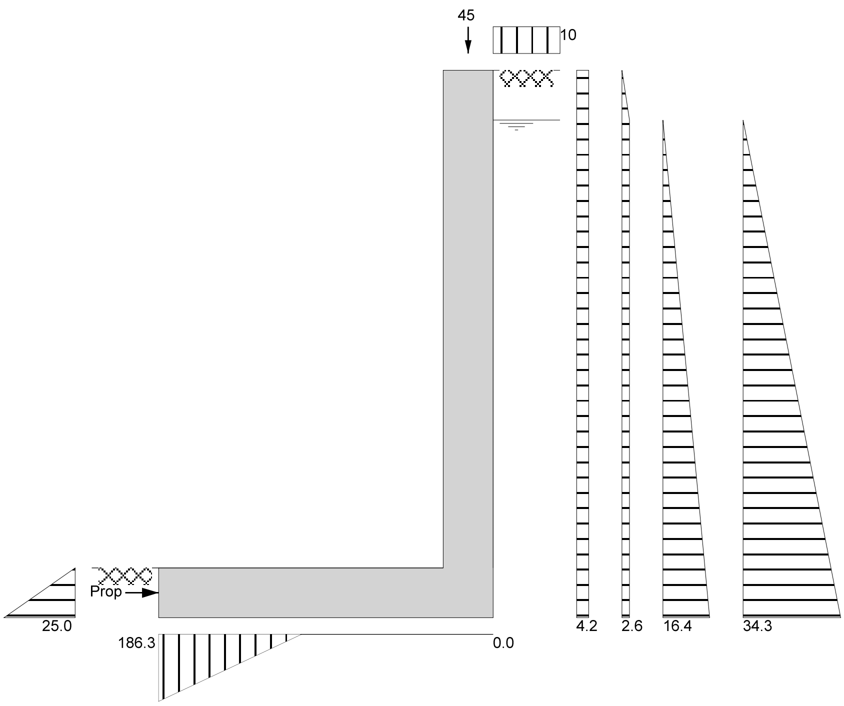
Applied vertical live load on wall W_{live} = **15.0** kN/m

Position of applied vertical load on wall l_{load} = **2175** mm

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

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

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



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

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **28.9** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **19.4** kN/m

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

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **93.3** kN/m

Horizontal forces on wall

Surcharge	$F_{sur} = K_a \times Surcharge \times h_{eff}$ = **16.1** kN/m
Moist backfill above water table	$F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **0.5** kN/m
Moist backfill below water table	$F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **9.2** kN/m
Saturated backfill	$F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **28.7** kN/m
Water	$F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **60.1** kN/m
Total horizontal load	$F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **114.6** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.4**
kN/m	
Propping force	$F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0\ kN/m)$
	F_{prop} = **83.8** kN/m

Overturning moments

Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **31** kNm/m
Moist backfill above water table	$M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **1.7** kNm/m
Moist backfill below water table	$M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **16.1** kNm/m
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **33.5** kNm/m
Water	$M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **70.1** kNm/m
Total overturning moment	$M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **152.4** kNm/m

Restoring moments

Wall stem	$M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **62.9** kNm/m
Wall base	$M_{base} = w_{base} \times l_{base} / 2$ = **22.8** kNm/m
Design vertical dead load	$M_{dead} = W_{dead} \times l_{load}$ = **65.3** kNm/m
Total restoring moment	$M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **150.9** kNm/m

Check bearing pressure

Design vertical live load	$M_{live} = W_{live} \times l_{load}$ = **32.6** kNm/m
Total moment for bearing	$M_{total} = M_{rest} - M_{ot} + M_{live}$ = **31.2** kNm/m
Total vertical reaction	$R = W_{total}$ = **93.3** kN/m
Distance to reaction	$x_{bar} = M_{total} / R$ = **334** mm
Eccentricity of reaction	$e = abs((l_{base} / 2) - x_{bar})$ = **841** mm
	Reaction acts outside middle third of base
Bearing pressure at toe	$p_{toe} = R / (1.5 \times x_{bar})$ = **186.3** kN/m^2
Bearing pressure at heel	p_{heel} = 0 kN/m^2 = **0** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

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

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **40.5** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **27.2** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **66** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **133.6** kN/m

Factored horizontal at-rest forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **36.3** kN/m

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **0.9** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **18.2** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **56.6** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **84.1** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **196.2** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **6.1** kN/m

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

F_{prop_f} = **153.2** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **70** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **3.3** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **31.9** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **66.1** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **98.1** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **269.3** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **88** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **31.9** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **143.6** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **263.5** kNm/m

Factored bearing pressure

Total moment for bearing $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **-5.8** kNm/m

Total vertical reaction $R_f = W_{total\_f}$ = **133.6** kN/m

Distance to reaction $x_{bar\_f} = M_{total\_f} / R_f$ = **-44** mm

Eccentricity of reaction $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **1219** mm

WARNING - Beyond scope of calculation

Bearing pressure at toe $p_{toe\_f} = R_f / (1.5 \times x_{bar\_f})$ = **-2043** kN/m²

Bearing pressure at heel p_{heel_f} = 0 kN/m² = **0** kN/m²

Rate of change of base reaction $rate = p_{toe\_f} / (3 \times x_{bar\_f})$ = **15615.03** kN/m²/m

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

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

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

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

Material properties

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

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

Base details

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

Cover to reinforcement in toe c_{toe} = **75** mm

Calculate shear for toe design

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base}$ = **23.1** kN/m

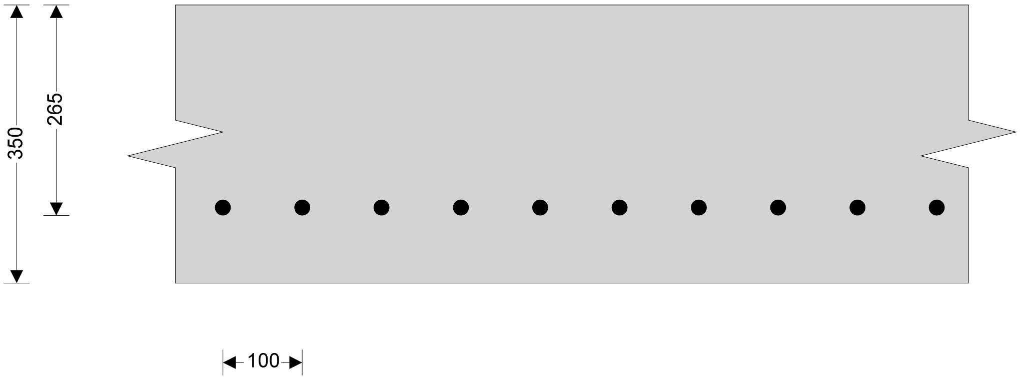
Total shear for toe design $V_{toe} = V_{toe\_wt\_base}$ = **23.1** kN/m

Calculate moment for toe design

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2)$ = **27.4**

kNm/m

Total moment for toe design $M_{toe} = M_{toe\_wt\_base}$ = **27.4** kNm/m



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2)$ = **265.0** mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu})$ = **0.010**

Compression reinforcement is not required

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

z_{toe} = **252** mm

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe})$ = **250** mm^2/m

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base}$ = **455** mm^2/m

Area of tension reinforcement required $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min})$ = **455** mm^2/m

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

Area of reinforcement 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 $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.087** N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_toe} = **0.867** N/mm^2

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

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

Material properties

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

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

Wall details

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

Cover to reinforcement in stem c_{stem} = **75** mm

Cover to reinforcement in wall c_{wall} = **30** mm

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **33** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **0.9** kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **16.4** kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **45.9** kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **68.1** kN/m

Calculate shear for stem design

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

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **63.6** kNm/m

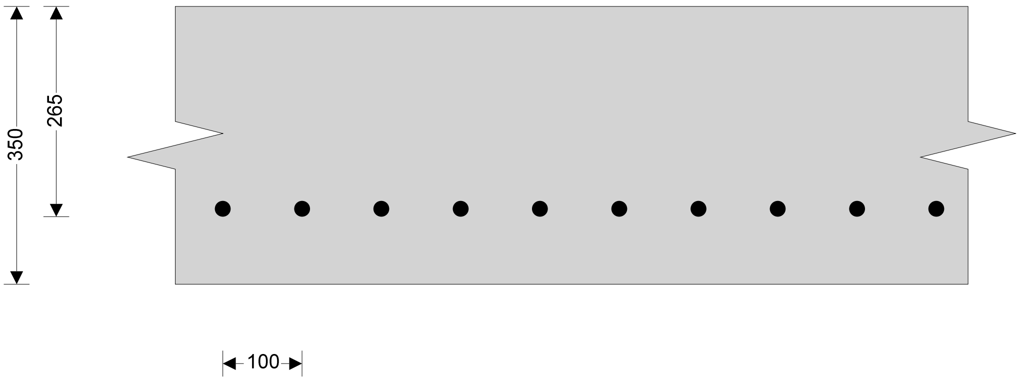
Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **3.1** kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times h_{sat} / 2$ = **25.8** kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3$ = **48.2** kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3$ = **71.5** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **212.3** kNm/m



Check wall stem in bending

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

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **265.0** mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.076**

Compression reinforcement is not required

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

z_{stem} = **240** mm

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **2029** mm^2/m

Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall}$ = **455** mm^2/m

Area of tension reinforcement required $A_{s\_stem\_req} = \text{Max}(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **2029** mm^2/m

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

Area of reinforcement provided $A_{s\_stem\_prov}$ = **3142** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.042** N/mm^2

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

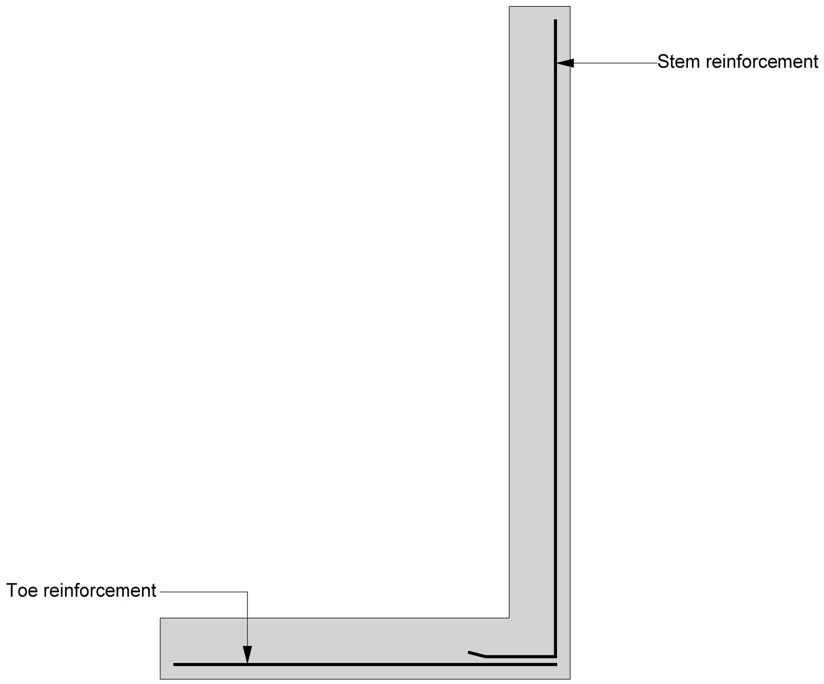
PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_stem} = **0.867** N/mm^2

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

Indicative retaining wall reinforcement diagram



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

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

CROFT STRUCTURAL ENGINEERS

Appendix B: Construction Sequence and Plans

1. Basement Formation Suggested Method Statement

1.1. This method statement provides an approach that will allow the basement design to be correctly considered during construction. The statement also contains proposals for 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 has been written by a Chartered Engineer. The sequencing has been developed using guidance from ASUC (Association of Specialist Underpinning Contractors).

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

1.4. Contact Party Wall Surveyors to inform them of any changes to this method statement.

1.5. On this development, the approach is: construct the underpin segments that will support the permanent steel work; insert the new steelwork; remove load from above and place it onto new supporting steelwork; cast the remainder of the retaining walls that will form the perimeter of the basement.

1.6. The cantilever pins are designed to be inherently stable without lateral support to the top of the wall. However, temporary props will be provided near the head and will provide support until the concrete has gained sufficient strength. 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 soil mass is to be removed in 1/3 portions and cross propping subsequently added as the central soil mass is removed.

1.7. A ground investigation with a borehole has been completed. Below is an abstract from the Soil Investigation report outlining the soil conditions:

The ground conditions encountered within the trial holes constructed on the site generally conformed to that anticipated from examination of the geology map. Made Ground was noted to overlie the Lynch Hill Gravel Member, which was in turn were underlain by the bedrock deposits of the London Clay Formation.

The ground conditions encountered during the investigation are described in this section. For more complete information about the Made Ground, Lynch Hill Gravel Member and the London Clay Formation at particular points, reference must be made to the individual trial hole logs within Appendix B.

The trial hole location plan can be viewed in Figure 4.

For the purposes of discussion the succession of conditions encountered in the trial holes in descending order can be summarised as follows:

Made Ground
Lynch Hill Gravel Member (BH1 only)
London Clay Formation (BH1 only)

Made Ground
Made Ground was encountered from ground level to a depth of 3.20m bgl in BH1 (TP/FE2) and for the full depth of TP/FE1, a depth of 0.67m bgl. The soils comprised a black/dark brown/grey brown silty gravelly sand to sandy gravelly silty clay. The sand was fine to coarse grained and the gravel rare to abundant, fine to coarse, sub-angular to sub-rounded flint, brick, concrete, tarmac, lignite, clinker and wood ash.

Lynch Hill Gravel Member
Soils described as representative of the Lynch Hill Gravel Member were encountered underlying the Made Ground to a depth of 5.00m bgl in BH1. These soils comprised a light brown clayey sandy gravel. The sand was fine grained and the gravel abundant, fine to medium, sub angular to rounded flint.

London Clay Formation
Soils described as the London Clay Formation were encountered underlying the soils of the Lynch Hill Gravel Member for the remaining depth of BH1, a depth of 8.00m bgl. These soils were described as a dark grey silty clay.

For details of the composition of the soils encountered at particular points, reference must be made to the individual trial hole logs within Appendix B.
may have obscured groundwater strikes.

Monitoring of the combined bio-gas and groundwater monitoring well installed in BH1 (installed to 8.00m bgl) by a Ground and Water Limited Engineer revealed a standing water level of 5.60m bgl on the 9th December 2015.

Changes in groundwater level occur for a number of reasons including seasonal effects and variations in drainage. Exact groundwater levels may only be determined through long term measurements from monitoring wells installed on-site. The investigation was undertaken in October and December 2015, when groundwater levels are likely to be rising towards their annual maximum (i.e. highest level).

Isolated pockets of groundwater may be perched within any Made Ground found at other locations around the site.

1.8. The bearing pressures have been limited to 200kN/m^2 as advised in the ground investigation report.

4.4 Groundwater Conditions

1.9. No groundwater was encountered during the intrusive investigation, however the drilling process may have obscured groundwater strikes.

Monitoring of the combined bio-gas and groundwater monitoring well installed in BH1 (installed to 8.00m bgl) by a Ground and Water Limited Engineer revealed a standing water level of 5.60m bgl on the 9th December 2015.

Changes in groundwater level occur for a number of reasons including seasonal effects and variations in drainage. Exact groundwater levels may only be determined through long term measurements from monitoring wells installed on-site. The investigation was undertaken in October and December 2015, when groundwater levels are likely to be rising towards their annual maximum (i.e. highest level).

Isolated pockets of groundwater may be perched within any Made Ground found at other locations around the site.

1.10. The structural waterproofer (not Croft) must comment on the proposed design and ensure that he is satisfied that the proposals will provide adequate waterproofing.

1.1. Provide engineers with concrete mix, supplier, delivery and placement methods two weeks prior to the first pour. Site mixing of concrete should not be employed apart from in small sections (less than $1m^3$). The contractor must provide a method on how to achieve site mixing to the correct specification. The contractor must undertake toolbox talks with staff to ensure site quality is maintained.

2. Enabling Works

2.1. The site is to be hoarded with ply board sheets, at least 2.2m high, to prevent unauthorised public access.

2.2. Licences for skips and conveyors should be posted on the hoarding.

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

2.3.1. 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 a bucket.

2.4. 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. Begin by placing cantilevered walls 1 and 2 noted on plans. (Cantilevered walls to be placed in accordance with Section 4.)

3.2. Needle and prop the floor over.

3.3. Insert steel over and sit on cantilevered walls.

3.3.1. Beams over 6m to be jacked on site to reduce deflections of floors.

3.3.2. Dry pack to steelwork. Ensure a minimum of 24 hours from casting cantilevered walls to dry-packing. Grout column bases

3.4. Excavate out first 1.2m around front opening, prop floor and erect conveyor.

3.5. Continue cantilevered wall formation around perimeter of basement following the numbering sequence on the drawings.

3.5.1. 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). No more than

3.5.2. Floor over to be propped as excavation progresses. Steelwork to support floor to be inserted as works progress.

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

3.7. Excavate and cast floor slab

3.7.1. Excavate 1/2 of the middle section of basement floor. As excavation proceeds, place Slim Shore props at a maximum of 2.5m c/c across the basement. Locate props at a third of the height of the wall.

3.7.2. Continue excavating the next 1/2 and prop.

3.7.3. Place below-slab drainage. Croft recommends that all drainage is encased in concrete below the slab and cast monolithically with the slab. Placing drainage on pea shingle below the slab allows greater penetration for water ingress.

3.7.4. Place reinforcement for basement slab.

3.7.5. Building Control Officer and Engineer are to be informed five working days before reinforcement is ready and invited for inspection.

3.7.6. Once inspected, pour concrete.

3.8. Provide structure to ground floor and water proofing to retaining walls as required. It is recommended to leave 3-4 weeks between completion of the basement and installing drained cavity. This period should be used to locate and fill any localised leakage of the basement

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 1000mm wide). Where excavation is greater than 1.0m deep, provide temporary propping to sides of excavation to prevent earth collapse (Health and Safety). A 1000mm width wall has a lower risk of collapse to the heel face.

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

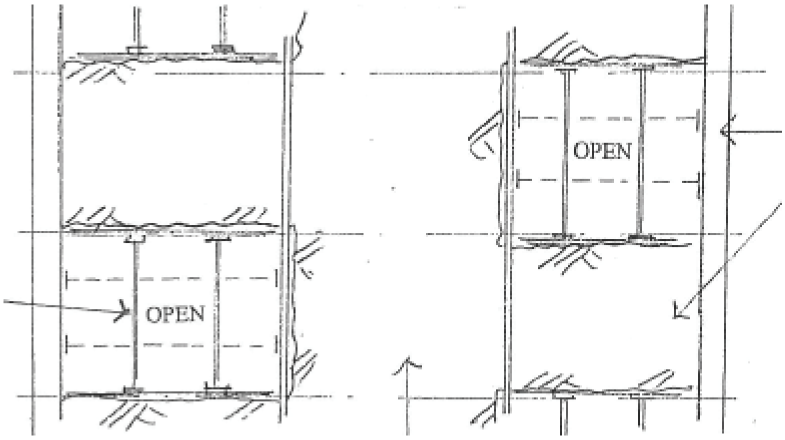


Figure 28 – Schematic Plan view of soil propping

Figure 29 Propping examples

Figure 30 Examples of excavations of pins

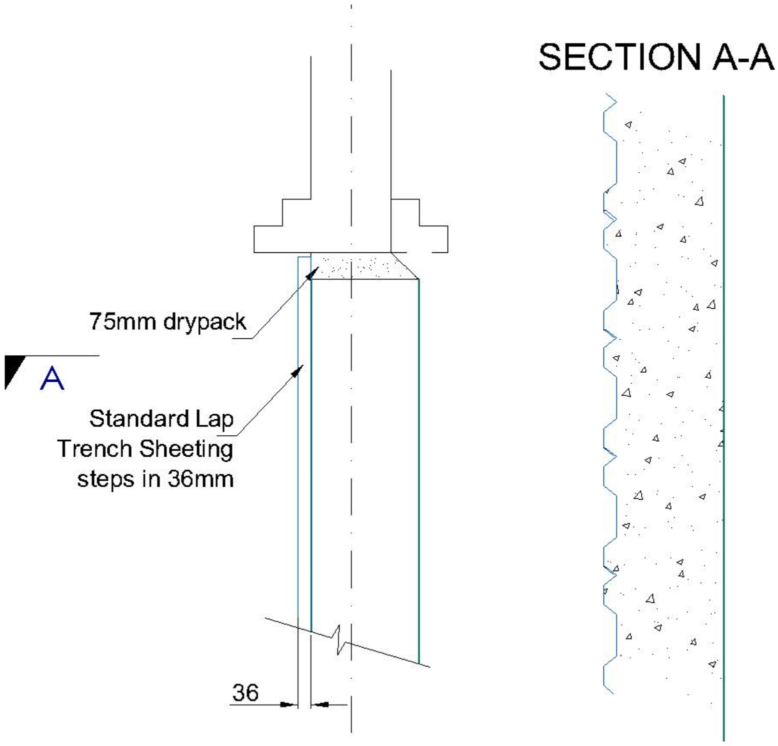
Figure 31 Examples of completed walls and back propping to central soil mass

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 as the excavation progresses.


Figure 32 Example of trench sheet back propping

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 trench sheets are left in a slight over spill may occur past the neighbours boundary wall line. Where this slight over spill is not allowed by the Party Wall Surveyors then cement particle board should be used as noted below.



4.5.3. Where soft spots are encountered, leave in trench sheets or alternatively back prop with precast lintels or sacrificial boards. 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 Acrows diagonally back to the ground.

4.5.4. 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.

4.5.5. 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 should be present to prevent delays due to ordering.

4.6. If cut face is not straight, or sacrificial boards noted previously have been used, place a 15mm cement particle board between sacrificial sheets or against the soil prior to casting. Cement particle board is to line up with the adjacent owner's 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 in any condition where overspill onto the adjacent owner's land is possible.

4.7. Excavate base. If soil over is unstable, prop top with PC lintel and sacrificial prop.

4.8. Visually inspect the footings and provide propping to local brickwork. If necessary install sacrificial Acrow, or pit props, and cast into the retaining wall.

4.9. Clear underside of existing footing.

4.10. Local Authority inspection to be carried out for approval of excavation base.

4.11. Place reinforcement for retaining wall base and stem. Drive H16 Bars U-bars into soil along centre line of stem to act as shear ties to adjacent wall underpin.

4.12. Site supervisor to inspect and sign off works before proceeding to next stage.

4.12.1. For pins 1, 3 and 5, inform the engineer five days before the reinforcement is ready, to allow for inspection of the reinforcement prior to casting.

4.13. Cast base. On short stems it is possible to cast base and wall at the same time. <u>It is essential that pokers/vibrators are used to compact concrete.</u>

4.14. Concrete Testing:

4.14.1. For first 3 pins take 4 cubes and test at 7 days then at 14 days and inform engineer of results. Test last cube at 28 days. If cube test results are low then action into concrete specification and placement method must be considered.

4.14.2. If results are good from first three pins, then from the 4th pin onwards take 2 cubes of concrete from every third pin 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.

Ensure that concrete is of sufficient strength; check engineer's specifications

4.14.3. A record of dates for the concrete pouring of each pin must be kept on site.

4.14.4. The location of where cubes were taken and their reference number must be recorded.

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

4.16. Place shuttering and pour concrete for retaining wall. Stop a minimum of 75mm from the underside of existing footing. <u>It is essential that pokers/vibrators are used, hitting shutters is **not** considered adequate.</u>

4.17. 24 hours after pouring the concrete pin, the gap shall be filled using a dry-pack mortar. Ram in dry-pack between the top of the retaining wall and existing masonry.

4.17.1. If gap is greater than 120mm, place a line of engineering bricks to the top of the wall. Dry pack from the engineering bricks to existing masonry.

4.18. After 24 hours, the temporary wall shutters can be removed.

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

4.20. 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

The existing ground floor will be demolished and new timber floor joists will be installed, supported on new steelwork spanning between the newly constructed retaining walls.

6. Approval

6.1. Building Control Officer/Approved Inspector to inspect pin bases and reinforcement prior to casting concrete.

6.2. Contractor to keep list of dates of pins inspected and cast.

6.3. One month after the work is completed, the contractor is to contact Adjoining Party Wall Surveyor to attend site and complete final condition survey and to sign off works.

CROFT STRUCTURAL ENGINEERS

7. 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 be pumped out.

Trench sheets should be placed at regular centres to deal with the ground. It is expected that the soil between the trench sheeting will arch. Looser soil will require 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 No.s trench sheets are used. It is for this design that these calculations have been provided.

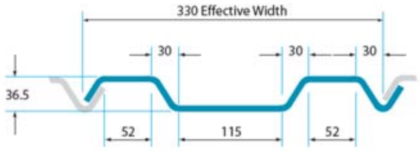
Soil and ground conditions are variable. Typically one finds that, in the temporary condition, clays are more stable and the C_U (cohesive) values in clay reduce the risk of collapse. It is this cohesive nature that allows clays to be cut into a vertical slope. For these calculations, weak sand and gravels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

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

STANDARD LAP TRENCH SHEETING

STANDARD LAP

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



Technical Information

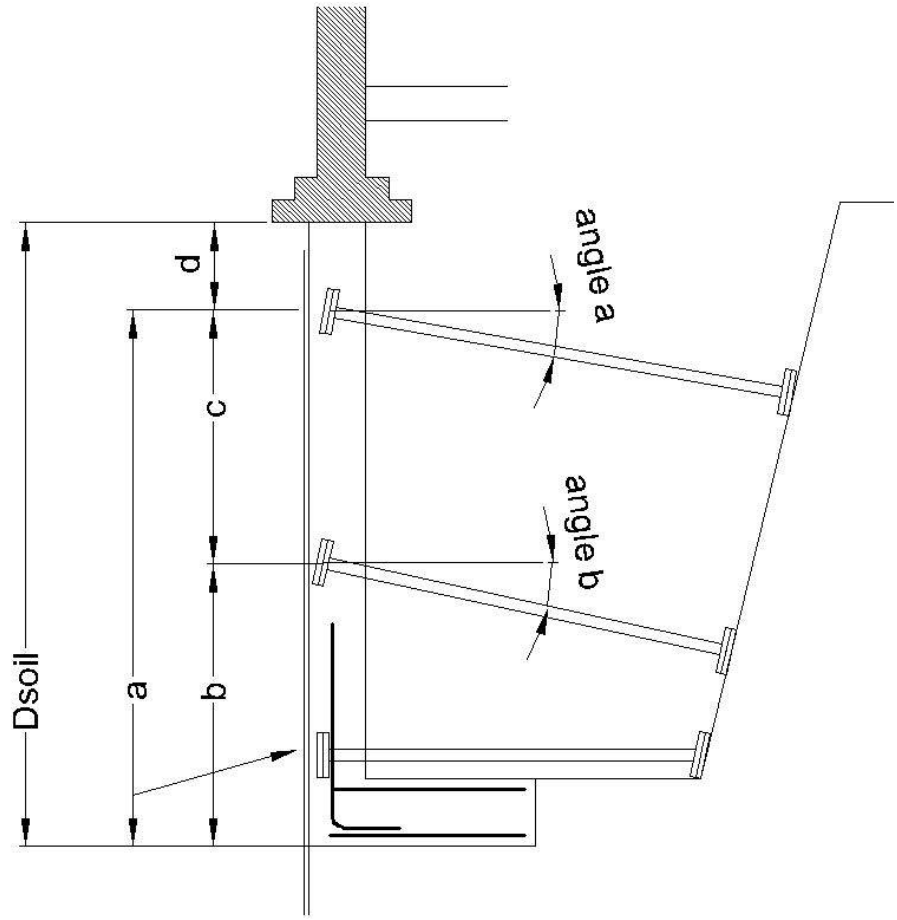
Effective width per sheet (mm)	330
Thickness (mm)	3.4
Depth (mm)	35
Weight per linear metre (kg/m)	10.8
Weight per m² (kg)	32.9
Section modulus per metre width (cm³)	48.3
Section modulus per sheet (cm³)	15.9
I value per metre width (cm⁴)	81.7
I value per sheet (cm⁴)	26.9
Total rolled metres per tonne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

Sxx = 15.9 cm^3

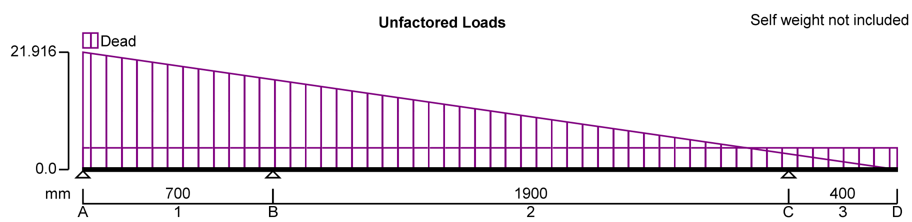
py = 275N/mm^2

Ixx = 26.9cm^4

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



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



<u>CONTINUOUS BEAM ANALYSIS - INPUT</u>

<u>BEAM DETAILS</u>

Number of spans = **3**

<u>Material Properties:</u>

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

<u>Support Conditions:</u>

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

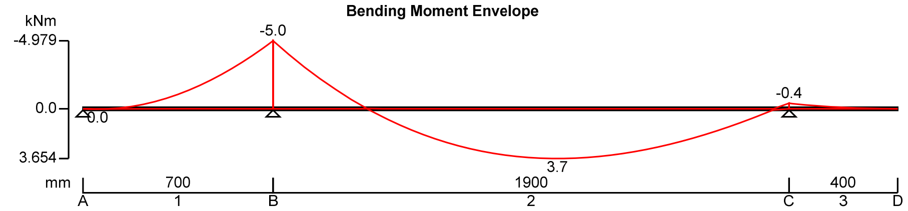
Support A	Max react = **-1.4** kN	Min react = **-1.4** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-32.8** kN	Min react = **-32.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-10.8** kN	Min react = **-10.8** kN	Max mom = **0.0** kNm	Min mom = **0.0**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

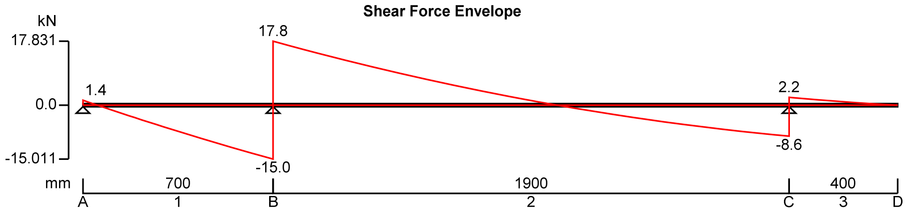
Beam Max/Min results - Combination Summary

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





Number of sheets; Nos = 2

Moment ; M_allowable = Sxx * py * Nos = **8.745**kNm

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

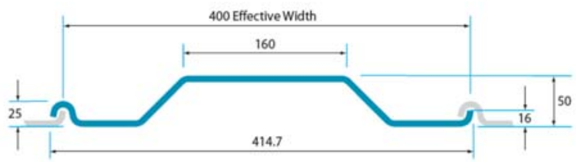
Any Acrow Prop is acceptable

CROFT STRUCTURAL ENGINEERS

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

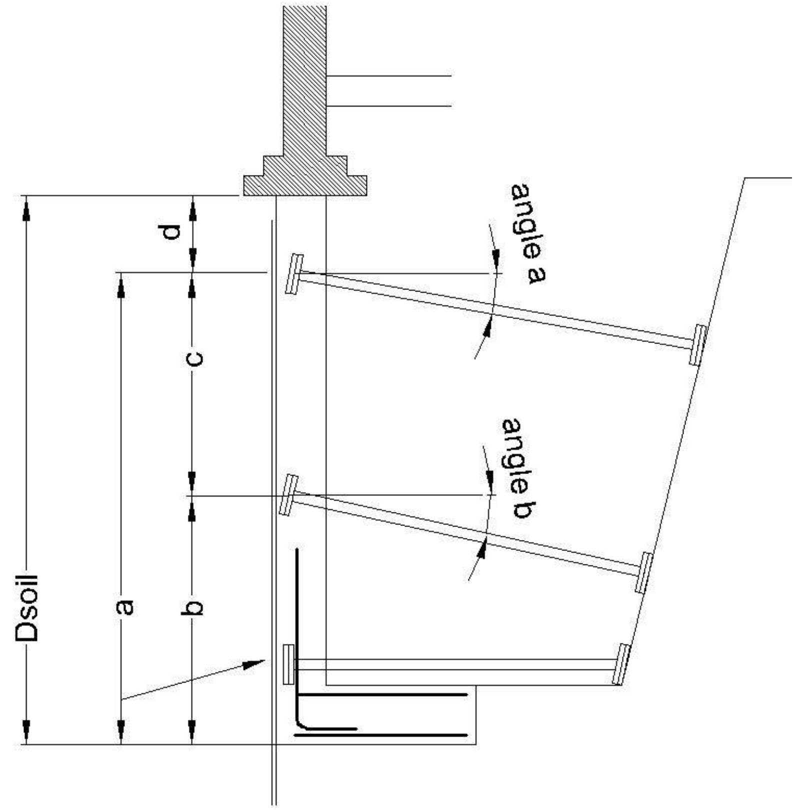
Effective width per sheet (mm)	400
Thickness (mm)	6.0
Depth (mm)	50
Weight per linear metre (kg/m)	21.90
Weight per m^2 (kg)	55.2
Section modulus per metre width (cm^3)	101
Section modulus per sheet (cm^3)	40.34
I value per metre width (cm^4)	250
I value per sheet (cm^4)	101
Total rolled metres per tonne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

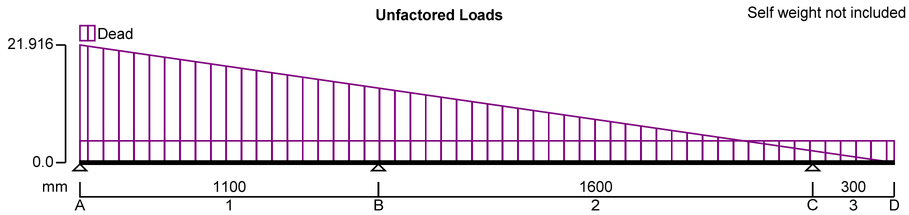


Length a a = **2.700** m

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.×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

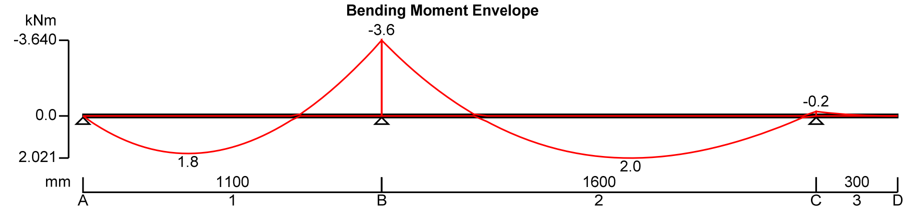
Support A	Max react = **-9.5** kN	Min react = **-9.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-28.0** kN	Min react = **-28.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-7.5** kN	Min react = **-7.5** kN	Max mom = **0.0** kNm	Min mom = **0.0**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

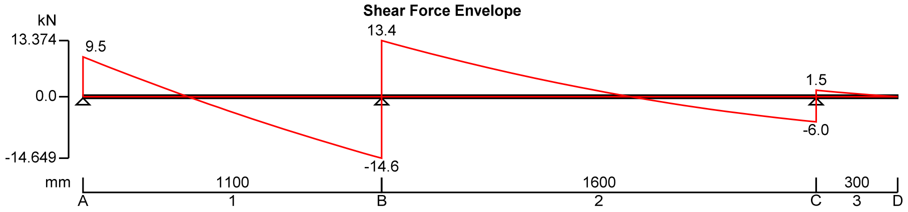
Beam Max/Min results - Combination Summary

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

Moment ; M_allowable = Sxx * py * Nos = **26.565**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 2.0 ft 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

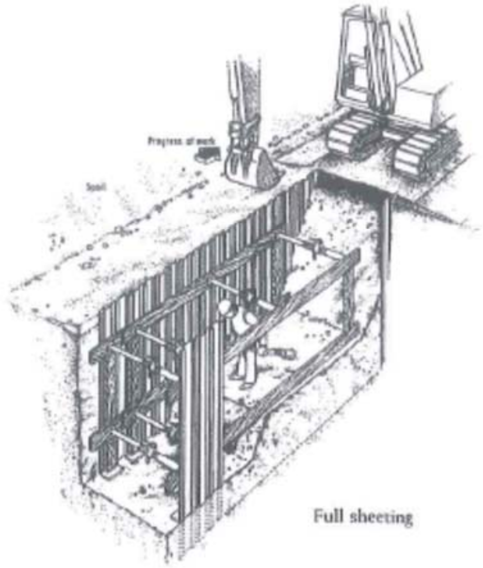
Shear; V = (14.6kN + 13.4kN) /2 = 14.000kN

Any Acrow Prop is acceptable

Sheeting requirements

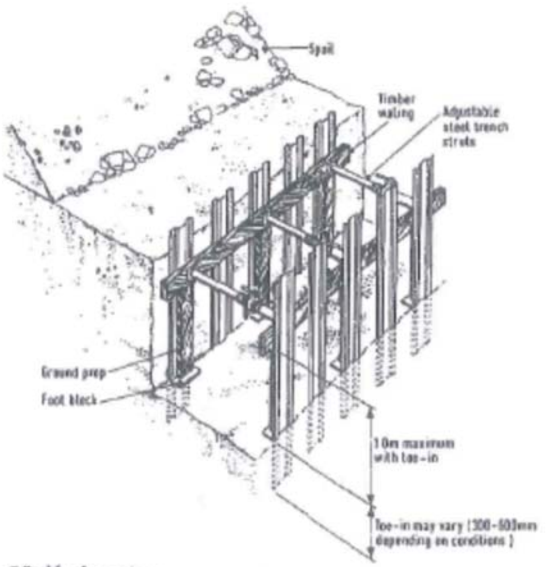
Ground Type	Trench Depth, D: ~~less than 1.2m[1]~~	1.2 to 3m	3 to 4.5m	4.5 to 6m
Sands and gravels Silt Soft Clay High compressibility Peat	~~Close, ½, ¼, ⅛ or nil~~	Close	Close	Close
Firm/stiff Clay Low compressibility Peat	~~¼, ⅛ or nil~~	½ or ¼	½ or ¼	Close or ½
Rock[2]	From ½ for incompetent rock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

Sheeting requirements



Full sheeting

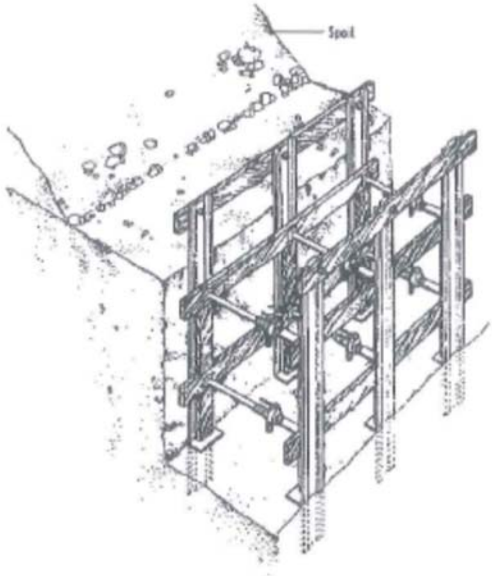
Sheeting requirements



Half sheeting
shown for 1.5 m deep trench

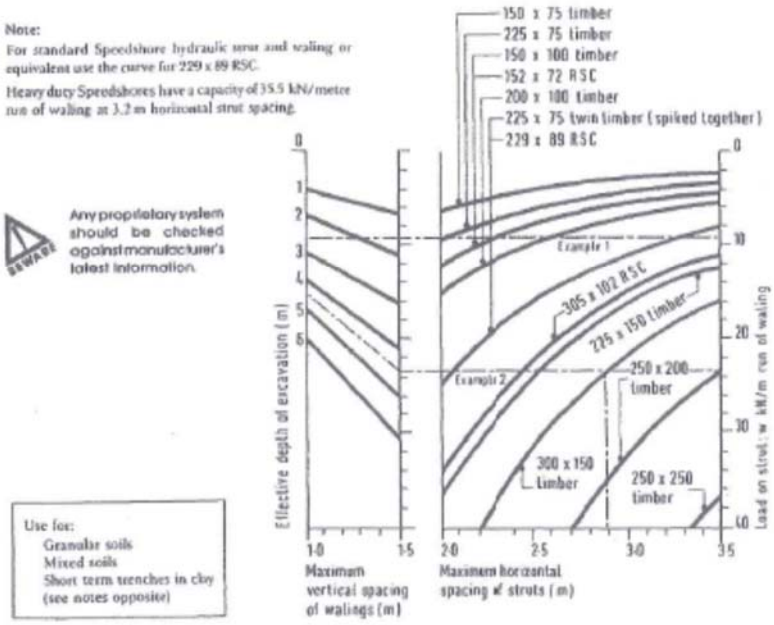
11/04/2011

Sheeting requirements



Quarter sheeting

Design to CIRIA 97



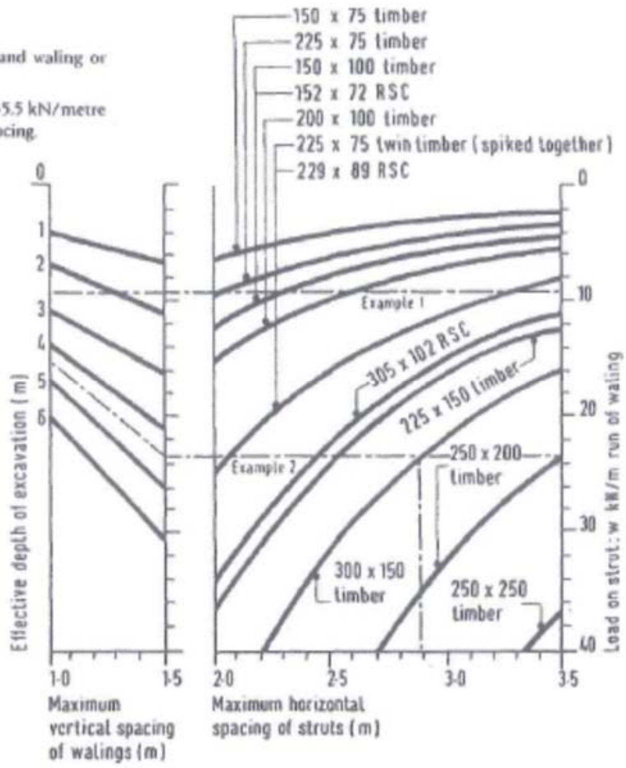
Note:

For standard Speedshore hydraulic strut and waling or equivalent use the curve for 229 x 89 RSC.

Heavy duty Speedshores have a capacity of 35.5 kN/metre run of waling at 3.2 m horizontal strut spacing.

BEWARE

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

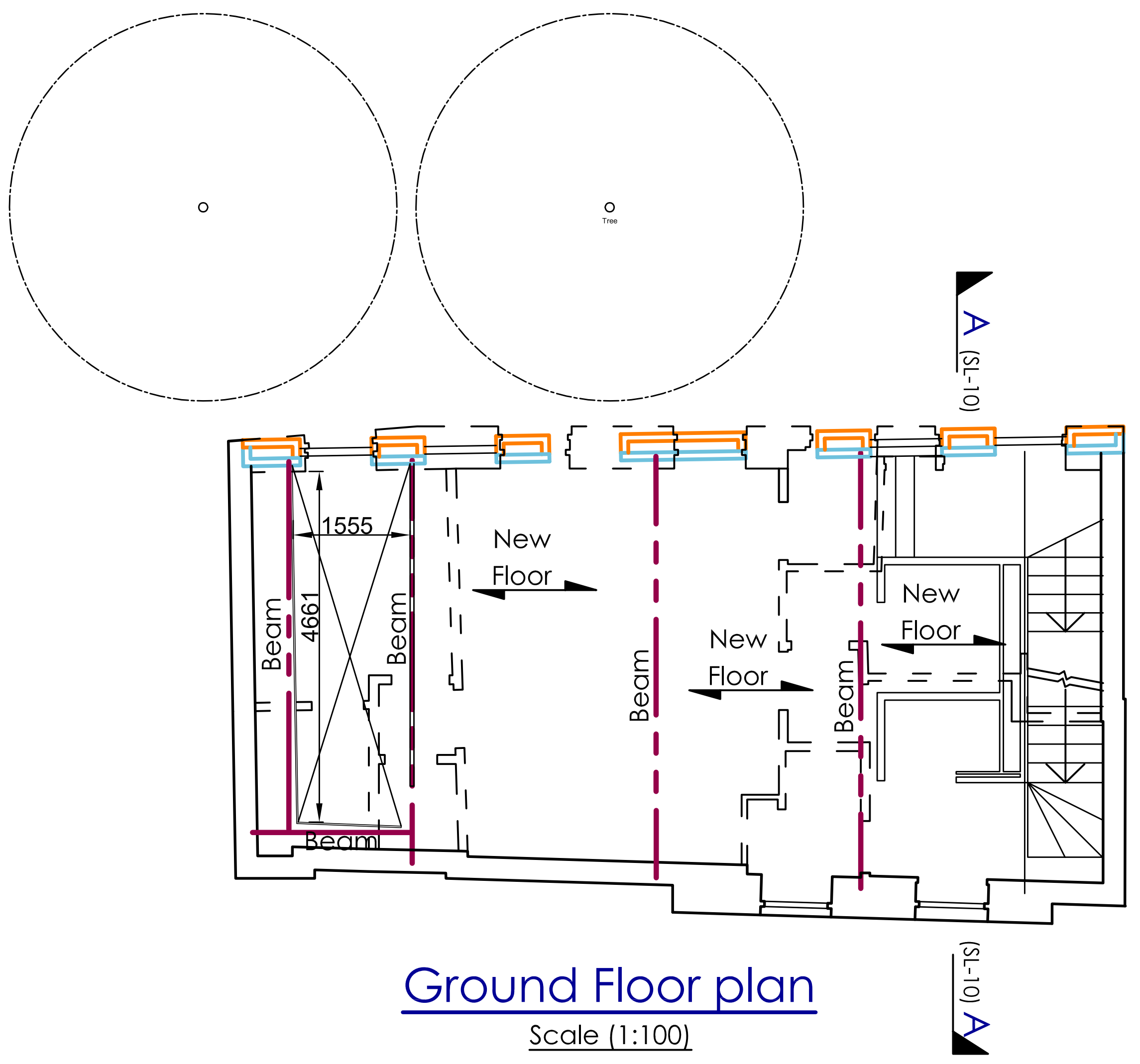


Use for:
- Granular soils
- Mixed soils
- Short term trenches in clay (see notes opposite)

Appendix C : Structural Drawings

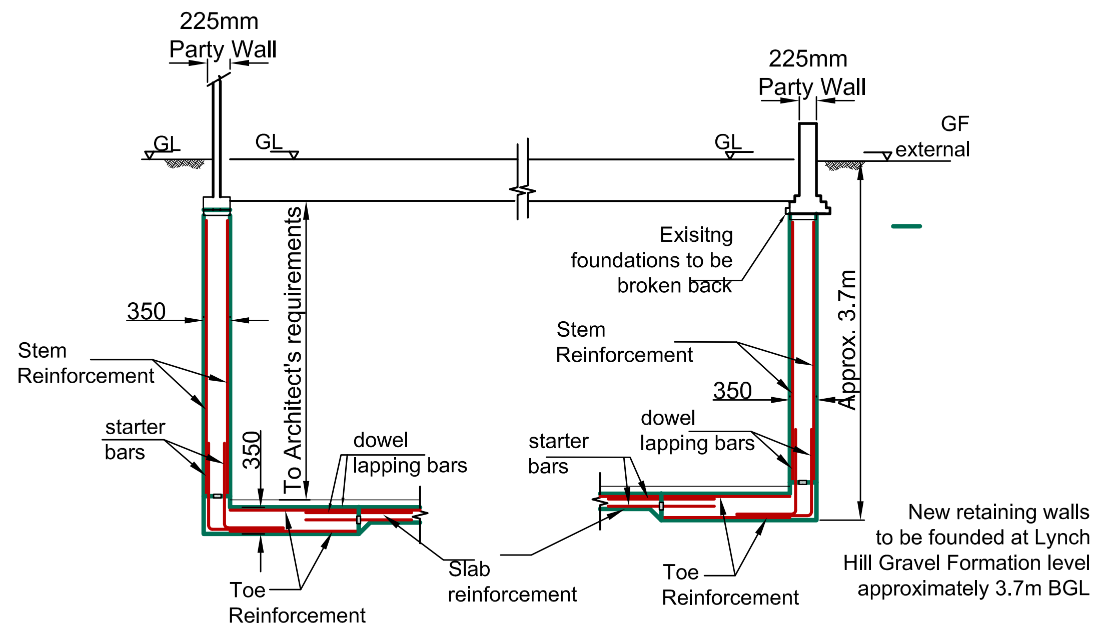
1:100 Basement Plan on A3 Showing Neighbouring basements if present
1:100 Ground Floor plan on A3 Showing Neighbouring property
1:100 Section on A3 Including section through Neighbouring Footings

Tree Plan on A3



Ground Floor plan

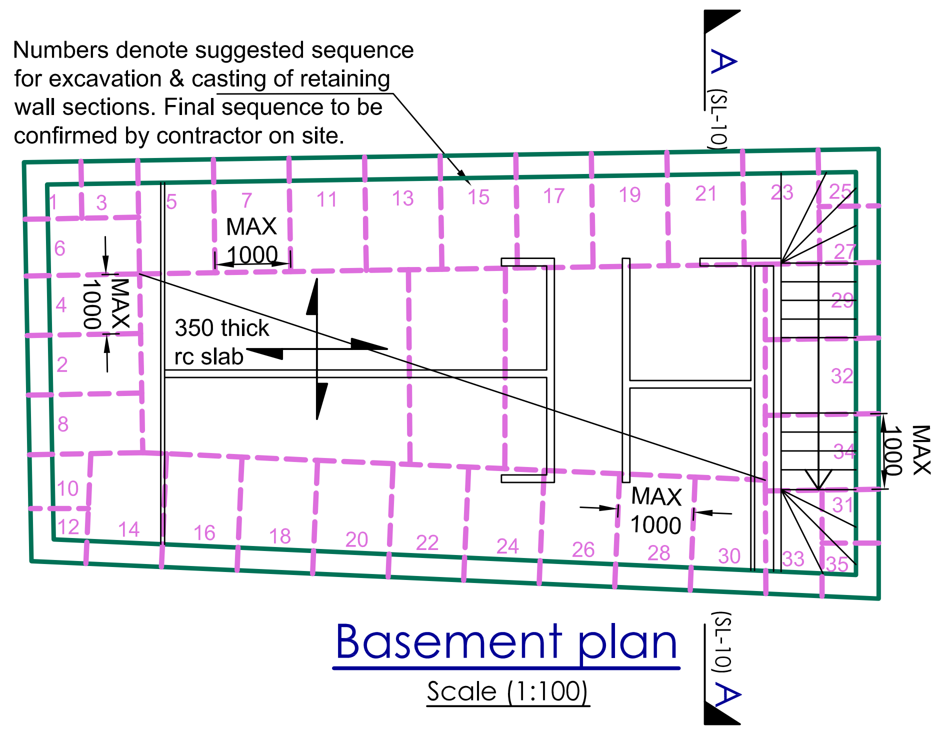
Scale (1:100)



Section A-A

Scale (1:100)

FOR PLANNING



Basement plan

Scale (1:100)

Rev	Date	Amendments
-	28-04-17	First issue for comment

Job nos: 150912	Client: Derek Savage
Dwg Nos: SL-10	Project: 6 Stuckley Street
date: Oct'15; drawn: EP; Chk'd: CT	Title : Basement and Ground Floor Plan and section
Scale: 1:100 @ A3; Rev: -	

Croft
Structural
Engineers

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

CROFT STRUCTURAL ENGINEERS

Appendix D : Monitoring Statement

8. Introduction

Basement works are intended to 6 Stukeley Street. The structural works for this require Party Wall Awards. This statement describes the procedures for the Principal Contractor to follow to observe any movement that may occur to the existing properties, and also describes mitigation measures to apply if necessary.

9. Risk Assessment

The purpose of this risk assessment is to consider the impact of the proposed works and how they impact the party wall. There are varying levels of inspection that can be undertaken and not all works, soil conditions and properties require the same level of protection.

Monitoring Level Proposed	Type of Works.
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

10. Scheme Details

This document has been prepared by Croft Structural Engineers Ltd. It covers the proposed construction of a new basement underneath the existing structure at 6 Stukeley Street

Scope of Works

The works comprise:

- Visual Monitoring of the party wall
- Attachment of Tell tales or Demec Studs to accurately record movement of significant cracks.
- Attachment of levelling targets to monitor settlement.
- The monitoring of the above instrumentation is in accordance with Appendix A. The number and precise locations of instrumentation may change during the works; this shall be subject to agreement with the Principal Contractor (PC).
- All instruments are to be adequately protected against any damage from construction plant or private vehicles using clearly visible markings and suitable head protection e.g. manhole rings or similar. Any damaged instruments are to be immediately replaced or repaired at the contractors own cost.

- Reporting of all data in a manner easily understood by all interested parties.
- Co-ordination of these monitoring works with other site operations to ensure that all instruments can be read and can be reviewed against specified trigger values both during and post construction.
- Regular site meetings by the Principal Contractor (PC) and the Monitoring Surveyor (MS) to review the data and their implications.
- Review of data by Croft Structural Engineers

In addition, the PC will have responsibility for the following:

- Review of methods of working/operations to limit movements, and
- Implementation of any emergency remedial measures if deemed necessary by the results of the monitoring.

The Monitoring Surveyor shall allow for settlement and crack monitoring measures to be installed and monitored on various parts of the structure described in Table 1 as directed by the PC and Party Wall Surveyor (PWS) for the Client.

Item	Instrumentation Type
Party Wall Brickwork	
Settlement monitoring	Levelling equipment & targets
Crack monitoring	Visual inspection of cracking, Demec studs where necessary

Table 1: Instrumentation

General

The site excavations and substructure works up to finished ground slab stage have the potential to cause vibration and ground movements in the vicinity of the site due to the following:

a) Removal of any existing redundant foundations / obstructions;
b) Installation of reinforced concrete retaining walls under the existing footings;
c) Excavations within the site

The purpose of the monitoring is a check to confirm building movements are not excessive.

This specification is aimed at providing a strategy for monitoring of potential ground and building movements at the site.

This specification is intended to define a background level of monitoring. The PC may choose to carry out additional monitoring during critical operations. Monitoring that should be carried out is as follows:

a) Visual inspection of the party wall and any pre-existing cracking
b) Settlement of the party wall

All instruments are to be protected from interference and damage as part of these works.

Access to all instrumentation or monitoring points for reading shall be the responsibility of the Monitoring Surveyor (MS). The MS shall be in sole charge for ensuring that all instruments or monitoring points can be read at each visit and for reporting of the data in a form to be agreed with the PWS. He shall inform the PC if access is not available to certain instruments and the PC will, wherever possible, arrange for access. He shall immediately report to the PC any damage. The Monitoring Surveyor and the Principal Contractor will be responsible for ensuring that all the instruments that fall under their respective remits as specified are fully operational at all times and any defective or damaged instruments are immediately identified and replaced.

The PC shall be fully responsible for reviewing the monitoring data with the MS - before passing it on to Croft Structural Engineers - determining its accuracy and assessing whether immediate action is to be taken by him and/or other contractors on site to prevent damage to instrumentation or to ensure safety of the site and personnel. All work shall comply with the relevant legislation, regulations and manufacturer's instructions for installation and monitoring of instrumentation.

Applicable Standards and References

The following British Standards and civil engineering industry references are applicable to the monitoring of ground movements related to activities on construction works sites:

1. BS 5228: Part 1: 1997 - Noise and Vibration Control on Construction and Open Sites -Part 1.Code of practice for basic information and procedures for noise and vibration control, Second Edition, BSI 1999.
2. BS 5228: Part 2: 1997 - Noise and Vibration Control on Construction and Open Sites -Part 2.Guide to noise and vibration control legislation for construction and demolition including road construction and maintenance, Second Edition, BSI 1997.
3. BS 7385-1: 1990 (ISO 4866:1990) - Evaluation and measurement for vibration in buildings - Part 1: Guide for measurement of vibrations and evaluation of their effects on buildings, First Edition, BSI 1990.
4. BS 7385-2: 1993 - Evaluation and measurement for vibration in buildings - Part 2: Guide to damage levels from ground-borne vibration, First Edition, BSI 1999.
5. CIRIA SP 201 - Response of buildings to excavation-induced ground movements, CIRIA 2001.

SPECIFICATION FOR INSTRUMENTATION

General

The Monitoring Contractor is required to monitor, protect and reinstall instruments as described. The readings are to be recorded and reported. The following instruments are defined:

a) Automatic level and targets: A device which allows the measurement of settlement in the vertical axis. To be installed by the MS.

b) Tell-tales and 3 stud sets: A device which allows measurement of movement to be made in two axes perpendicular to each other. To be installed by the MS.

Monitoring of existing cracks

The locations of tell-tales or Demec studs to monitor existing cracks shall be agreed with Croft Structural Engineers.

Instrument Installation Records and Reports

Where instrumentation is to be installed or reinstalled, the Monitoring Surveyor, or the Principal Contractor, as applicable, shall make a complete record of the work. This should include the position and level of each instrument. The records shall include base readings and measurements taken during each monitoring visit. Both tables and graphical outputs of these measurements shall be presented in a format to be agreed with the CM. The report shall include photographs of each type of instrumentation installed and clear scaled sections and plans of each instrument installed. This report shall also include the supplier's technical fact sheet on the type of instrument used and instructions on monitoring.

Two signed copies of the report shall be supplied to the PWS within one week of completion of site measurements for approval.

Installation

All instruments shall be installed to the satisfaction of the PC. No loosening or disturbance of the instrument with use or time shall be acceptable. All instruments are to be clearly marked to avoid damage.

All setting out shall be undertaken by the Monitoring Surveyor or the Principal Contractor as may be applicable. The precise locations will be agreed by the PC prior to installation of the instrument.

The installations are to be managed and supervised by the Instrumentation Engineer or the Measurement Surveyor as may be applicable.

Monitoring

The frequencies of monitoring for each Section of the Works are given in Appendix A.

The following accuracies/ tolerances shall be achieved:

Party Wall settlement	$\pm$1.5mm
Crack monitoring	$\pm$0.75mm

CROFT STRUCTURAL ENGINEERS

REPORT OF RESULTS AND TRIGGER LEVELS

General

Within 24 hours of taking the readings, the Monitoring Surveyor will submit a single page summary of the recorded movements. All readings shall be immediately reviewed by Croft Structural Engineers prior to reporting to the PWS.

Within one working day of taking the readings the Monitoring Contractor shall produce a full report (see below).

The following system of control shall be employed by the PC and appropriate contractors for each section of the works. The Trigger value, at which the appropriate action shall be taken, for each section, is given in Table 2, below.

The method of construction by use of sequential underpins limits the deflections in the party wall.

Below are the trigger limits

During works measurements are taken, these are compared with the limits set out below:

MOVEMENT		CATEGORY	ACTION
Vertical	Horizontal		
0mm-4mm	0-3mm	Green	No action required
4mm-7mm	3-5mm	AMBER	Detailed review of Monitoring: Check studs are OK and have not moved. Ensure site staff have not moved studs. If studs have moved reposition. Relevel to ensure results are correct and tolerance is not a concern. Inform Party Wall surveyors of amber readings. Double the monitoring for 2 further readings. If stable revert back. Carry out a local structural review and inspection. Preparation for the implementation of remedial measures should be required. Double number of lateral props Implement remedial measures review method of working and ground conditions
>7mm	>5mm	RED	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

Table 2 – Movement limits between adjacent sets of Tell-tales or stud sets

Any movements which exceed the individual amber trigger levels for a monitoring measure given in Table 2 shall be immediately reported to the PWS, and a review of all of the current monitoring data for all monitoring measures must be implemented to determine the possible causes of the trigger level being exceeded. Monitoring of the affected location must be increased and the actions described above implemented. Assessment of exceeded trigger levels must not be carried out in isolation from an assessment of the entire monitoring regime as the monitoring measures are inter-related. Where required, measures may be implemented or prepared as determined by the specific situation and combination of observed monitoring measurement data.

Standard Reporting

1 No. electronic copy of the report in PDF format shall be submitted to the PWS.

The Monitoring Surveyor shall report whether the movements are within (or otherwise) the Trigger Levels indicated in Table 2. A summary of the extent of completion of any of the elements of works and any other significant events shall be given. These works shall be shown in the form of annotated plans (and sections) for each survey visit both local to the instrumentation and over a wider area. The associated changes to readings at each survey or monitoring point shall be then regulated to the construction activity so that the cause of any change, if it occurs, can be determined.

The Monitoring Surveyor shall also give details of any events on site which in his opinion could affect the validity of the results of any of the surveys.

The report shall contain as a minimum, for each survey visit the following information:

a) The date and time of each reading:
b) The weather on the day:
c) The name of the person recording the data on site and the person analysing the readings together with their company affiliations;
d) Any damage to the instrumentation or difficulties in reading;
e) Tables comparing the latest reading with the last reading and the base reading and the changes between these recorded data;
f) Graphs showing variations in crack width with time for the crack measuring gauges; and
g) Construction activity as described. It is very important that each set of readings is associated with the extent of excavation and construction at that time. Readings shall be accompanied by information describing the extent of works at the time of readings. This shall be agreed with the PC.

Spread-sheet columns of numbers should be clearly labelled together with units. Numbers should not be reported to a greater accuracy than is appropriate. Graph axis should be linear and clearly labelled together with units. The axis scales are to be agreed with the PC before the start of monitoring and are to remain constant for the duration of the job unless agreed otherwise. The specified trigger values are also to be plotted on all graphs.

The reports are to include progress photographs of the works both general to the area of each instrument and globally to the main Works. In particular, these are to supplement annotated plans/sections described above. Wherever possible the global photographs are to be taken from approximately the same spot on each occasion. The locations of these points on site are to be Croft Structural Engineers drawing M-PL-01.

Erroneous Data

All data shall be checked for errors by the Monitoring Surveyor prior to submission. If a reading that appears to be erroneous (i.e. it shows a trend which is not supported by the surrounding instrumentation), he shall notify the PC immediately, resurvey the point in question and the neighbouring points and if the error is repeated, he shall attempt to identify the cause of the error. Both sets of readings shall be processed and submitted, together with the reasons for the errors and details of remedial works. If the error persists at subsequent survey visits, the Monitoring Surveyor shall agree with the PC how the data should be corrected. Correction could be achieved by correcting the readings subsequent to the error first being identified to a new base reading.

The Monitoring Surveyor shall rectify any faults found in or damage caused to the instrumentation system for the duration of the specified monitoring period, irrespective of cause, at his own cost.

Trigger Values

Trigger values for maximum movements as listed in Table 2. If the movement exceeds these values then action may be required to limit further movement. The PC should be immediately advised of the movements in order to implement the necessary works.

It is important that all neighbouring points (not necessarily a single survey point) should be used in assessing the impact of any movements which exceed the trigger values, and that rechecks are carried out to ensure the data is not erroneous. A detailed record of all activities in the area of the survey point will also be required as specified elsewhere.

Responsibility for Instrumentation

The Monitoring Surveyor shall be responsible for: managing the installation of the instruments or measuring points, reporting of the results in a format which is user friendly to all parties; and immediately reporting to all parties any damage. The Monitoring Surveyor shall be responsible for informing the PC of any movements which exceed the specified trigger values listed in Table 2 so that the PC can implement appropriate procedures. He shall immediately inform the PWS of any decisions taken.

CROFT STRUCTURAL ENGINEERS

APPENDIX A
MONITORING FREQUENCY

INSTRUMENT	FREQUENCY OF READING
Settlement monitoring and Monitoring existing cracks	**Pre-construction** Monitored once. **During construction** Monitored after every pin is cast for first 4 no. pins to gauge effect of underpinning. If all is well, monitor after every other pin. **Post construction works** Monitored once.

CROFT STRUCTURAL ENGINEERS

APPENDIX B

An Analysis on allowable settlements of structures (Skempton and MacDonald (1956))

The most comprehensive studies linking self-weight settlements of buildings to structural damage were carried out in the 1950's by Skempton and MacDonald (1956) and Polshin and Tokar. These studies show that damage is most often caused by differential settlements rather than absolute settlements. More recently, similar empirical studies by Boscardin and Cording (1989) and Boone (1996) have linked structural damage to ground movements induced by excavations and tunnelling activities.

In 1955 Skempton and MacDonald identified the parameter $\delta\rho/L$ as the fundamental element on which to judge maximum admissible settlements for structures. This criterion was later confirmed in the works of GRANT *et al.* [1975] and WALSH [1981]. Another important approach to the problem was that of BURLAND and WROTH [1974], based on the criterion of maximum tensile strains.

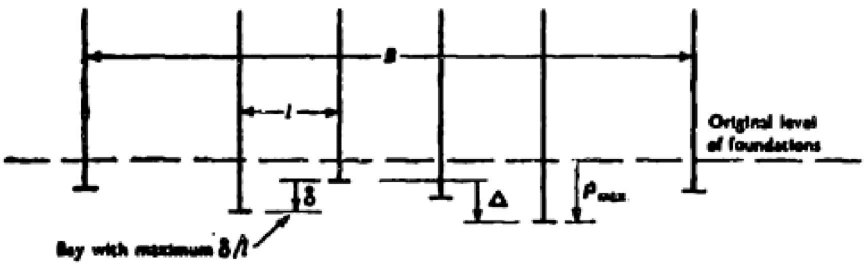


Figure 2.1 – Diagram illustrating the definitions of maximum angular distortion, δ/l, maximum settlement, ρ_{max}, and greatest differential settlement, Δ, for a building with no tilt (Skempton and MacDonald, 1956).

Figure 33: Diagram illustrating the definitions of maximum angular distortion, δ/l, maximum settlement, ρ_{max}, and greatest differential settlement , Δ , for a building with no tilt (Skempton and MacDonald, 1956)

The differential settlement is defined as the greatest vertical distance between two points on the foundation of a structure that has settled, while the angular distortion, is the difference in elevation between two points, divided by the distance between those points.

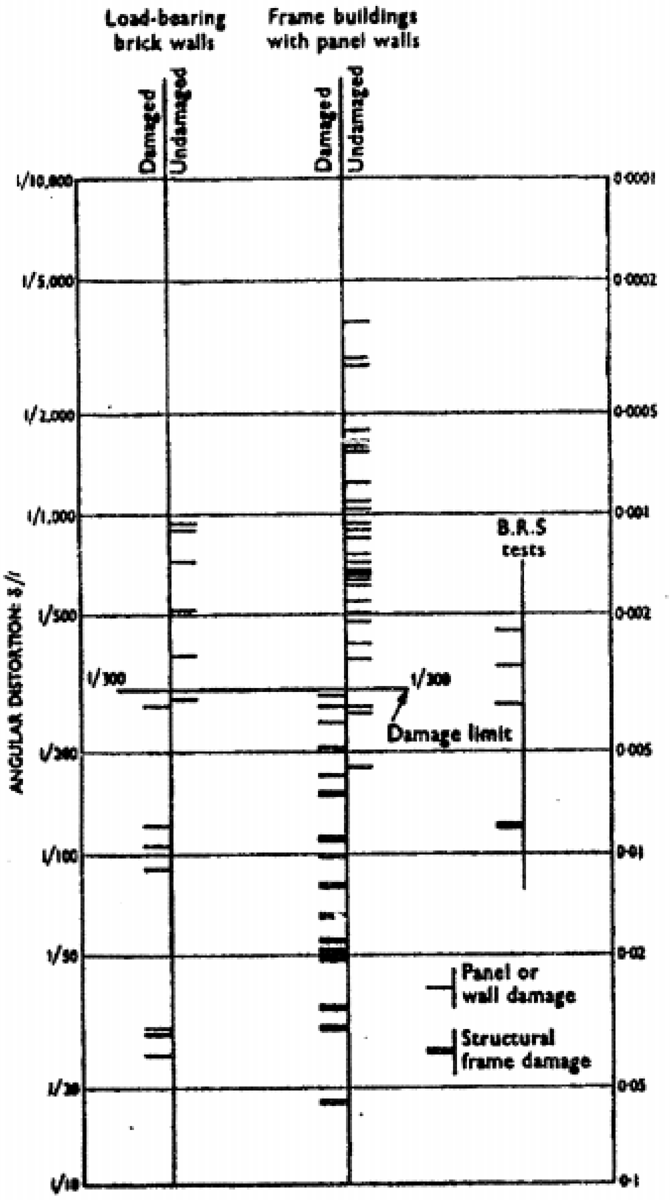
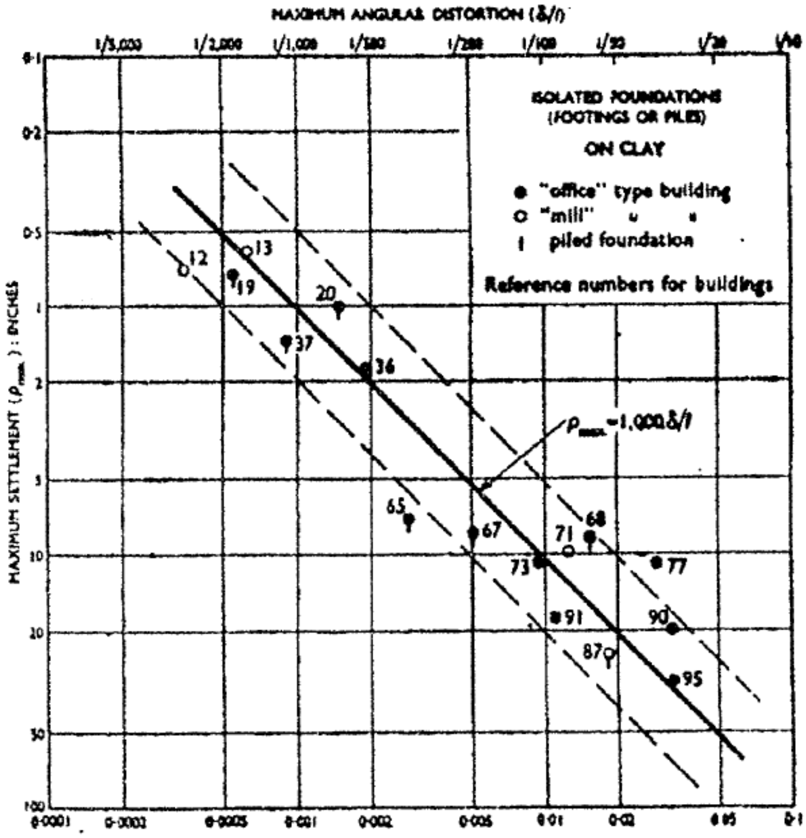
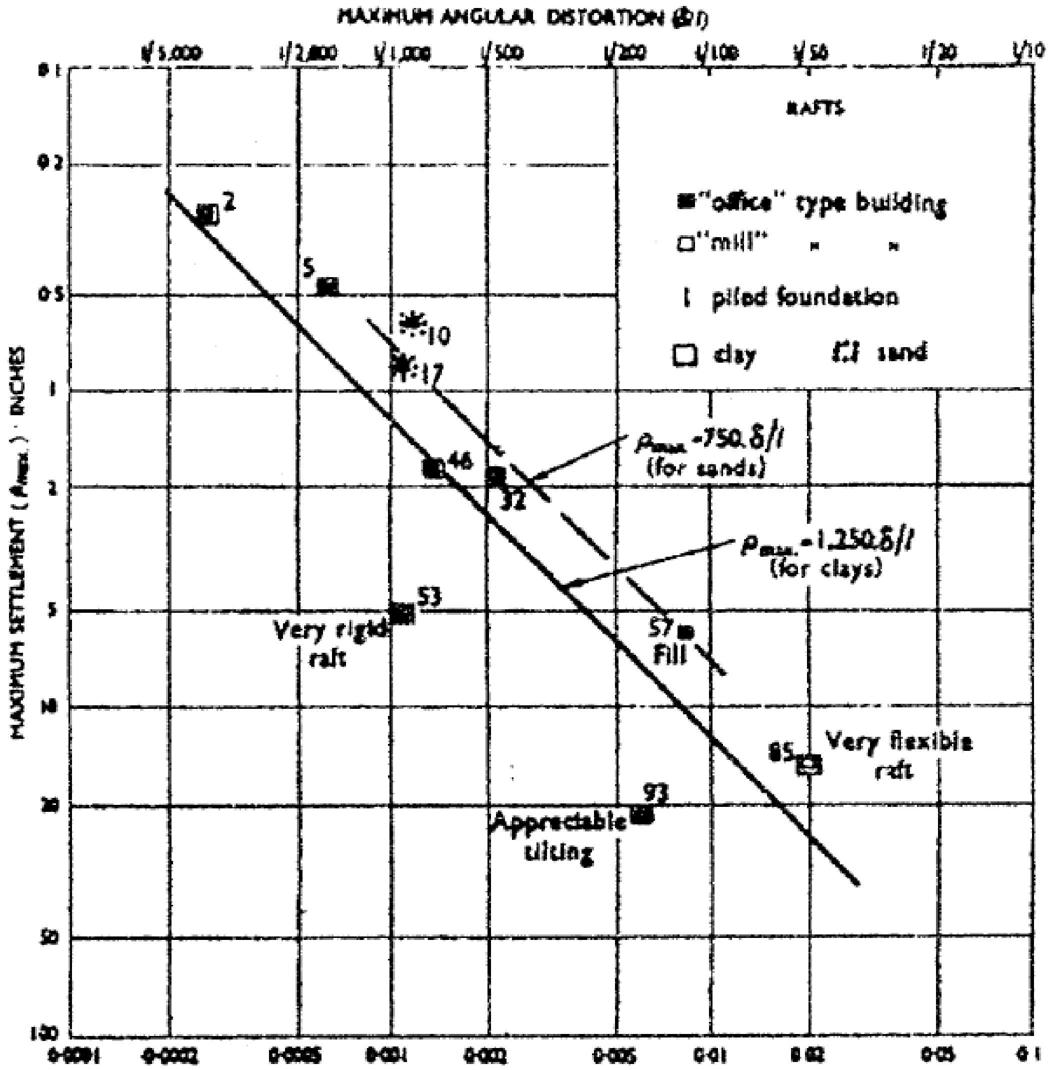


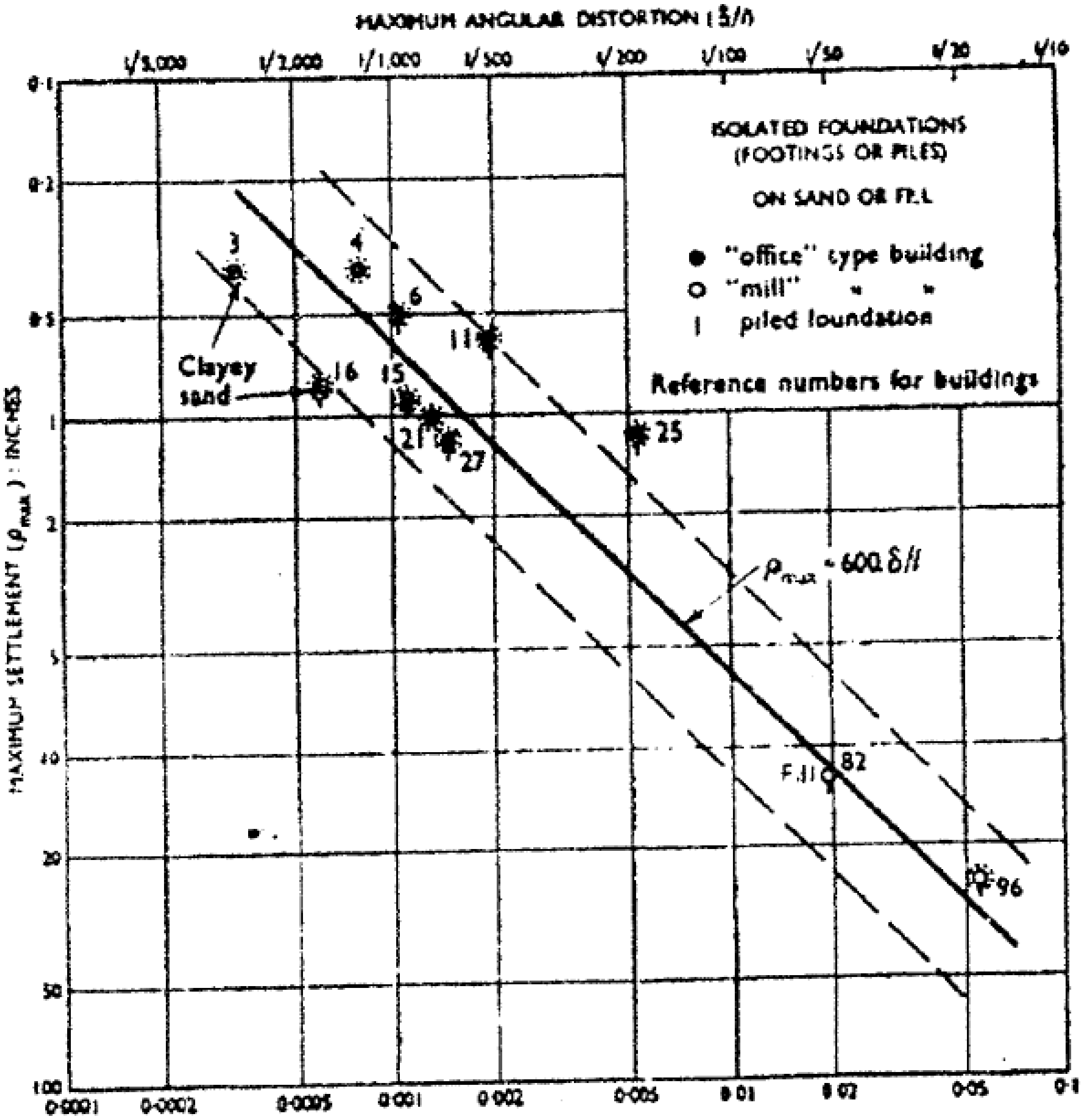
Figure 34: Skempton and MacDonald's analysis of field evidence of damage on traditional frame buildings and loadbearing brick walls

Data from Skempton and MacDonald's work suggest that the limiting value of angular distortion is 1/300. Angular distortion, greater than 1/300 produced visible cracking in the majority of buildings studied, regardless of whether it was a load bearing or a frame structure. As shown in the figure 2.

Other key findings by Skempton and MacDonald include limiting values of δ/l for structure, and a relationship between maximum settlement, ρ_{max} and δ/l for structures founded on sands and clays. The charts below show these relations for raft foundations and isolated footings.




MAXIMUM ANGULAR DISTORTION (δ/l)
1/5,000
1/2,800
1/1,000
1/500
1/200
1/100
1/50
1/20
1/10
RAFTS
"office" type building
"mill"
piled foundation
clay
sand
$\rho_{max.} = 750\,\delta/l$ (for sands)
$\rho_{max.} = 1{,}250\,\delta/l$ (for clays)
Very rigid raft
Fill
Very flexible raft
Appreciable tilting
MAXIMUM SETTLEMENT ($\rho_{max.}$) · INCHES
0·0001
0·0002
0·0005
0·001
0·002
0·005
0·01
0·02
0·05
0·1


MAXIMUM ANGULAR DISTORTION (δ/l)
1/5,000
1/2,000
1/1,000
1/500
1/200
1/100
1/50
1/20
1/10
ISOLATED FOUNDATIONS (FOOTINGS OR PILES)
ON SAND OR FILL
"office" type building
"mill"
piled foundation
Reference numbers for buildings
Clayey sand
ρmax = 600 δ/l
MAXIMUM SETTLEMENT (ρmax) : INCHES
0·1
0·2
0·5
1
2
5
10
20
50
100
0·0001
0·0002
0·0005
0·001
0·002
0·005
0·01
0·02
0·05
0·1

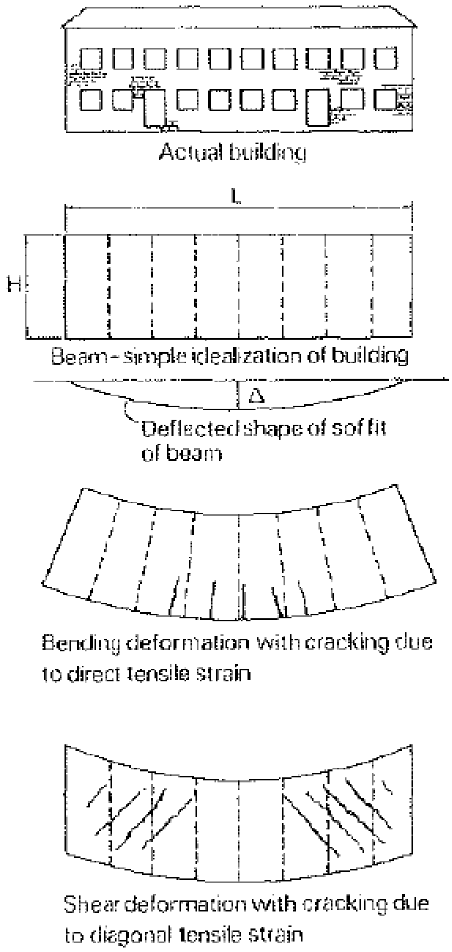
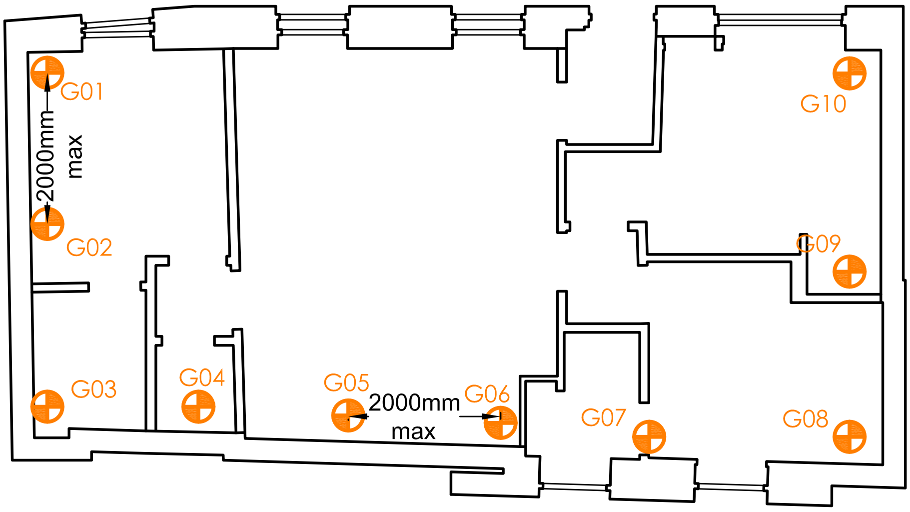


TABLE I

Angular distorsion	Characteristic situation
1/300	Cracking of the panels in frame buildings of the traditional type, or of the walls in load-bearing wall buildings;
1/150	Structural damage to the stanchions and beams;
1/500	Design limit to avoid cracking;
1/1000	Design limit to avoid any settlement damage.

Croft Structural Engineers	Clockshop Mews, r/o 60 Saxon Rd, London, SE25 5EH. 020 8684 4744 www.croftse.co.uk	Project: 6 Stukeley Street Title : Basement Monitoring	Job nos 150912 Dwg/Calc Nos M-PL-01 Rev - drawn EP Chk'd	Scale 1:100 date 28-04-17



Ground floor monitoring plan

Scale (1:100)

FOR PLANNING

MONITORING: = Monitoring point (see monitoring method statement for details)

Vertical movement along wall	Horizontal movement along wall	Category	Action (see monitoring statement for further details)
0 - 4mm	0 - 3mm	Green	• Carry on with works
4 - 7mm	3 - 5mm	Amber	• Carry out a local structural review; • Prepare for the implementation of remedial measures should they be required.
> 7mm	> 5mm	Red	• Implement structural support, as required; • Cease works; • Review monitoring data and implement revised method of works; • Inform Croft Structural Engineers

Key

Denotes position of Leveling Targets, fixed to party wall 500mm & 2000mm above Ground Floor Level.Additional monitoring may be required for any cracking noted in the Party Wall Surveyor's survey.