Job Number: 150333

CROFT STRUCTURAL ENGINEERS

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Basement Impact Assessment

Property Details

The Coach House
98A Priory Road
West Hampstead
Camden
NW6 3NT

Client Information

Michael Cohen
9 Spaniards Close
NW11 6TH

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

Hydrogeology Report	Land Stability Report
(Separate Report) Hannah Fraser CGeol	(Separate Report) Jon Smithson CGeol

Revision	Date	Comment
-	23/06/2015	First Issue
1	23/09/2015	Alterations in Summary, Section 4 and Appendix A

Contents

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Executive (non-technical) Summary

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

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

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

Project Summary

Description of Property

The existing property is a two-storey building, which has a courtyard and a conservatory to the rear and a driveway at the front.

Proposed Works

The proposed works require:

- A basement under the property.
- Light wells to the front and rear
- Superstructure works above the basement
  - Demolition of the conservatory
  - An extension to the side and rear of the property
  - A new floor at second floor level

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

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

1. Excavate front to allow for conveyor to be erected.

	2. Safely and securely support the existing building above • 3. Safely and securely support the excavations. This must include propping at high level. 4. Form lightwell with cantilevered retaining walls 5. Slowly work from the front to the rear inserting narrow cantilevered retaining walls sequentially using well developed and understood underpinning methods. 6. Prop retaining walls in temporary condition back to the central soil 'dumpling'. 7. Prop across the width of the basement, excavate central soil 'dumpling' and cast basement slab. 8. Waterproof internal space with a drained cavity system. Drainage, stability and potential ground movements are addressed towards the end of Section 4.
Stage 1 – Screening	Screening identified areas of concern and concluded a requirement to proceed to a scoping stage for the potential impacts relating to Land Stability, Hydrogeology, Surface Water and Flooding.
Stage 2 – Scoping	The Scoping stage identified the potential impacts and set the parameters required for further studying of the areas of concern that were 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 structural engineer inspected the building to determine the current condition of the property. Visual inspections were completed of the adjacent properties to determine if there were signs of structural movement. The neighbouring land has been excavated: there are basements below the neighbouring properties on the same street. This information was obtained from planning drawings available from Camden Council's website.

<table>
<tr><td></td><td>A ground investigation with 7m deep boreholes was completed.
• The formation level of the basement will be in London clay
• Initial standpipe readings indicated a water table at 4.2m below ground level

Laboratory testing was undertaken on the soil samples.

Ground water has been measured over repeat visits to determine water levels and flows.
• A repeat reading observed water at 1.02m below ground level.</td></tr>
<tr><td>Stage 4 – Impact assessment</td><td>Land stability
The Geologist has concluded that the basement will not make the area unstable.

The movement assessment of the basement and its construction are SLIGHT ie 1-0 on the Burland scale.

It is concluded that with the construction of the new basement at 98a Priory Road should not have significant impacts on land stability provided that:

• Groundwater inflow, if encountered is properly controlled and is monitored before, during and after construction.
• The construction of the basement is carried out by a competent who will adopt suitable measures to maintain the stability of the excavations
• Care is taken to minimise disturbance to trees and their roots.
• Concrete is designed to account for the sulphate conditions anticipated.
• Monitoring of the structures is carried out before, during and after construction.

Hydrogeology

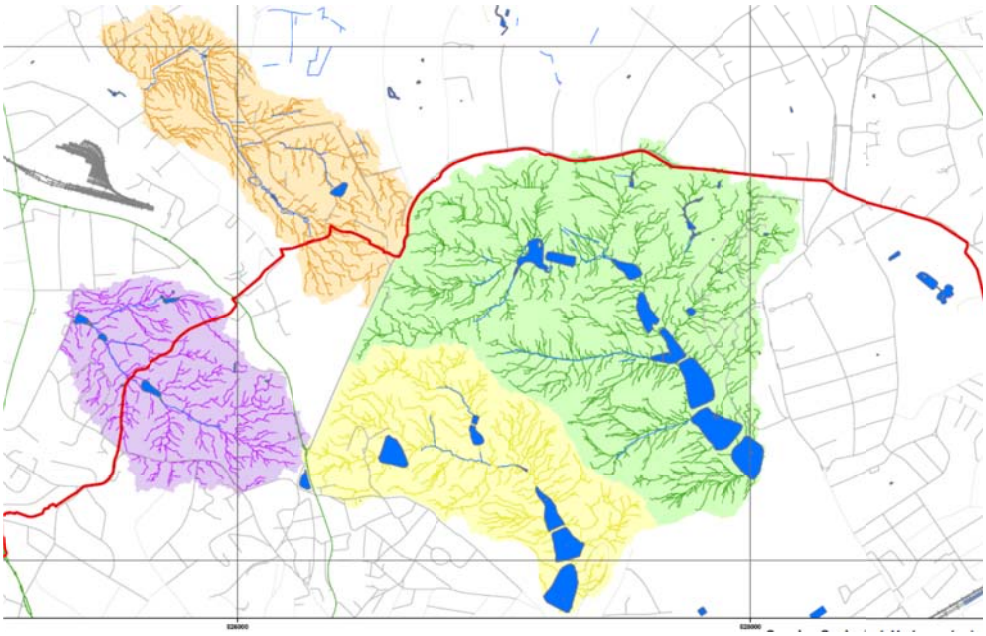
The basement design should include protection against groundwater ingress, and measures should be taken to protect the excavation against groundwater ingress during construction.

There is the potential for groundwater to back-up around the proposed basement structure, which may affect neighbouring basements and cellars. The basement design should include groundwater drainage systems to prevent groundwater backing up around the development, and thereby protect neighbouring properties from impact.

Seasonal fluctuations in groundwater combined with backing up of groundwater levels around the basement structure have the potential to</td></tr>
</table>

	cause daylighting of groundwater at the surface, and/or damage to neighbouring basements and cellars. The basement design should allow for seasonal fluctuations in groundwater elevations, which may rise above 1 m below ground level. Installation of appropriate groundwater drainage systems will control over groundwater elevations, and prevent daylighting of groundwater at surface. It is recommended that ongoing monitoring is undertaken to confirm likely groundwater elevations during times of seasonally high groundwater elevations. The proposed basement structure should be adequately protected against permeation of soil moisture. **Drainage & Surface Water Flow** To control the amount of water in the ground, type I material is proposed immediately below the basement structure. 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. This risk is inherent in all subterranean structures which have an incoming supply of water. The risk can be reduced to acceptable levels with appropriate design measures.

1. Screening Stage

	This stage should identify any areas for concern and therefore focus effort for further investigation. The questions below are taken from the Camden CPG 4 – Basements and Lightwells.
Land Stability	Refer to the Chartered Geologist's Report.
Subterranean Flow	Refer to the Hydrogeologist's report (completed by a Chartered Geologist).
Surface Flow and Flooding	
	Question 1: **Is the site within the catchment of the pond chains on Hampstead Heath?**  *Figure 1: Extract from figure 14 of the Hydrogeological Study* **No**. The site lies outside the areas denoted by Figure 14 of the Hydrogeological Study (extract shown above)
	Question 2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route?

Unknown – Due to the construction of the rear extension and the rear lightwell, the flow of water into the ground and the existing surface water drainage system may change. Carry forward to scoping

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

Unknown – Due to the construction of the rear extension and the rear lightwell, the impermeable areas may change. Carry forward to scoping

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

No. Drainage from 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 will not 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 the site situated in Flood Zone 1. Distance from nearest surface watercourse >1km
Tidal flooding	No	Site location is ‘inland’ and topography > 40mAOD.
Flooding from rising / high groundwater	No	Site is located on low permeability London Clay.

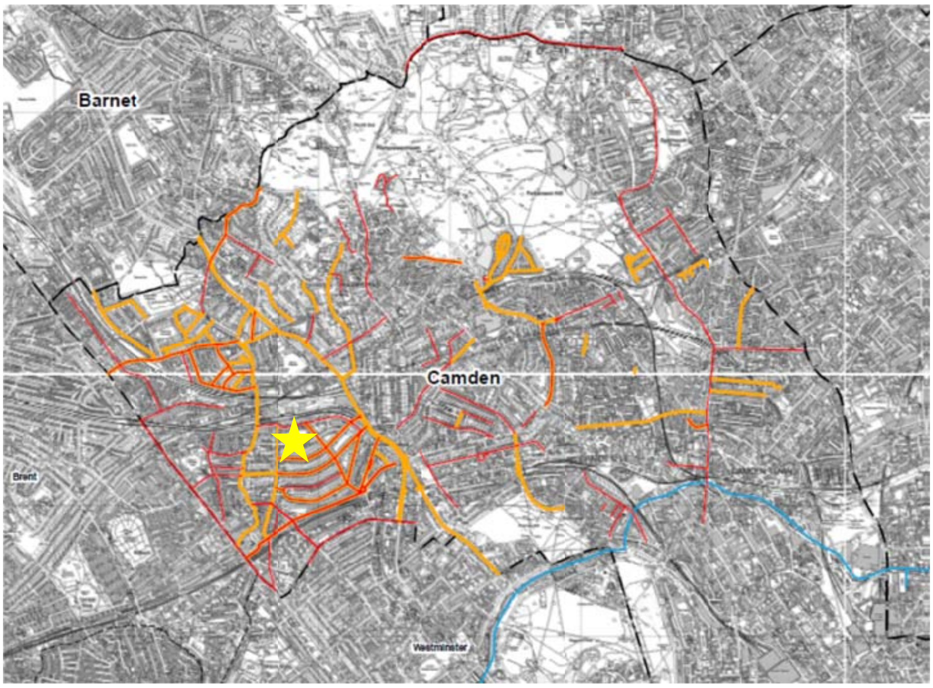
Surface water (pluvial) flooding	Yes	The Coach House is noted on list of streets that were flooded in 1975 or 2002 (shown graphically below)
		
Flooding from infrastructure failure	Yes	Drainage at or near the site could potentially become blocked or cracked and overflow or leak. Drainage of the basement areas may rely on pumping.
Flooding from reservoirs, canals and other artificial sources	No	From inspection of OS maps, there are no reservoirs, canals or other artificial sources in the vicinity of the site that could give rise to a flood risk.

	 *Figure 2: Extract from OS map showing contour lines* Carry forward to Scoping Stage

2. Scoping Stage

	This stage identification the potential impacts of the areas of concern highlighted in the Screening phase.
Land Stability	Refer to the Chartered Geologist's report.
Subterranean Flow	Refer to the Chartered Hydrogeologist's report
Surface Flow & Flooding	**Conceptual Model** The proposed works at the Coach House 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 <u>not</u> an additional self-contained unit intended to serve as a standalone dwelling. Lightwells increase the impermeable hardstanding slightly at the rear, which may increase the flow of drainage into the existing surface water drainage system.
	Question 1: Is the site within the catchment of the pond chains on Hampstead Heath? No further information required from Scoping stage
	Question 2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route? **Unknown –** there may be a marginal change in the increase of impermeable hardstanding at the rear of the property. It is not known how much surface water currently enters the ground in the rear courtyard hence the unknown extent (if any) of the change of flow. 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?

	Unknown – there will be hardstanding to the rear of the property. It is possible that the net area of hard surfaced external areas may increase but this will be marginal. Carry forward to Site Investigation & Desk Study
	Question 4. Will the proposed basement result in changes to the inflows (instantaneous and long term) of surface water being received by adjacent properties or downstream watercourses? No further information required from Scoping stage
	Question 5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses? No further information required from Scoping stage
	Question 6 : Is the site in an area identified to have surface water flood risk according to either the Local Flood Risk Management Strategy or the Strategic Flood Risk Assessment or is it at risk from flooding, for example because the proposed basement is below the static water level of nearby surface water feature? It is evident from the screening study that the only significant flood risks at the Coach House are due to surface water (pluvial) flooding and failure of existing sewers in the vicinity of the site. Carry forward to Site Investigation & Desk Study

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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

Geoff Watson, a structural engineer from Croft SE, visited the Coach House for a walkover survey.

The inspection was on 1April 2015. The data collected from this survey corroborates and adds to information obtained from the desk study.

Proposed Development

The existing property is a two-storey building, which has a courtyard and a conservatory to the rear and a driveway at the front.

The current proposal is to extend the building to the side and to the rear. There is also a proposal for a basement. This will be below the new footprint of the building and will include a lightwell at the front and another at the rear.

Location

The property is located in a built up area. Mature trees are present in the vicinity. The surrounding area is relatively flat with a slight slope downwards from north-west to south-east.

Figure 3: Existing Site

The front yard is paved over. The rear courtyard is landsacped with pea shingle. There is a gully in the front courtyard.

Figure 4: Front yard with gully shown

There are also gullies present at the edges of Priory Road, where it intersects with Canfield Gardens.

Figure 5: View of Priory Road looking North toward the Coad House

Site History

The site and the vicinity have been residential for over one hundred years. Prior to this the area was occupied by fields

Figure 6: area map from 1850

Figure 7: area map from 1895

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

Local Bombing

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

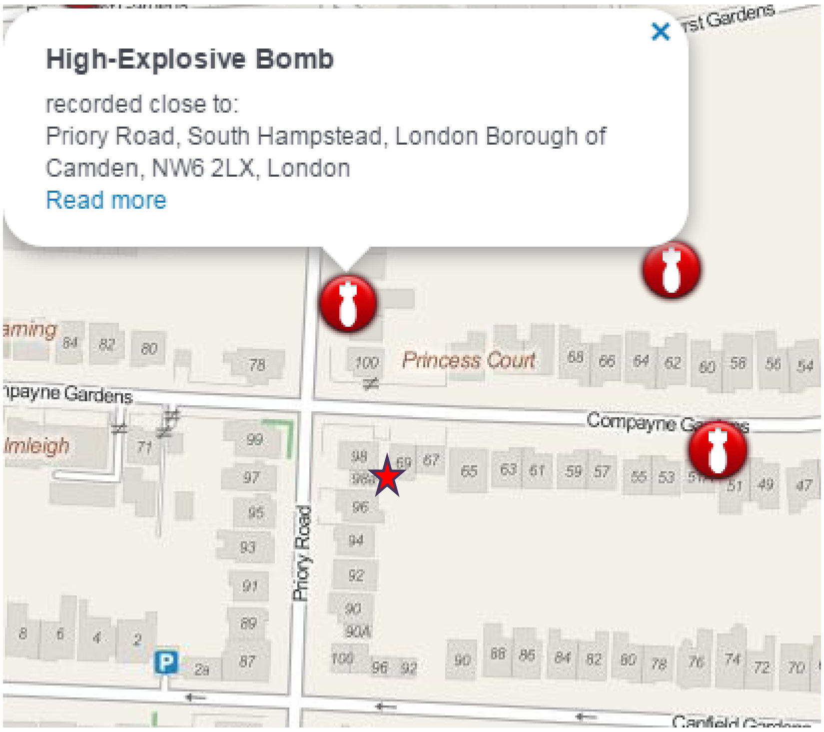


Figure 8: extract from bomb survey map (http://bombsight.org/)

Listed Buildings

Is the building or adjacent buildings listed

No, neither the property not the adjacent buildings are listed

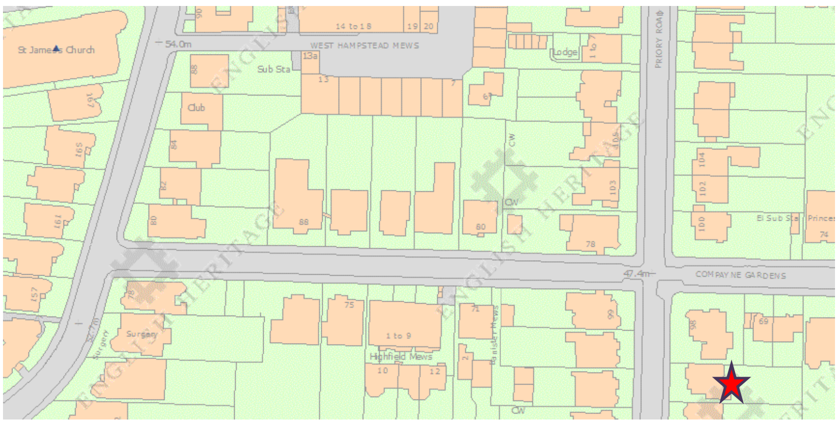
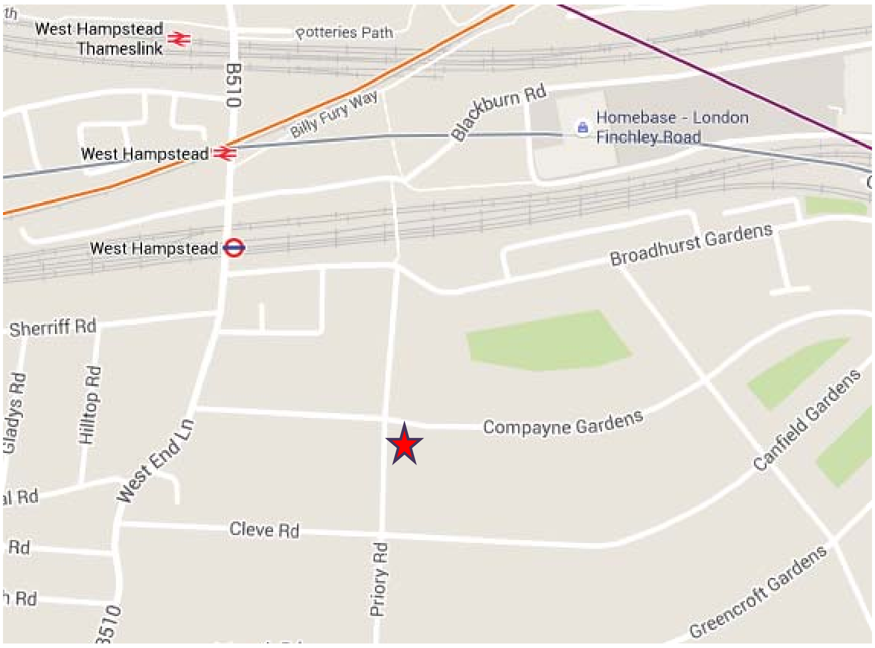


Figure 9: extract showing listed buildings (nearest listed building is shown in top left)

	Highways, Rail and London Underground **Is the site within 5m of the footpath/alleyway/highway?** **Yes.** Site is within 5m of the footpath; the road surface is further than 5m from the proposed front lightwell.
London Underground and Network Rail	**Is the site over (or within the exclusion zone) of any tunnels, e.g. railway lines?** **No.** Nearest is the railway line is over 100m from the site.  *Figure 10: map showing proximity to rail lines*
UK Power Networks	**Will the basement works affect any UK Power Network Assets?** No, there no significant items of electrical infrastructure (such as pylons or substations) in the immediate vicinity.
Proximity of Trees	Some semi-mature trees, shrubbery and general vegetation exist in the neighbouring gardens. There are trees close by with have tree presentation orders. These are across the road and are not present in the neighbouring gardens.

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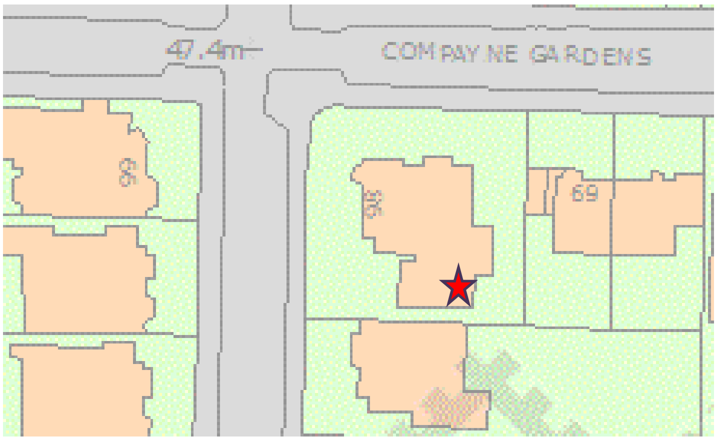
	No trees will be removed for the basement construction.
Building Defects	A visual inspection was undertaken of the existing building with particular attention given to movement to the building. The defects noted were: • Minor cracking on the wall of the bathroom at ground floor level. Given the narrow width of the crack, this is considered a non-structural defect which can be amended with standard decorative works.
	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 external elevations of were taken of the adjacent properties.  *Figure 11: Adjacent properties*
Nos 98/98B Priory Road, Properties to Left	Property Age : mid-Victorian (~150 years old) Property use : residential Number of storeys : the property is three storeys high above ground level. Is a basement present? : Yes – there is a cellar; a window to this was noted during the site visit. Drawings of the property available on Camden Council's website confirm the presence of a cellar. The approximate extent of this, in relation to the proposed structure, is shown in the drawing in Appendix B. The depth of the basement will need to be confirmed at detailed design stage.

Figure 12: view of window above just above ground level, in neighbouring property

Structural Defects Noted – None noted

Structural assessment of ongoing movement: from observing the external façade of the building, there were no visible signs of movement.

Figure 13: front of No 98 Priory Road

No. 96 Priory Road, Property to Right	Property Age : 150 years old (approx.) Property use : residential

	Number of storeys : three storeys high above ground level Is a basement present? : Yes. The approximate extent of this, in relation to the proposed structure, is shown in the drawing in Appendix B. The depth of the basement will need to be confirmed at detailed design stage. Structural Defects Noted – none noted Structural assessment of ongoing movement:  *Figure 14: front of No 96 Priory Road* The external façade of the property does not show any signs of movement.
No 69 Compayne Gardens, Property to Rear	Property Age : 30-60 years old Property use : residential Number of storeys : two Is a basement present? - No (drawings available on Camden Councils website do not show a basement; there are no signs of basement lightwells from street view or from the view of the rear garden.

	Structural Defects Noted: no structural defects were noted from the outside Structural Assessment of ongoing movement: there are no signs of ongoing movement visible from the outside.  *Figure 15: rear of No 69 Compayne Gardens*  *Figure 16: front of No 69 Compayne Gardens*
Local Topography	As mentioned previously, the area surrounding the property has a general slope, downwards from north-west to south-east. The slope is gradual; there are no retaining walls for sudden changes in elevation.

Ground Investigation	Refer to the ground investigation report by Ground and Water, which is submitted as a separate document.
Geology	Refer to the ground investigation report, the hydrogeology report and the land stability assessment, submitted separately.

Surface Flow & Flooding	
Areas of hard standing present on site	Existing area of hardstanding outside is ; Area = 60m^2  *Figure 17: Street View*
Rainwater down pipes, drains, manholes and gulleys	As described previously, there is a surface water drainage gully in the front yard and pea-shingle drainage in the rear yard.
Local Water Sources	**Are there any ponds lakes or water courses on the site or adjacent sites?** No, there are not surface water features (natural or man-made) on the adjacent sites.
	Field Investigation Ground investigation specialists visited the site and subsequently produced are report for the existing ground and groundwater conditions.

	Monitoring, Reporting and Investigation The ground investigation report, which has data from initial site investigations and data from subsequent monitoring, is available as a separate report. Data relevant to land stability and subterranean flow is examined separate documents as described below.
Land Stability	Refer to Chartered Geologist's 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	
Ground Investigation Brief	The ground investigation was completed by Ground and Water. From the Scoping stage, we considered that their brief should cover: • One trial pit to confirm the existing foundations. • Two bore holes to a depth of 7m below ground level (i.e. approximately twice the depth of the proposed basement). • Stand pipe to be inserted to monitor ground water; record initial strike and the water level after 1 month. • Site testing to determine in-situ soil parameters. SPT testing to be undertaken. • Laboratory testing to confirm soil make up and properties. • Historic maps and the walk over survey did not highlight any significant contamination sources; therefore no site contamination testing of the ground has been requested. • Factual report on soil conditions. • Interpretative reports • Calculation of bearing pressures from SPT. • Indication of Ø (angle of shearing resistance) from SPT.

	• Indication of soil type The ground investigation report is provided separately.

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4. Basement Impact Assessment	
Subterranean Flow	Refer to the Hydrogeologist's report : conclusions are stated in the Executive Summary
Land Stability	Refer to Geologist's Report: conclusions re stated in the Executive Summary
Conservation and Listed Buildings	If the property is in a conservation area, or it is listed then management plan for demolition and construction may be needed. This is not included in this BIA document and is not within Croft Structural Engineer's brief.

Flood Risk Assessment

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

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

Site Location

The site is approximately 160m^2 in size. It is located in a densely built-up area. Residential houses exist either side of the site. These buildings are at the same level. There are gardens to the rear of the site. Immediately to the front, the road is relatively flat. There are no rivers or surface water features within 250km of the site.

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

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

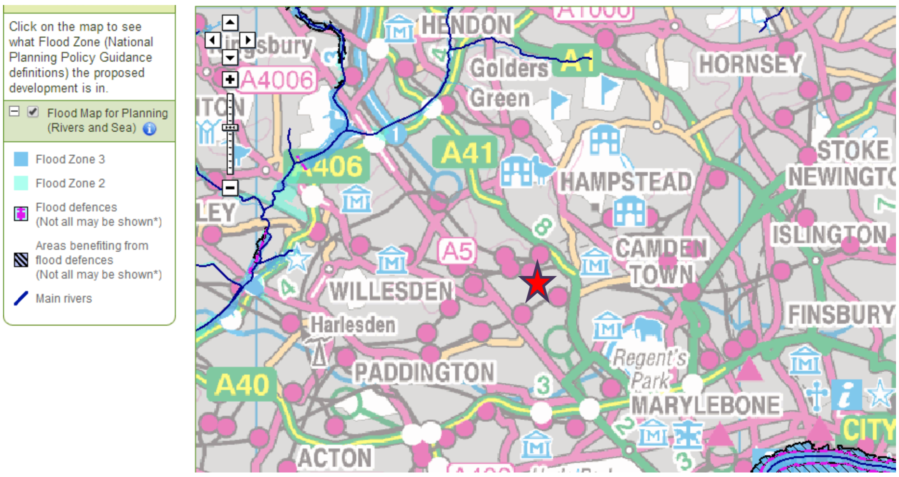
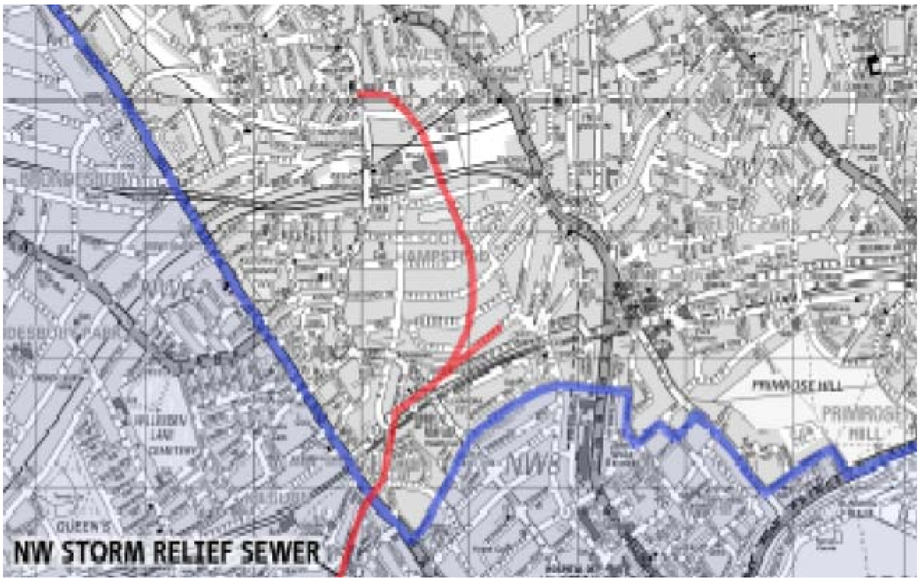


Figure 18: Flood map for planning (Environment Agency)

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

Potential surface water (pluvial) flooding	Priory Road is reported to have flooded in 2002. It is understood that this flooding was due to the Thames Water relief sewer being overloaded.  *Figure 19: Location of Thames Water's North-west storm relief sewer (in red)* It is also understood that Thames Water subsequently increased the capacity of this relief system: the likelihood of flooding of this nature is now significantly reduced. The level of the front yard is at a slightly higher level than the road. Surface water flow, influenced by gravity alone, will be towards the road, and will continue downhill. There is a possibility that, should the nearby drains become blocked or reach their capacity, wind driven rain may enter the front light-well.
Potential flooding from infrastructure failure	In addition to the storm water relief sewer previously mentioned, there is believed to be a trunk sewer running along the length of the Priory Road. Blockage or failure of either of these may result in excess flow from the Priory Road may accumulate in the front yard. The hard standing in the front yard and the brick wall which separates it from No 96, significantly inhibit the flow of any excess water into the neighbouring property. This will continue to be the case under the proposed development. The added risk of flooding is therefore greater for the owner for 98A than for the adjacent owner. The risk of damage to the property is greatest for the new proposed basement: if the surface water drains become blocked and overflow, then water may enter the front lightwell and damage the basement.

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

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Drainage Assessment

Hard standing

The main design change resulting in the reduction of hardstanding is the removal of the pea-shingle area (approximately 10m^2 in plan area) in the rear courtyard. The proposed landscaping for the rear yard has not been designed in detail. It is possible that an area similar in size could be incorporated. This would result in the proportion of hard-standing remaining unchanged. These calculations assume that this design feature will not be used and therefore cover the worst case.

Existing Hard Standing = 150 m^2

Proposed Hardstanding = 160 m^2

Percentage Increase in Hard standing = 6.7 %

SUDS Assessment

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

7 %

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

SUDs is required only to counteract impacts of a loss in hardstanding in the rear yard. This may be the case if the pea-shingle landscaping is not re-instated at the detailed design stage. A proposed solution is presented in the following sub-sections.

SUDS Calculations

The calculations below refer to the rear yard. The area of hardstanding is 20m^2. This is equivalent to 0.002 hectares (due to rounding presentations within the calculations, this is misleadingly presented as '0.00 ha' in the table below).

ATTENUATION DESIGN

In accordance with CIRIA publication C697 - The SUDS Manual

Tedds calculation version 1.0.01

EA_Defra method

Site characteristics

Location London

Hydrological region 6 Soil type (W.R.A.P map)

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Standard percentage runoff	SPR = **0.47**	Average annual rainfall
5yr rainfall of 60min duration	M5_60min = **20.0** mm	Rainfall ratio
Global warming rainfall factor	$p_{climate}$ = **0%**	
Imperv. area req. att storage	α = **100.0** %	

Catchment details

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

Greenfield runoff rates

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

Estimated site discharges

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

Estimated attenuation volume - 1 year

Attenuation storage vol	$Uvol_{1yr}$ = **154.7** m^3 / hectare	Basic storage volume
FEH rainfall factor	FF_{1yr} = **0.90**	Storage volume ratio
Adjusted storage volume	ASV_{1yr} = **0.35** m^3	Hydrological regional vol
ratio	HR_{1yr} = **1.01**	
Final est. attenutation storage	Vol_{1yr} = **0.35** m^3	

Estimated attenuation volume - 30 year

Attenuation storage vol	$Uvol_{30yr}$ = **344.5** m^3 / hectare	Basic storage volume
FEH rainfall factor	FF_{30yr} = **0.85**	Storage volume ratio
Adjusted storage volume	ASV_{30yr} = **0.83** m^3	Hydrological regional vol
ratio	HR_{30yr} = **1.02**	
Final est. attenutation storage	Vol_{30yr} = **0.84** m^3	

Estimated attenuation volume - 100 year

Attenuation storage vol	$Uvol_{100yr}$ = **434.1** m^3 / hectare	Basic storage volume
FEH rainfall factor	FF_{100yr} = **0.80**	Storage volume ratio
Adjusted storage volume	ASV_{100yr} = **1.14** m^3	Hydrological regional vol
ratio	HR_{100yr} = **1.03**	
Final est. attenutation storage	Vol_{100yr} = **1.17** m^3	

Attenuation storage required

Max attenuation storage reqd V_{req_max} = **1.1** m^3

Interception storage

Interception rainfall depth	d_{int} = **5** mm	Interception storage
reqd	V_{int_req} = **0.08** m^3	

Long term storage

Prop of paved area draining	α = **1.0**	Prop of pervious area
draining	β = **0.5**	
Rainfall 100years, 6 hour	RD = **60.1** mm	Extra runoff over g'field
runoff	Vol_{xs} = **0.40** m^3	

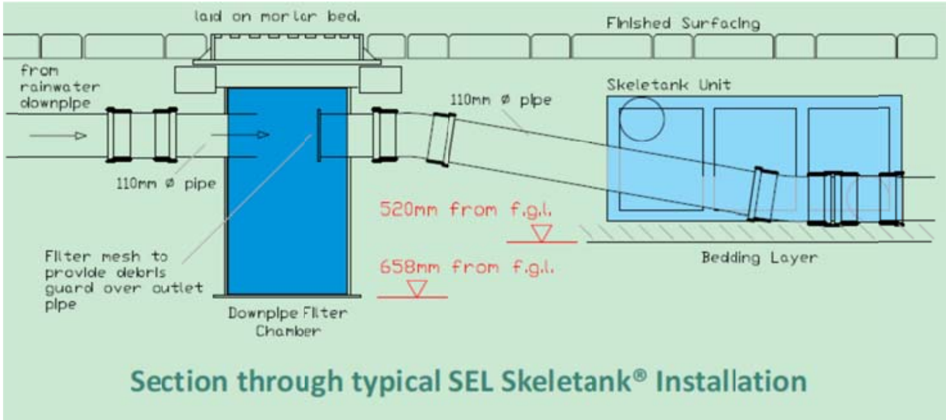
Treatment volume

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

Mitigation Measures

From the SUDS calculations, a volume of 1.2m³ is required for storage we would recommend that onsite storage if used.

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



Section through typical SEL Skeletank® Installation

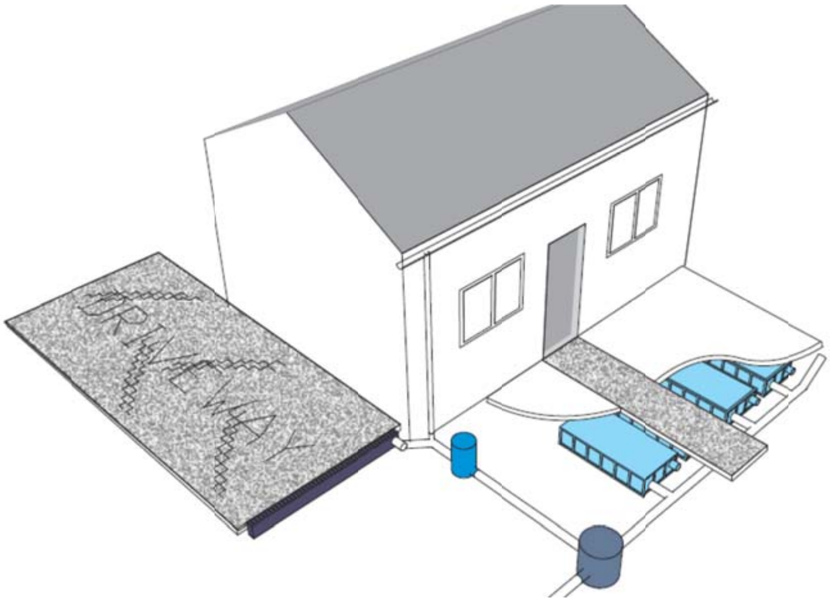


Figure 20 Diagrammatic representation only

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

Effects on existing drainage

Not build over agreements known of.

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

The potential for ground water to rise and cause flooding above the ground surface is addressed later in 'Basement Design & Construction Impacts'.

Trees

Root Protection Zone

RPA = 1.5 x Crown diameter.

The basement is not within the root protection area (RPA) of the trees noted below:

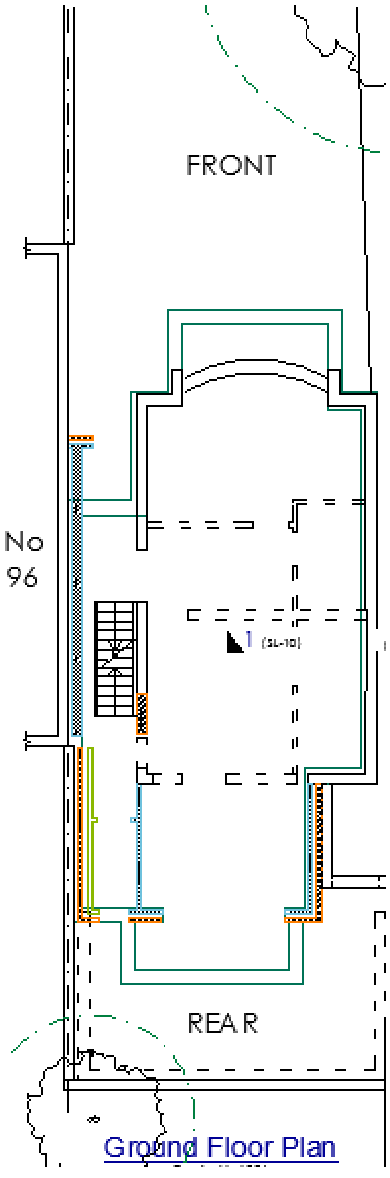
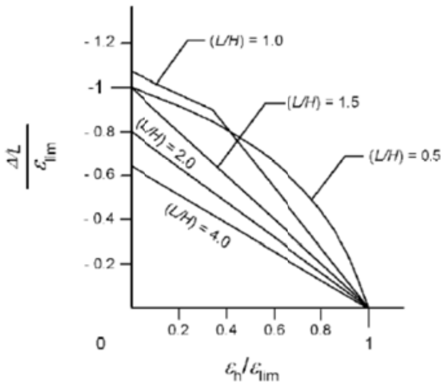
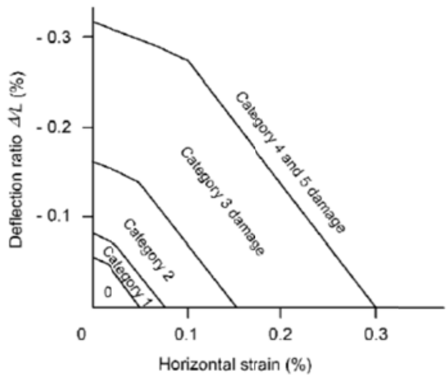


Figure 21. Proposed site plan with basement outline and trees shown; green dashed lines indicate approximate extent of root protection areas

Conclusion

The basement does not cut into the tree root protection zone.

The increased depth of foundations, necessary for the basement, places the new foundations outside the effects of trees. The building will be more stable due to the new basement.

Ground Movement Assessment & Predicted Damage Category	
	This assessment covers both short term and long term movements relating to the construction and the performance of the permanent works. The design and construction methodology aims to limit damage to the existing building on the site and to all adjoining buildings to Category 1 as set out in Table 2.5 of CIRIA report C 580 . This assessment has used empirical means as set out in CIRIA2 C 580 Embedded Retaining Walls: Guidance for Economic Design.  (b) Influence of horizontal strain on Δ/L / ε_{lim} (after Burland, 2001)  (c) Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for (L/H) = 1.0 (after Burland, 2001)

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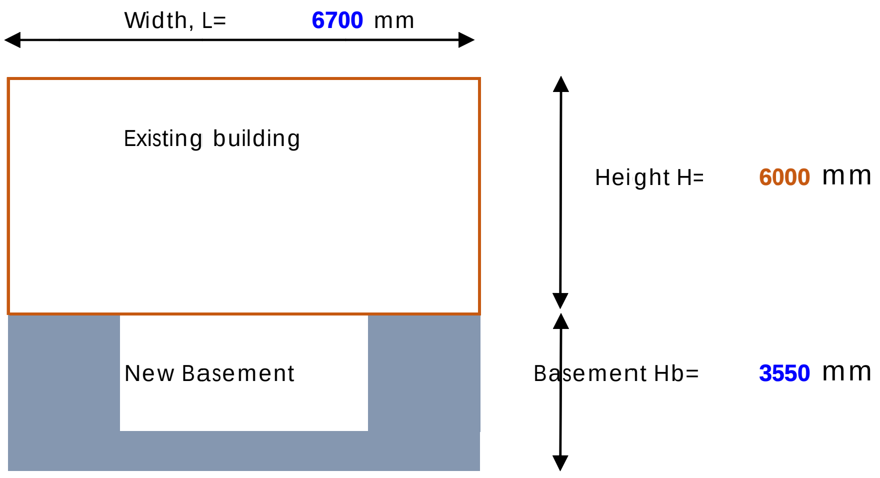
Predicted Movement

Movement



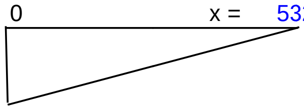
L/H = 1.11667

Movement Assessment CIRIA C580: Embedded retaining walls - guidance for ecomonic design

Potential movement due to installation of wall — using parameters from Table 2.2 of CIRIA C580

Horizontal Surface Movement / Wall Depth				=	0.05%	
max δ_h =	0.05%	x	3550	=	1.775	mm
Distance behind wall wall to neglibible movement (mulitple of wall depth)				=	1.5	
L =	3550	x	1.5	=	5325	mm
Vertical Surface Movement / Wall Depth				=	0.05%	
max δ_v =	0.05%	x	3550	=	1.775	mm
Distance behind wall wall to neglibible movement (mulitple of wall depth)				=	1.5	
L =	3550	x	1.5	=	5325	mm
movment gradient (vertical and horizontal)				=	0.3	mm/m

x = 0 x = 5325 mm (distances are measured from underpinned wall)



δ_v = 1.8 mm

δ_h = 1.8 mm

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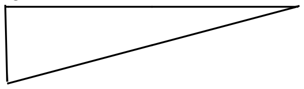
<u>Potential movement due to excavation of wall</u>

using parameters from Table 2.4 of CIRIA C580
(excavation will be propped during construction)

Horizontal Surface Movement / Wall Depth			=	0.15%	
max δ_h =	0.15%	x 3550	=	5.325	mm
Distance behind wall wall to neglibible movement (mulitple of wall depth)			=	4	
L =	3550	x 4	=	14200	mm
Movement gradient (horizontal)			=	0.4	mm/m

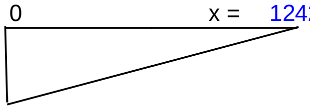
x = 0 x = 14200 mm (distances are measured from underpinned wall)



δ_h = 5.3 mm

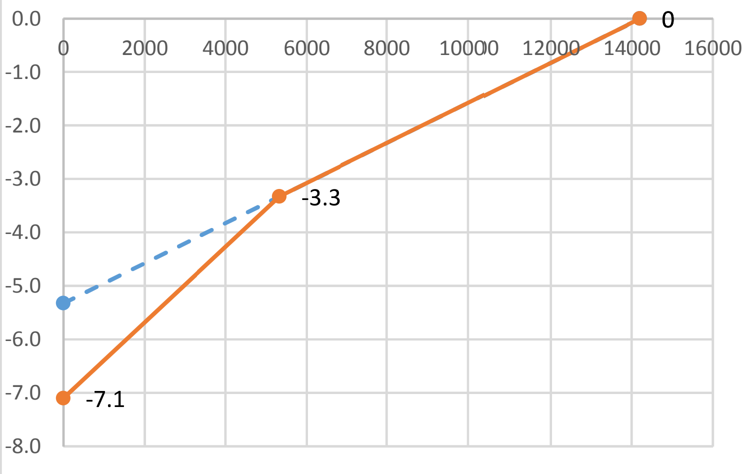
Vertical Surface Movement / Wall Depth			=	0.10%	
max δ_v =	0.10%	x 3550	=	3.55	mm
Distance behind wall wall to neglibible movement (mulitple of wall depth)			=	3.5	
L =	3550	x 3.5	=	12425	mm
movment gradient (vertical)			=	0.3	mm/m

x = 0 x = 12425 mm (distances are measured from underpinned wall)



δ_v = 3.6 mm

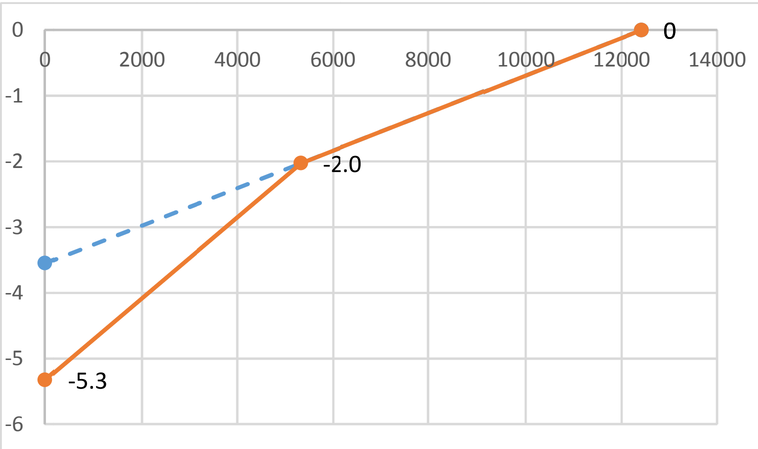
Horizontal Movement	x	δ_h
Excavation	0	-5.3
	14200	0
Installation	0	-1.8
	5325	0
Total movement	0	-7.1
	5325	-3.3
	14200	0



Determine Horizontal Movement

$$\max \delta / m = \frac{7.1 \text{ mm}}{14200 \text{ mm}} = 0.05\%$$

Vertical Movement	x	δ_v
Excavation	0	-3.55
	12425	0
Installation	0	-1.8
	5325	0
Total movement	0	-5.3
	5325	-2.0
	12425	0



Determine Vertical Movement

$$\delta_v = \frac{5.325 \text{ mm}}{12425 \text{ mm}} = 0.04\%$$

Table 2.5 CIRIA C580

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

Anticipated Damagae May be Categorised as **'Negligible' to 'Very Slight' ; Category 0-1**

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

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

Monitoring

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

Risk Assessment

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

		Monitoring 4 Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical monitoring movement by standard optical equipment Lateral movement between walls by laser measurements	New basements greater than 2.5m and shallower than 4m Deep in gravels Basements up to 4.5m deep in clays Underpinning works to grade I listed building

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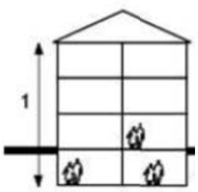
Basement Design & Construction Impacts

Foundation type	Reinforced concrete cantilevered retaining walls The design for the retaining walls has been calculated using software by TEDDS. The software is specifically designed for retaining walls and ensures the design is kept to a limit to prevent damage to the adjacent property. The overall stability of the walls is designed using K_a & K_p values, while the design of the wall uses K_o values. This approach minimises the level of movement from the concrete affecting the adjacent properties. The ground investigations have highlight that water is present below the proposed formation level of the basement. The walls are designed to cope with the hydrostatic pressure. The water table was low; the design of the walls however considers the long term items. It is possible that a water main may break, causing local high water table. To account for this, the walls are designed for water at full basement wall height. 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.
Roads	The front lightwell is the closest part of the basement to the road. It is greater than 5m away. Highways loadings would not be required however, given the possibility that larger vehicles may enter the front driveway, a surcharge of 10kN/m² should be allowed for in the design.
Intended use of structure and user requirements	Family/domestic use
Loading Requirements (EC1-1)	see table below
Part A3 Progressive collapse	Number of Storeys: 3 Is the Building Multi Occupancy? No

Loading Requirements (EC1-1):

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

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

<table>
<tr><td></td><td>Class 1 Single occupancy houses not exceeding 4 storeys</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>N / A</td></tr>
</table>


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

Hydrostatic pressure will be applied to the wall

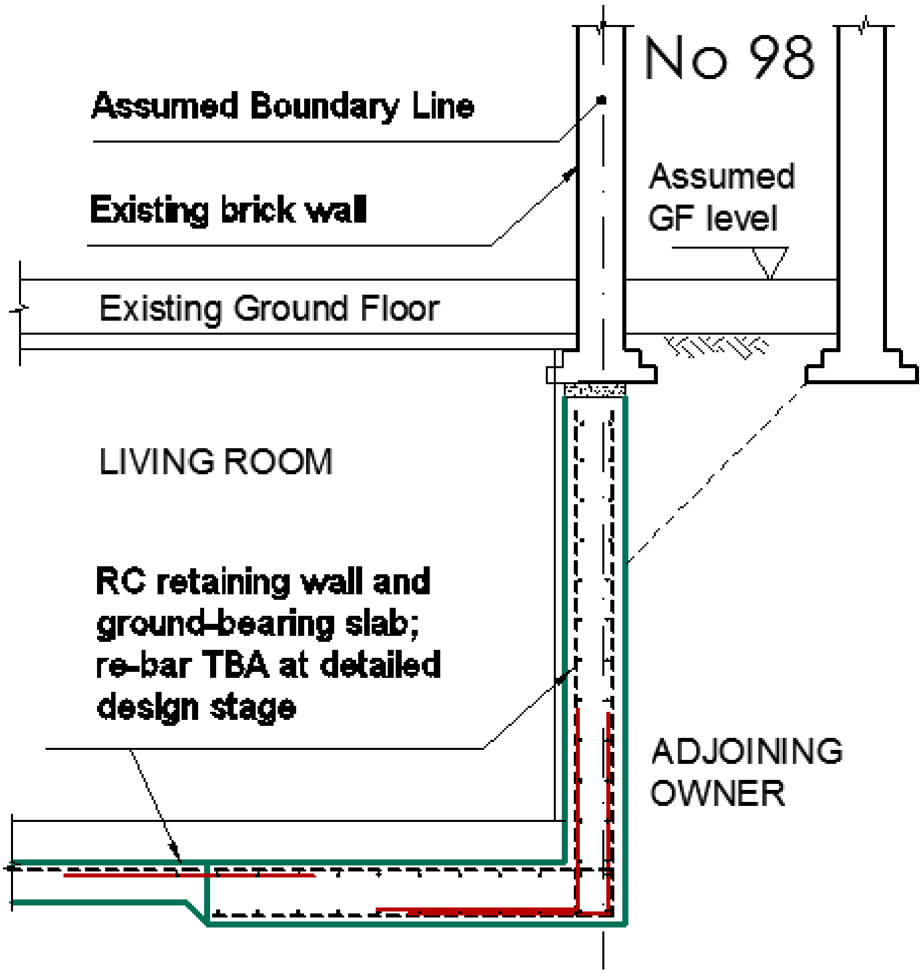
Imposed loading will surcharge the wall.</td></tr>
<tr><td>Retained soil Parameters</td><td>Design overall stability to K_a & K_p values. Lateral movement necessary to achieve K_a mobilisation is height/500 (from Tomlinson). This is tighter than the deflection limits of the concrete wall.</td></tr>
<tr><td>Water Table</td><td>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, showed water to be present at approximately 1m below ground level. This is likely to be a perched water table. To account for the worst case scenario, the design of the retaining wall is designed with the water table at full height. This will allow for local failure of water mains, drainage and storm water.</td></tr>
</table>

Drainage and Damp Waterproofing	From the ground investigation, the borehole has shown that made ground and head deposits exist down to a depth of 1.25m. These materials are significantly more permeable than the London clay below. These will be partially interrupted by the proposed basement. However, given that made ground and head deposits is present (and will continue to exist) to the front and rear of the property, ground water can continue to migrate through the more permeable strata as before. The effects of excess ground water will also be mitigated by the use of SuDS, as described previously. The likelihood of ground water rising to the surface and flooding neighbouring properties is therefore unlikely to increase under the proposed scheme. It is recommended that a water proofing specialist is employed to ensure all the water proofing requirements are met. Croft Structural Engineers are not the waterproofing designer nor act as the structural waterproof designer. Croft is not the structural waterproofer. The waterproofing specialist must name who is their structural waterproofer. The Structural waterproofer must inspect the structural details and confirm that are happy with the robustness. Due to the construction nature of the segmental basement it is not possible to water proof the joints. All water proofing must be made by the waterproofing specialist. They should make review of our details and recommend to us if water bars and stops are necessary. The waterproof design must not assume that the structure is watertight. To help reduce water floor through joints in the segmental pins all faces should be; • Cleaned of all debris and detritus • Faces between pins should be needle hammered to improve key • All pipe work and other penetrations should have puddle flanges or hydrophilic strips
Localised Dewatering	Localised dewater to pins may be necessary. Some engineers may raise the theoretical questions about pumping of water causing localised settlement. We believe that this argument is a red herring when applied to single storey basements and our reason for stating this is: • The water table in the area is variable, • The water level naturally rises and falls over time and does not lead to subsidence • The water table has naturally been rising and falling for over the last 20,000 years, any fines that will have been removed from the soil would have done so already. • If the water table rises and falls naturally why does this not cause subsidence due to fine removals every year? It does not because the soil has been soil is naturally consolidated by the rise and fall of

	the water table in the area. • The effect of local pumping for small excavations will not affect the local area. • There is only a risk of subsidence from large scale pumping of soil which lowers the water table below is natural lowest level. The construction of the basement would not involve a complete excavation before any single retaining wall is constructed: the walls will be constructed progressively as the excavation progresses. Once these walls are cast, they will form a partial barrier against the any ground water. Full scale dewatering around the complete perimeter of the excavated area would therefore not be required.
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 the engineers responsible for the design of the permanent structure at detailed design stage. Particular attention should be paid to the point loads from above.
Geological Assessment of Land Stability	Has the retaining wall design been assessed by a Chartered Geologist? Yes inspected see supplementary report.

Retaining Wall Calculation

Below Party Wall - Permanent Design



For worst case, allow for hydrostatic forces upto the top of the retaining wall.
Use lowest anticipated vertical loads on top of wall.
The above parameters will give the worst case design for the wall stem

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06

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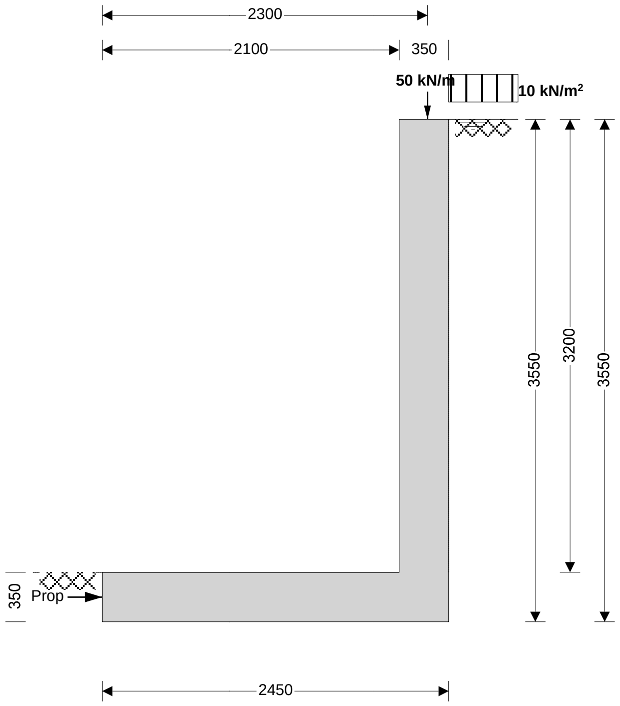


Wall details

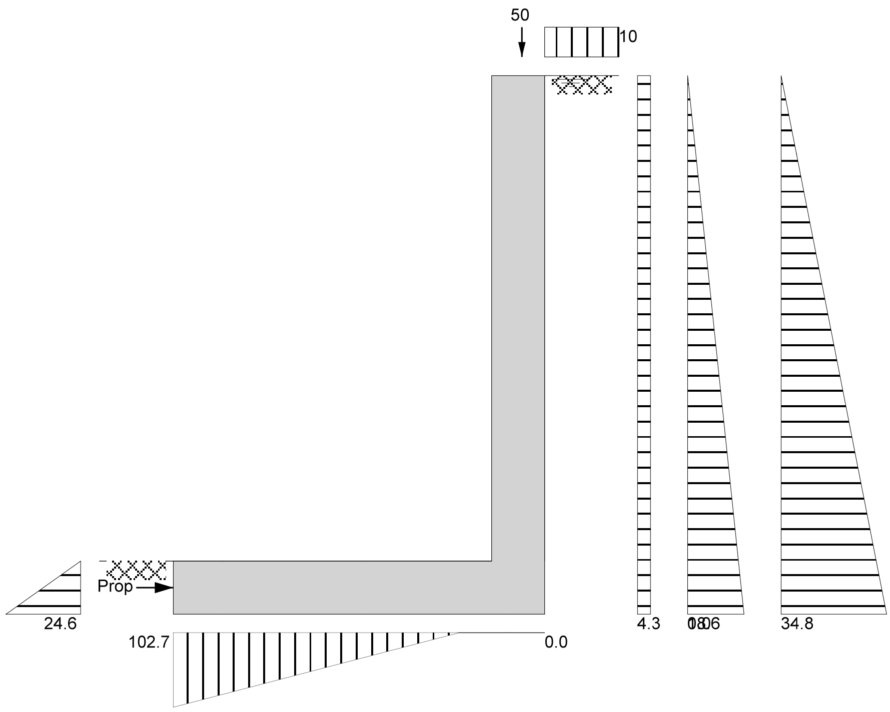
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3200** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **2100** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **2450** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **3550** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** mm
Position of downstand	l_{ds} = **900** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **3550** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **3550** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **22.0** kN/m^3	Saturated density	γ_s = **22.0** kN/m^3
Design shear strength	ϕ' = **20.0** deg	Angle of wall friction	δ = **12.5** deg
Design shear strength	ϕ'_b = **20.0** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **22.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **120** kN/m^2

Using Coulomb theory

Active pressure	K_a =**0.440**	Passive pressure	K_p = **3.374**
At-rest pressure	K_0 = **0.658**		

Loading details

Surcharge load	Surcharge = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **50.0** kN/m	Vertical live load	W_{live} = **0.0** kN/m
Horizontal dead load	F_{dead} = **0.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m
Position of vertical load	l_{load} = **2300** mm	Height of horizontal load	h_{load} = **0** mm



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

Calculate propping force

Propping force F_{prop} = **73.2** kN/m

Check bearing pressure

Total vertical reaction R = **96.7** kN/m Distance to reaction x_{bar} = **628** mm

Eccentricity of reaction e = **597** mm

Reaction acts outside middle third of base

Bearing pressure at toe p_{toe} = **102.7** kN/m^2 Bearing pressure at heel p_{heel} = **0.0** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

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

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

Calculate propping force

Propping force F_{prop} = **73.2** kN/m

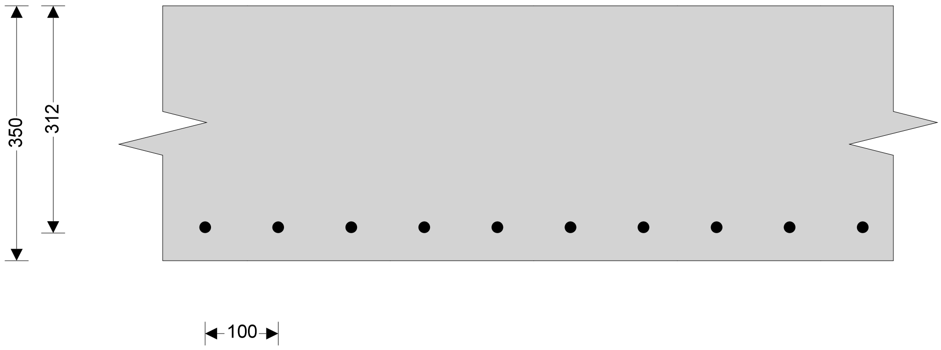
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

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



Design of retaining wall toe

Shear at heel V_{toe} = **111.1** kN/m Moment at heel M_{toe} = **250.5** kNm/m

Compression reinforcement is not required

Check toe in bending

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

Area required $A_{s\_toe\_req}$ = **2001.2** mm^2/m Area provided $A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.356** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.679** N/mm^2

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

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

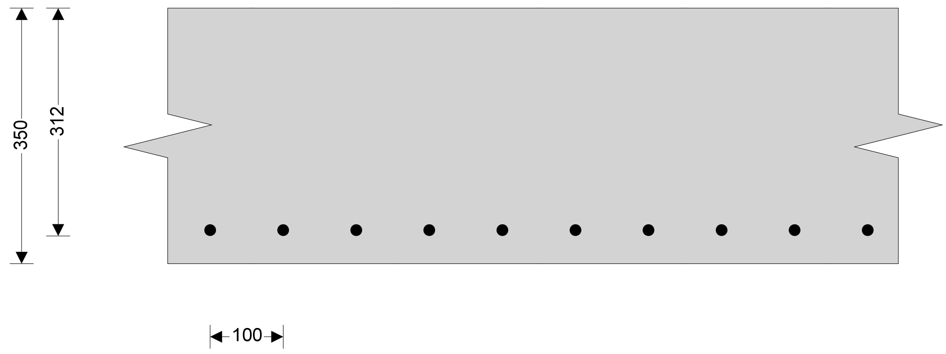
Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **18.4** kN/m Moment at base of stem M_{stem} = **196.1** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **1536.5** mm^2/m Area provided $A_{s\_stem\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

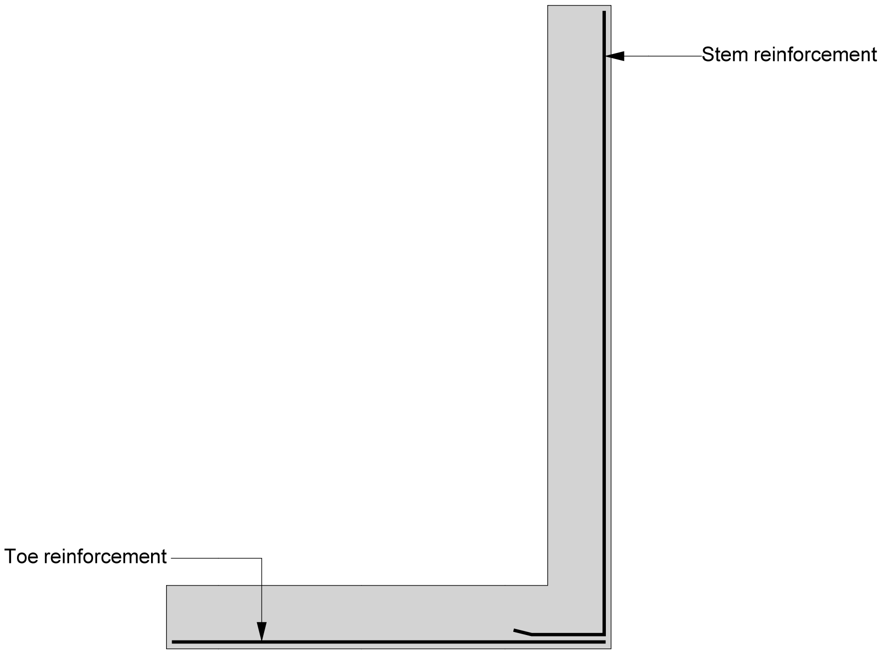
Design shear stress v_{stem} = **0.059** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.679** N/mm^2

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

Indicative retaining wall reinforcement diagram



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

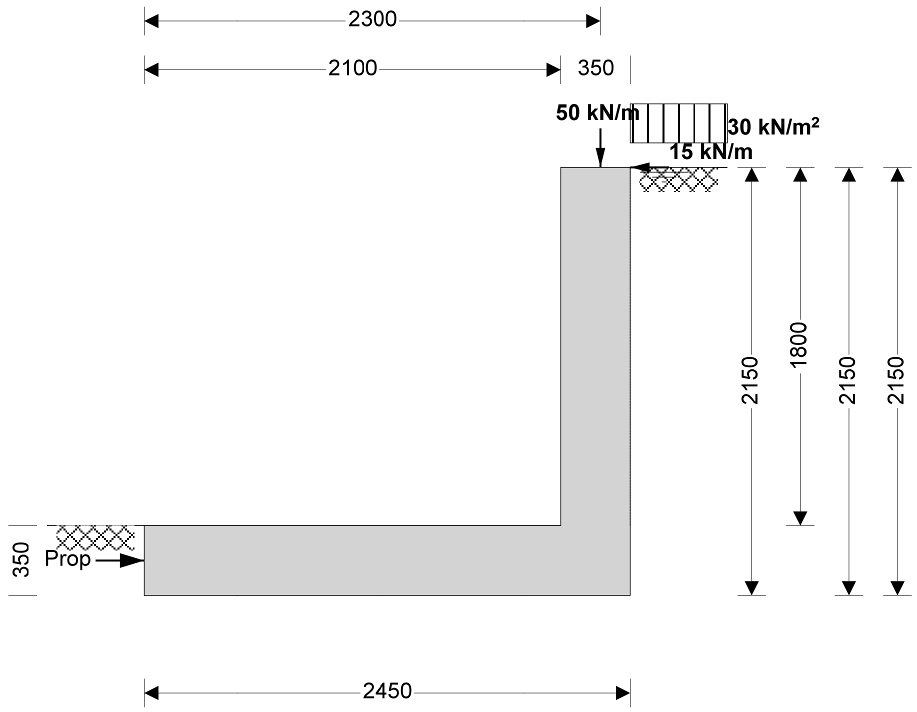
Below party wall, lower portion - permanent design

The lower portion of the retaining wall will resist the surcharge loading from the external wall of the neighbouring property.
The wall is modelled wilth a reduced height. Ther lateral earth pressures and hydrostatic pressures above the (reduced) wall are applied as an equivalent horizontal load at the head.

The calculation below is applicable to the design for the reinforcement required for the lower half of the retaining wall only.

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **1800** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **2100** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **2450** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **2150** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** mm
Position of downstand	l_{ds} = **1500** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **2150** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **2150** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **20.0** deg	Angle of wall friction	δ = **18.6** deg
Design shear strength	ϕ'_b = **20.0** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **22.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **120** kN/m^2

Using Coulomb theory

Active pressure	K_a =**0.429**	Passive pressure	K_p = **3.374**
At-rest pressure	K_0 = **0.658**		

Loading details

Surcharge load	Surcharge = **30.0** kN/m^2		
Vertical dead load	W_{dead} = **50.0** kN/m	Vertical live load	W_{live} = **0.0** kN/m
Horizontal dead load	F_{dead} = **15.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m
Position of vertical load	l_{load} = **2300** mm	Height of horizontal load	h_{load} = **2150** mm

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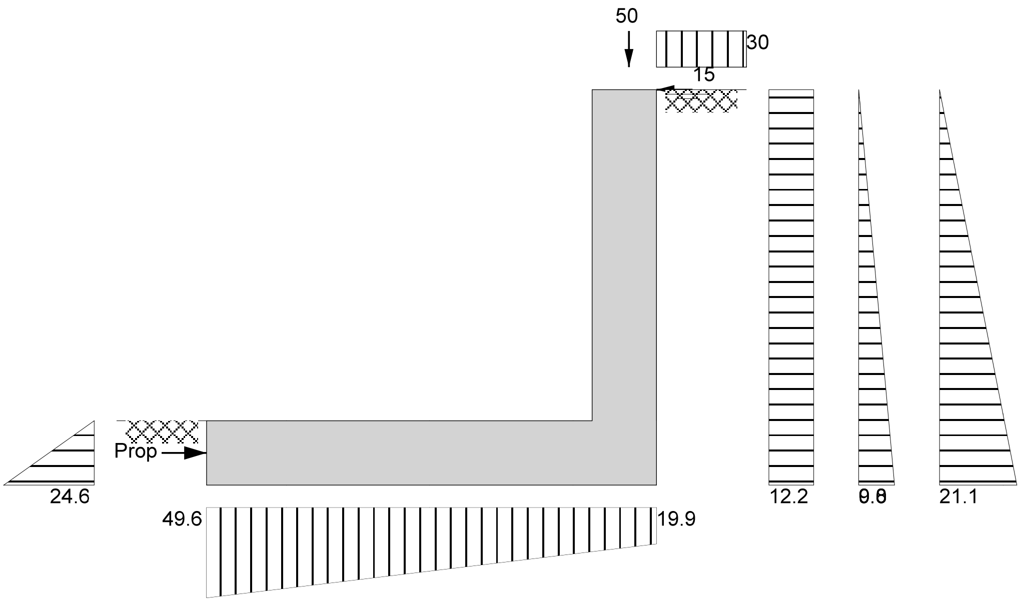

Loads shown in kN/m, pressures shown in kN/m^2

Calculate propping force

Propping force F_{prop} = **41.4** kN/m

Check bearing pressure

Total vertical reaction R = **85.1** kN/m Distance to reaction x_{bar} = **1051** mm

Eccentricity of reaction e = **174** mm

Reaction acts within middle third of base

Bearing pressure at toe p_{toe} = **49.6** kN/m^2 Bearing pressure at heel p_{heel} = **19.9** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

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RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

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

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

Calculate propping force

Propping force F_{prop} = **41.4** kN/m

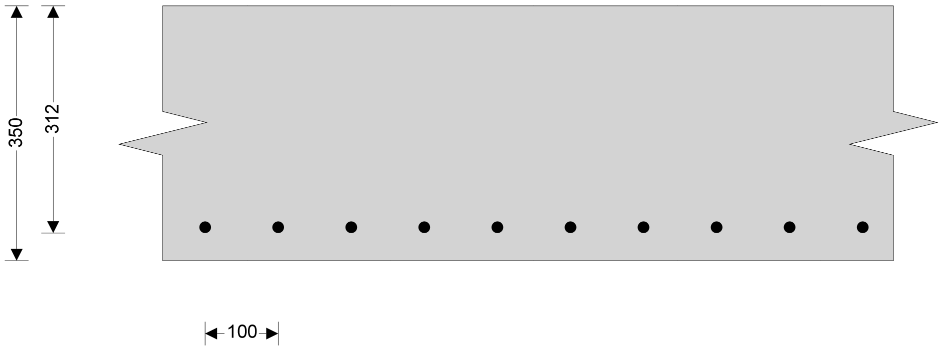
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

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



Design of retaining wall toe

Shear at heel V_{toe} = **94.8** kN/m Moment at heel M_{toe} = **162.0** kNm/m

Compression reinforcement is not required

Check toe in bending

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

Area required $A_{s\_toe\_req}$ = **1256.1** mm^2/m Area provided $A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.304** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.679** N/mm^2

v_{toe} < v_{c_toe} - No shear reinforcement required

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

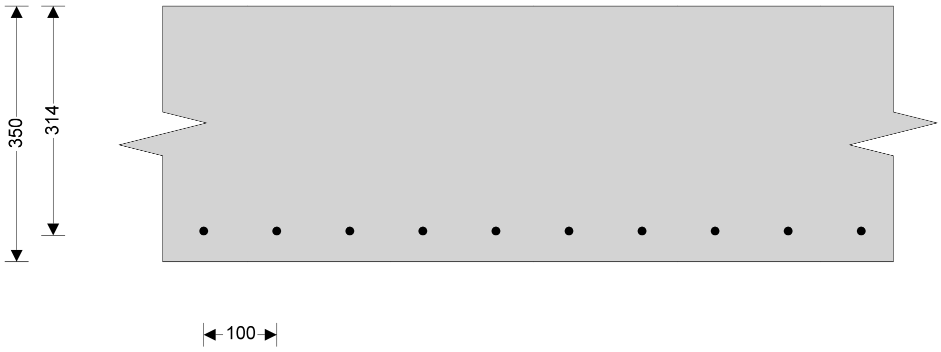
Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **18.5** kN/m Moment at base of stem M_{stem} = **126.0** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **970.7** mm^2/m Area provided $A_{s\_stem\_prov}$ = **1131** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

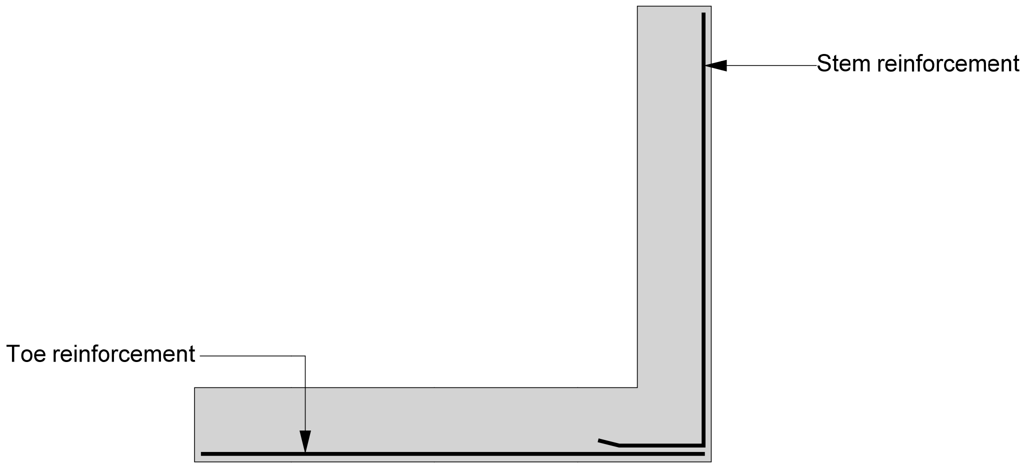
Design shear stress v_{stem} = **0.059** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.559** N/mm^2

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

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)
Stem bars - 12 mm dia.@ 100 mm centres - (1131 mm^2/m)

Below Party Wall - Temporary Condition

Temporary conditions apply during construction
Water will be pumped away from the excavations
For worst case, apply maximum anticipated vertical loads at top of wall and lowest anticipated surcharge
The parameters above will give the worst case loading for the base design and the bearing pressure on the soil.

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06

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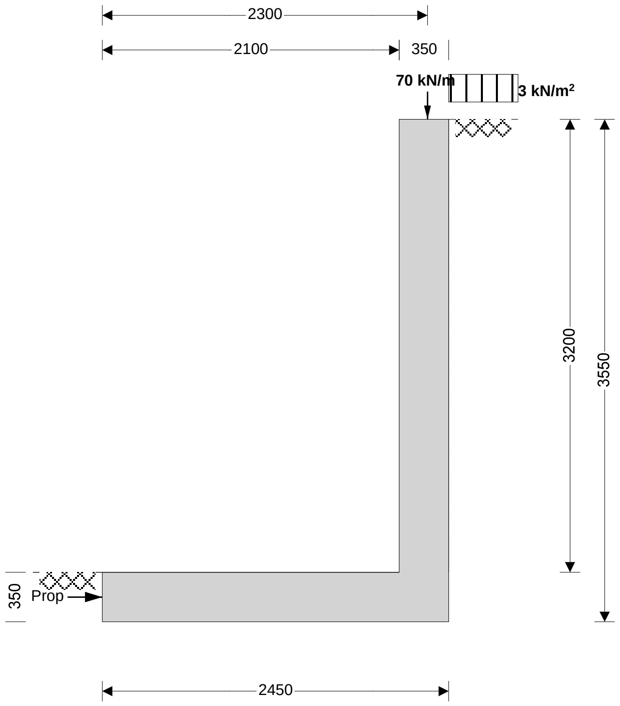


Wall details

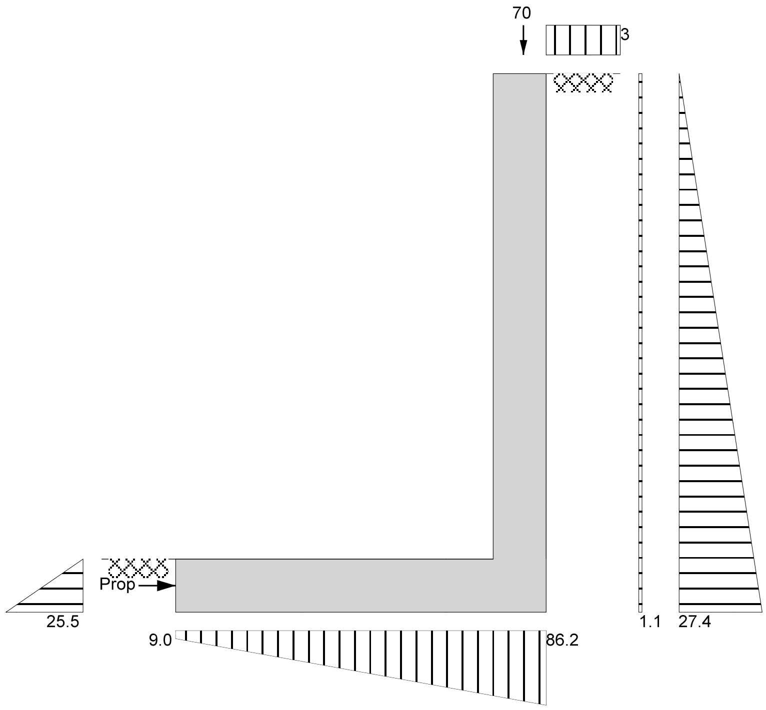
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3200** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **2100** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **2450** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **3550** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** mm
Position of downstand	l_{ds} = **1800** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **0** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **3550** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **20.0** deg	Angle of wall friction	δ = **12.5** deg
Design shear strength	ϕ'_b = **20.0** deg	Design base friction	δ_b = **20.0** deg
Moist density	γ_{mb} = **22.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **120** kN/m^2

Using Coulomb theory

Active pressure	K_a =**0.440**	Passive pressure	K_p = **3.525**
At-rest pressure	K_0 = **0.658**		

Loading details

Surcharge load	Surcharge = **2.5** kN/m^2		
Vertical dead load	W_{dead} = **60.0** kN/m	Vertical live load	W_{live} = **10.0** kN/m
Horizontal dead load	F_{dead} = **0.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m
Position of vertical load	l_{load} = **2300** mm	Height of horizontal load	h_{load} = **0** mm



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

Calculate propping force

Propping force F_{prop} = **9.2** kN/m

Check bearing pressure

Total vertical reaction R = **116.7** kN/m Distance to reaction x_{bar} = **1556** mm

Eccentricity of reaction e = **331** mm

Reaction acts within middle third of base

Bearing pressure at toe p_{toe} = **9.0** kN/m^2 Bearing pressure at heel p_{heel} = **86.2** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

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RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

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

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

Calculate propping force

Propping force F_{prop} = **9.2** kN/m

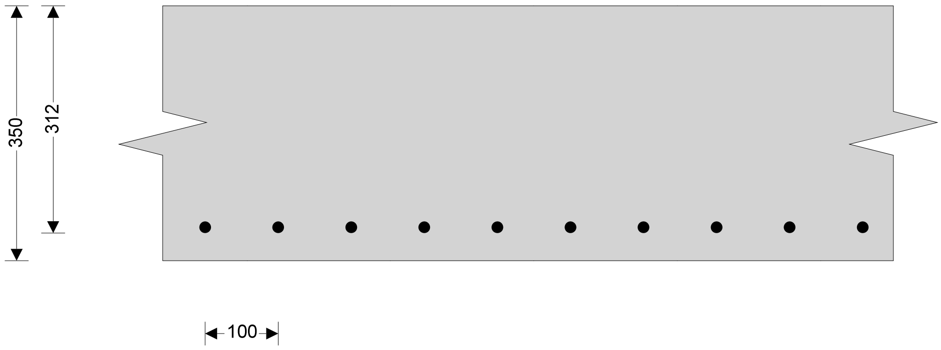
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

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



Design of retaining wall toe

Shear at heel V_{toe} = **115.6** kN/m Moment at heel M_{toe} = **138.7** kNm/m

Compression reinforcement is not required

Check toe in bending

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

Area required $A_{s\_toe\_req}$ = **1075.5** mm^2/m Area provided $A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.370** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.679** N/mm^2

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

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

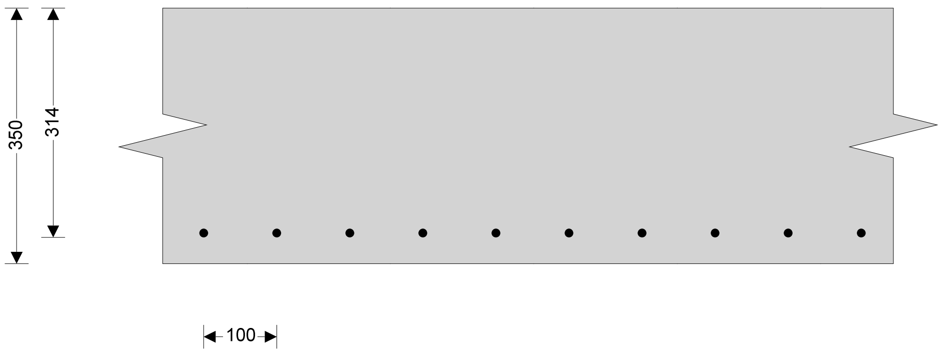
Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **40.1** kN/m Moment at base of stem M_{stem} = **120.4** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **927.6** mm^2/m Area provided $A_{s\_stem\_prov}$ = **1131** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

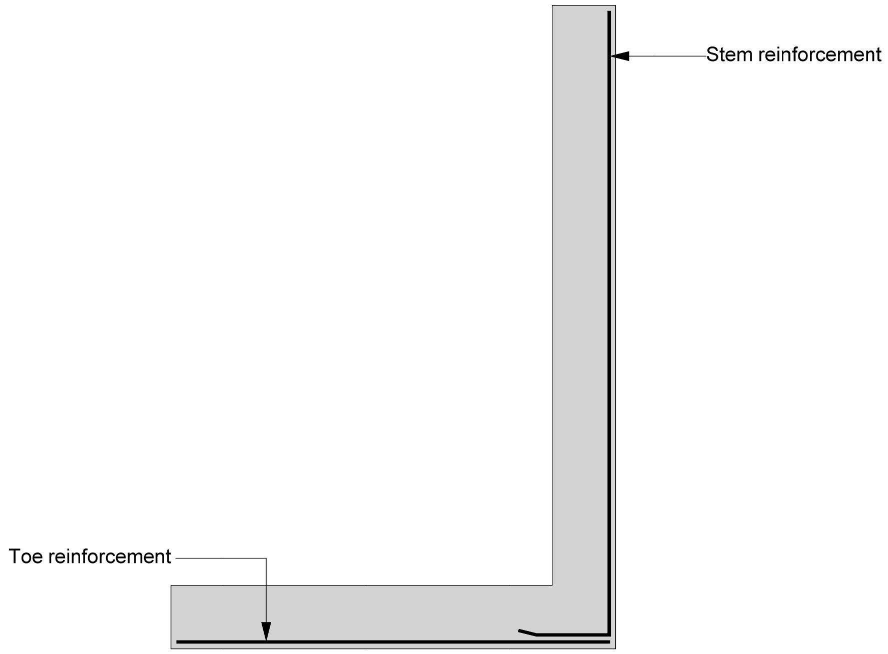
Design shear stress v_{stem} = **0.128** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.559** N/mm^2

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

Indicative retaining wall reinforcement diagram

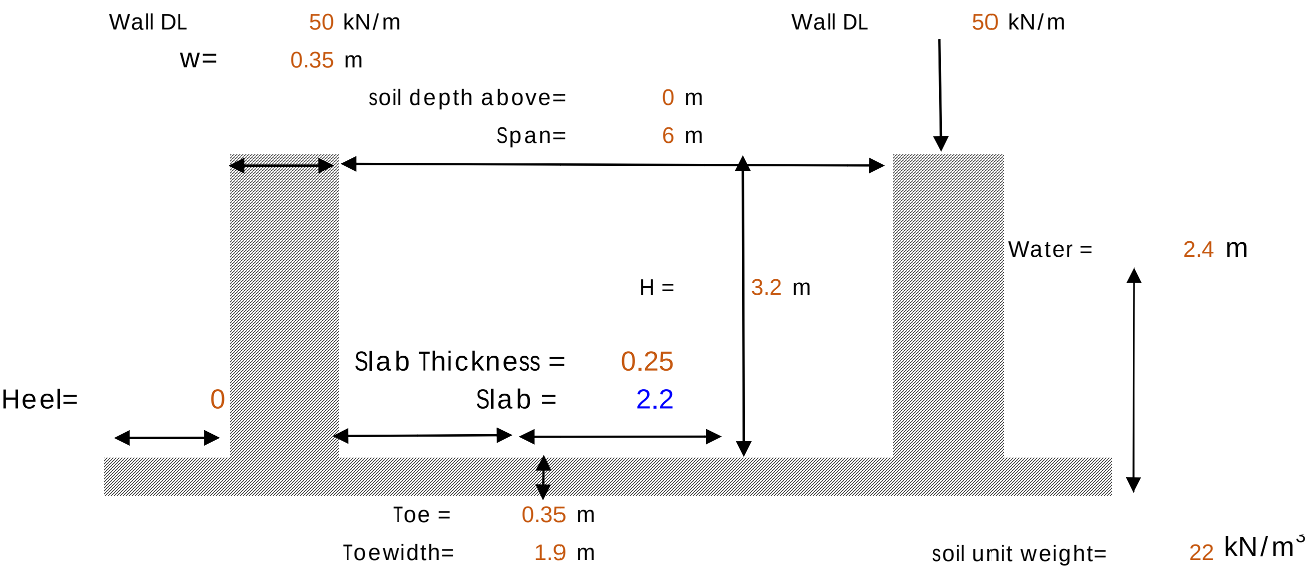


Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)
Stem bars - 12 mm dia.@ 100 mm centres - (1131 mm^2/m)

Additional Calculations

Ref

Slab Uplift



Uplift Calc

Total Dead Load =	Slab=	13.75 kN/m				
	Toe and heel =	39.375 kN/m				
	Wall =	56				
	Soil=(	0 +	0) x 2 +	0 =	0	13.76
	Total Dead load =	209.1 kN/m				
Total Uplift Force=		160.8 kN/m	f.o.s.=	**1.3 No Global Uplift**		

Slab Uplift

Slab =	6.3 kN/m	Uplift =	24
Service Moment =	-79.9 kNm/m		
Factored Design moment=	-93.5 kNm/m		
Factored Design shear =	-62.3 kN/m		

Global Heave

Weight of building =	209.1 kN/m		
Weight of soil removed =	471.7		
% change	56%	place	56% of slab area as heave protection
width of heave protection =	3.3 m	place	3.3 m of slab area as heave protection

Job Number: 150333

Ref

Lateral Thrust

Wall DL 50 kN/m ϕ 30 Wall DL 50 kN/m

w= 0.35 m δ 18

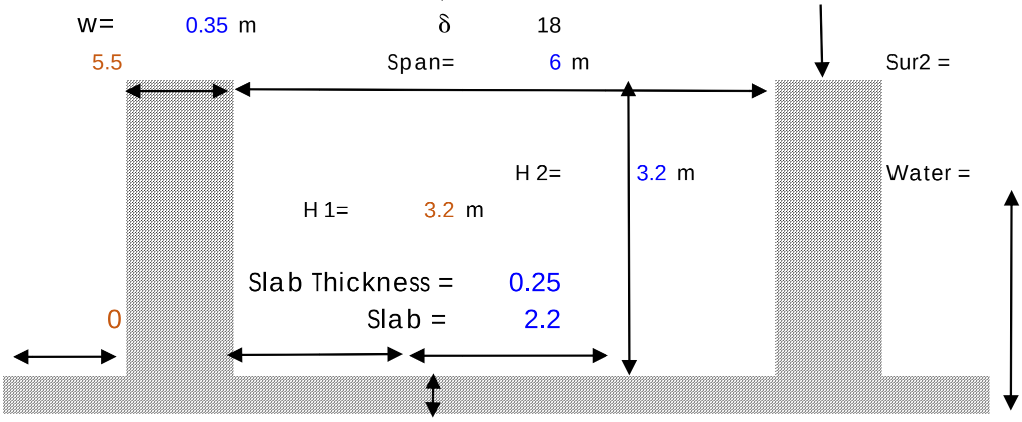
Sur l = 5.5 Span= 6 m Sur2 = 5.5

H 2= 3.2 m Water = 2.4 m

H 1= 3.2 m

Slab Thickness = 0.25

Heel= 0 Slab = 2.2



Toe = 0.35 m

Toewidth= 1.9 m

57.2958

k_p = 3

k_a = 0.33333

Thrust from left = 15.466667 Thrust from Right 15.4667

Resitance from left = 172.8 Resitance from Right = 172.8

Equilibrioum check Kp from right adequate

Kp from left adequate

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

Appendix A: Construction Method Statement

Contents

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The Coach House, 98A Priory Road

1. Basement Formation Suggested Method Statement.

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

1.2. This method statement has been written by a Chartered Engineer. The sequencing has been developed considering guidance from ASUC.

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

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

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

1.6. Prior to construction, the excavations for the basement retaining walls will be propped. This will include propping at high level which will remain in place as sacrificial propping when the concrete is cast.

1.7. The cantilever pins are designed to be inherently stable during the construction stage without temporary propping to the head. However, propping at high level should be installed to increase the safety margin during construction and to keep associated ground movements to a minimum. The base benefits from propping, this is provided in the final condition by the ground slab. In the temporary condition the edge of the slab is buttressed against the soil in the middle of the property, also the skin friction between the concrete base and the soil provides further resistance. The central slab is to be poured in a maximum of a 1/3 of the floor area.

1.8. A ground investigation has been undertaken. The soil conditions are made ground on head clay and London clay.

1.9. The bearing pressures have been limited to 120kN/m^2.

1.10. The water table is expected to be encountered at 4.2m below ground level. This is below the formation level of the basement.

2. Enabling Works

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

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

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

2.4. Dewater: Perched water may be encountered at 1.25m; any water likely to enter the excavations should be pumped away from the site.

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

3. Basement Sequencing

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

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

3.3. Place cantilevered walls 1, 2 and 3 noted on plans. (Cantilevered walls to be placed in accordance with section 4.)

3.4. Needle the bay/front wall above.

3.5. Insert steel over and sit on cantilevered walls.

3.0

3.5.1. Drypack to steelwork. Ensure a minimum of 24 hours from casting cantilevered walls to dry packing.

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

3.7. Place cantilevered retaining wall to the left side of front opening. After 48 hours place cantilevered retaining wall to the right side of front opening.

3.8. Excavate out first 1m around front opening, prop floor and erect conveyor.

3.9. Continue cantilevered wall formation around perimeter of basement following the numbering sequence on the drawing SL-10.

3.9.1. Excavation for the next numbered sequential sections of underpinning shall not commence until at least 8 hours after drypacking of previous works. Excavation of adjacent pin to not commence until 48 hours after drypacking. (24hours possible due to inclusion of Conbextra 100 cement accelerator to dry pack mix).

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

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

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

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

4. Underpinning and Cantilevered Walls

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

4.2. Provide propping to floor where necessary.

4.3. Excavate first section of retaining wall (no more than 1000mm wide). Where excavation is greater than 1.2m deep provide temporary propping to sides of excavation to prevent earth collapse (Health and Safety). A 1000mm width of wall has a lower risk of collapse to the heel face.

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

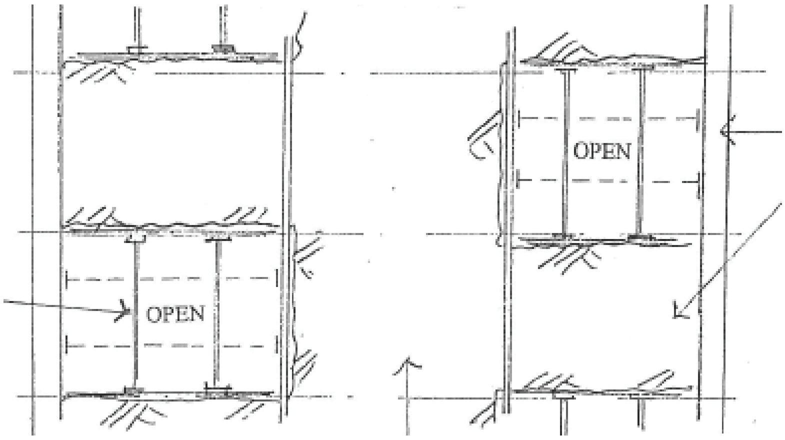

Figure 22 – Schematic Plan view of Soil Propping

Figure 23 Propping

Figure 24 Excavation of Pin

Figure 25 Completed Wall

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

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

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

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

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

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

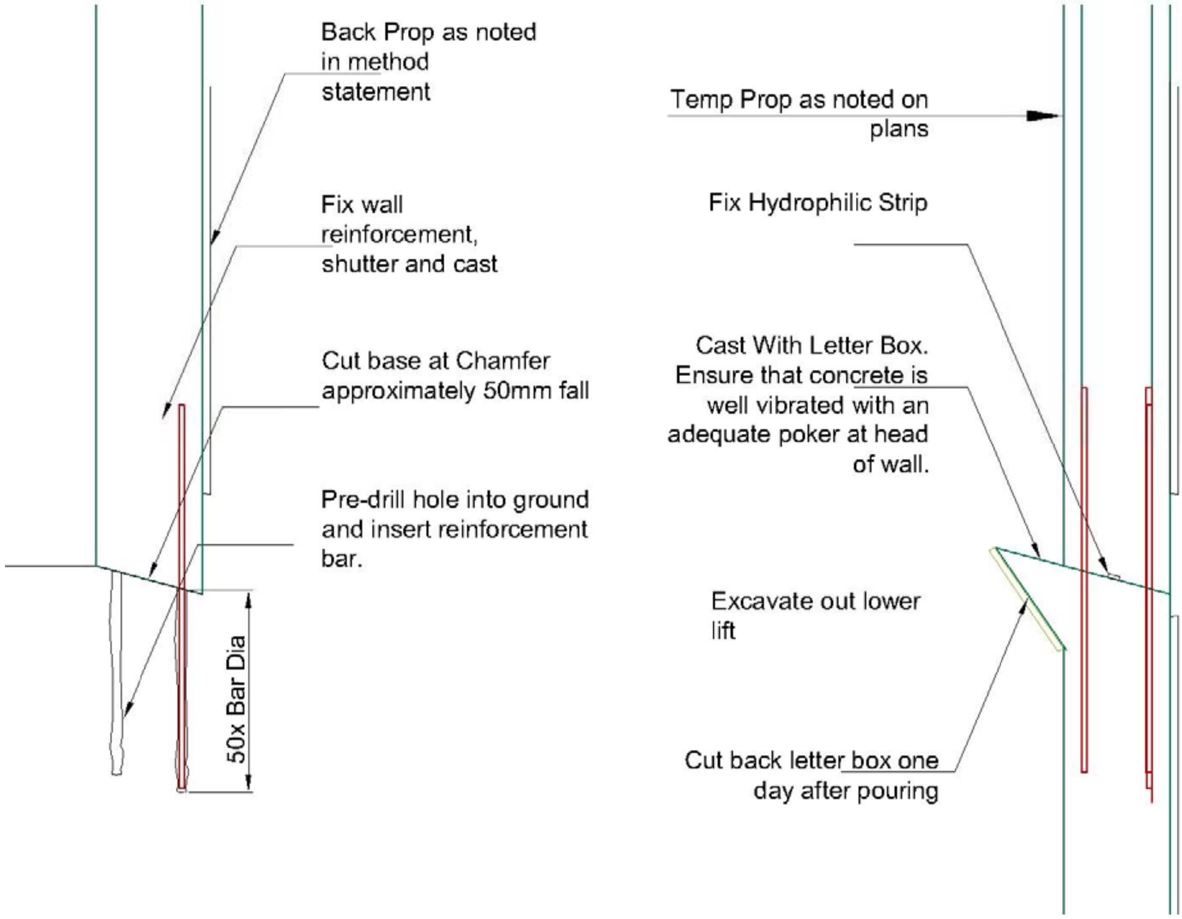
4.7. Underpinns can be completed in segmental lifts (eg top section of wall followed by bottom section of wall).

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

4.7.1. Place reinforcement for retaining wall segmental lift

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

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



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

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

4.10. Clear underside of existing footing.

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

4.12. Place blinding.

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

4.14. Cast base. (on short stems it is possible to cast base and wall at same time)

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

Ensure that Concrete is of sufficient strength, check engineer’s specifications

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

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

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

4.19. Place shuttering and pour concrete for retaining wall. Stop a minimum of 75mm from the underside of existing footing.

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

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

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

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

5. Floor Support

The materials used for the existing ground floor are to be confirmed. Support for timber and concrete floors is described here.

Timber Floor

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

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

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

Concrete Ground bearing slabs

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

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

5.6. Insert Steelwork and dry pack to underside of floor

5.7. Between steelwork place 100wide x 100dp PC lintels at a maximum spacing of 400mm

5.8. If necessary Brick up to the 50mm below underside of floor

5.9. Dry pack between lintel/brickwork to underside of slab.

5.10. Remove props

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

6. Supporting existing walls above basement excavation

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

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

6.1.2. Needles are to be spaced to prevent the brickwork above 'saw-toothing'. Where brickwork is in good condition, needles must be placed at a maximum of 1000mm centers. Lighter needles or strong boys should be placed at tighter centres under door thresholds

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

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

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

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

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

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

7. Approval

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

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

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

CROFT STRUCTURAL ENGINEERS

8. Trench sheet design and temporary prop Calculations

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

Trench sheets should be placed at close centres to deal with the ground. It is expected that the soil between the trench sheeting will arch. Looser soil will required tighter centres. It is typical for udnerpins to be placed at 1000c/c; in this condition the highest load on a trench sheet is when two 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:

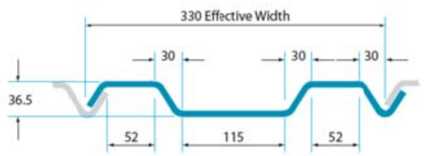
Surcharge	sur = **10.** kN/m^2	
Soil density	δ = **20** kN/m^3	
Angle of friction	ϕ = **25** °	
Soil depth	Dsoil = **3000.000** mm	
	k_a = (1 - sin(ϕ)) / (1 + sin(ϕ))	= **0.406**
	k_p = 1 / k_a	= **2.464**
Soil Pressure bottom	soil = k_a * δ * Dsoil	= **21.916**kN/m^2
Surcharge pressure	surcharge = sur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

CROFT STRUCTURAL ENGINEERS

Standard Lap Trench Sheeting

STANDARD LAP

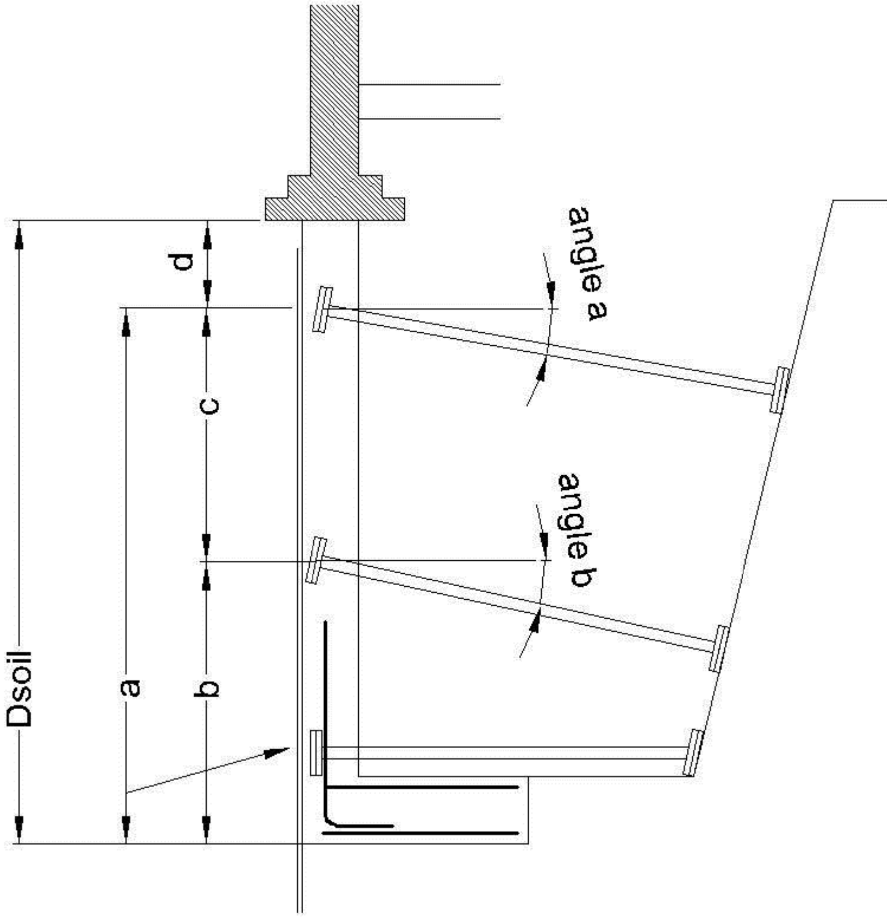
The overlapping trench sheeting profile is designed primarily for construction work and also temporary deployment.



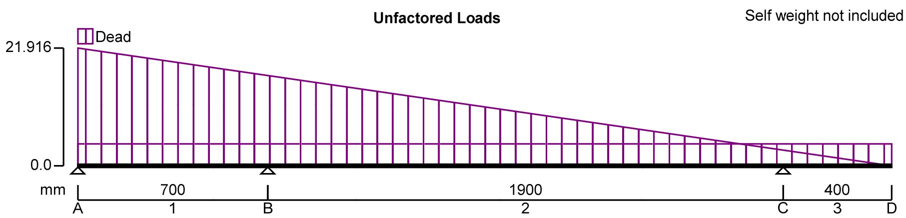
Technical Information

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

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



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



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **3**

Material Properties:

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

Support Conditions:

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

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

Span Definitions:

Span 1 Length = **700** mm Cross-sectional area = **12864** mm^2 Moment of inertia = **269.×10^3** mm^4

Span 2 Length = **1900** mm Cross-sectional area = **12864** mm^2 Moment of inertia = **269.×10^3** mm^4

Span 3 Length = **400** mm Cross-sectional area = **12864** mm^2 Moment of inertia = **269.×10^3** 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

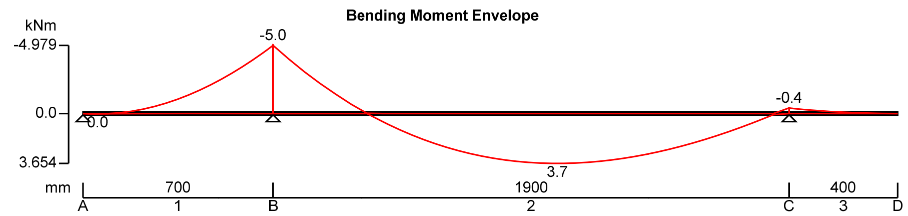
Support D Max react = **0.0** kN Min react = **0.0** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

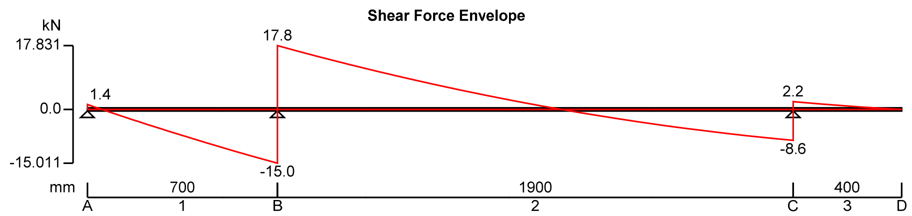
Beam Max/Min results - Combination Summary

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

Mallowable = Sxx * py * Nos = **8.745k**Nm

SRU 4.0

Safe working loads for Acrow Props — loads given in kN

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

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

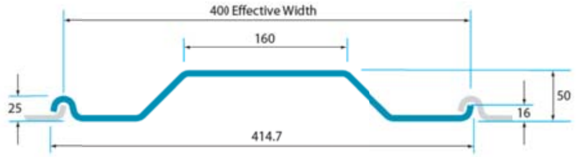
Any Acro Prop is accetpable

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

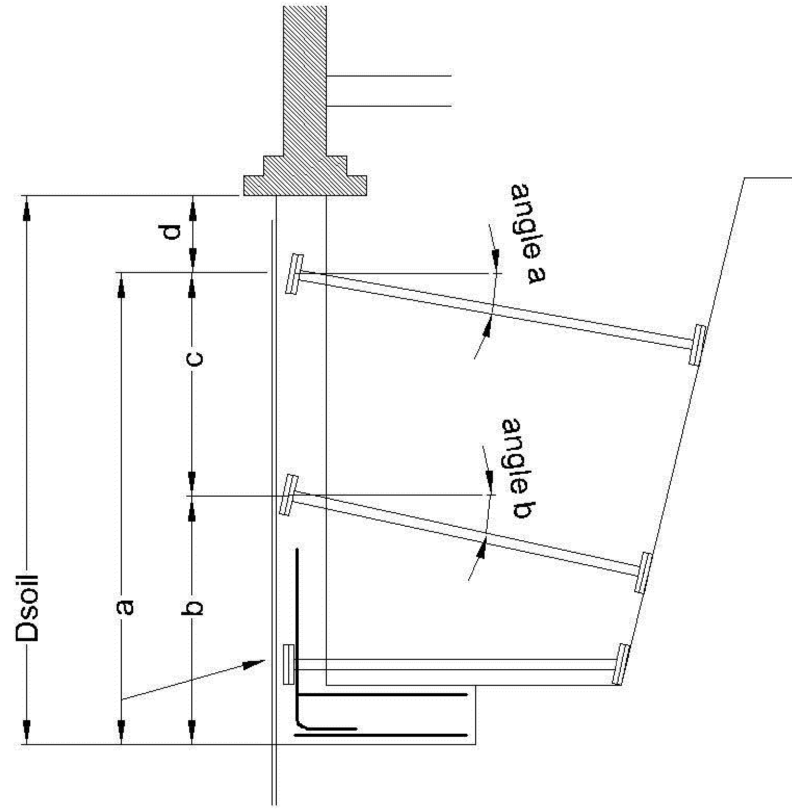
Effective width per sheet (mm)	400
Thickness (mm)	6.0
Depth (mm)	50
Weight per linear metre (kg/m)	21.90
Weight per m^2 (kg)	55.2
Section modulus per metre width (cm^3)	101
Section modulus per sheet (cm^3)	40.34
I value per metre width (cm^4)	250
I value per sheet (cm^4)	101
Total rolled metres per tonne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

Sxx = 48.3cm^3

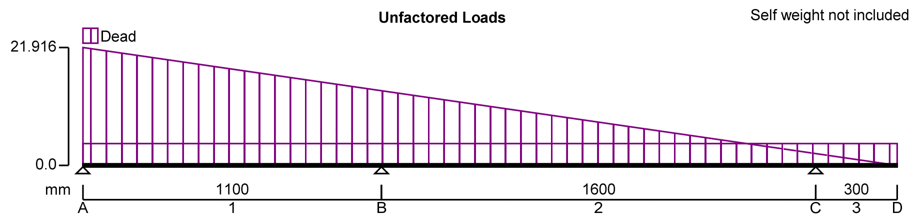
py = 275N/mm^2

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

CROFT STRUCTURAL ENGINEERS

Span 3 Length = **300** mm Cross-sectional area = **17806** mm^2 Moment of inertia = **269.×10^3** mm^4

LOADING DETAILS

Beam Loads:

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

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

LOAD COMBINATIONS

Load combination 1

Span 1 1×Dead

Span 2 1×Dead

Span 3 1×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Support Reactions - Combination Summary

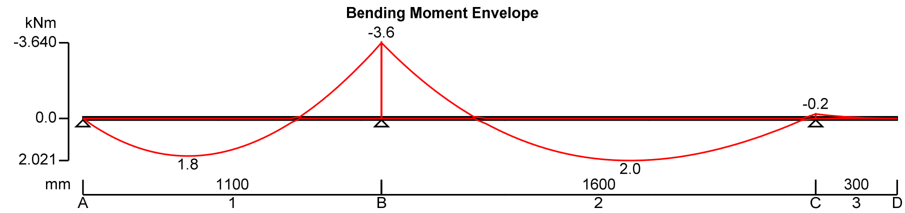
Support A	Max react = **-9.5** kN	Min react = **-9.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-28.0** kN	Min react = **-28.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-7.5** kN	Min react = **-7.5** kN	Max mom = **0.0** kNm	Min mom = **0.0**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

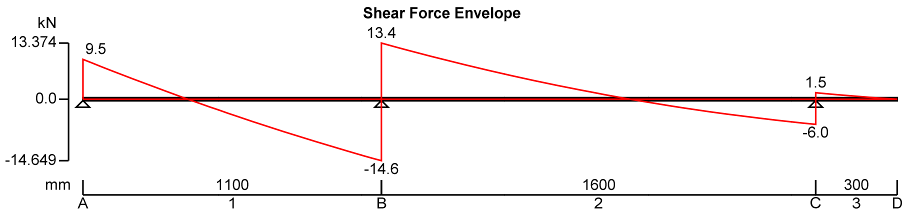
Beam Max/Min results - Combination Summary

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

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

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





Number of sheets Nos = 2

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

SRU 4-0

Safe working loads for Acrow Props — loads given in kN

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

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

Any Acro Prop is accetpable

Sheeting requirements

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

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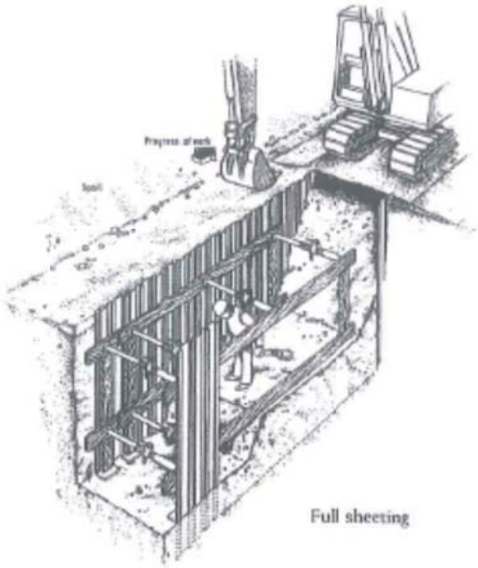
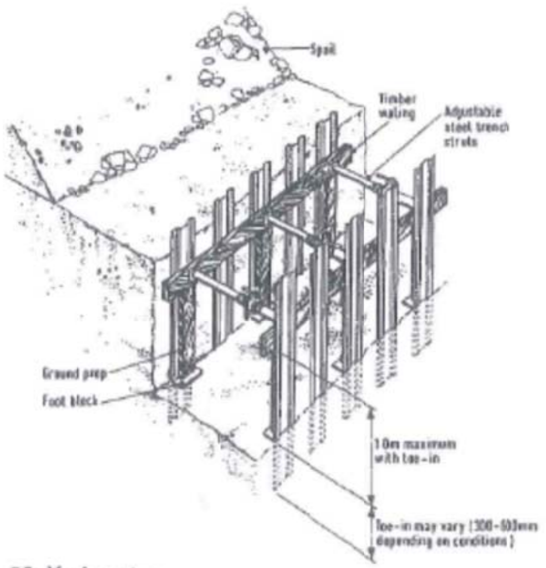
Sheeting requirements



Full sheeting

Sheeting requirements



Half sheeting
shown for 1.5 m deep trench

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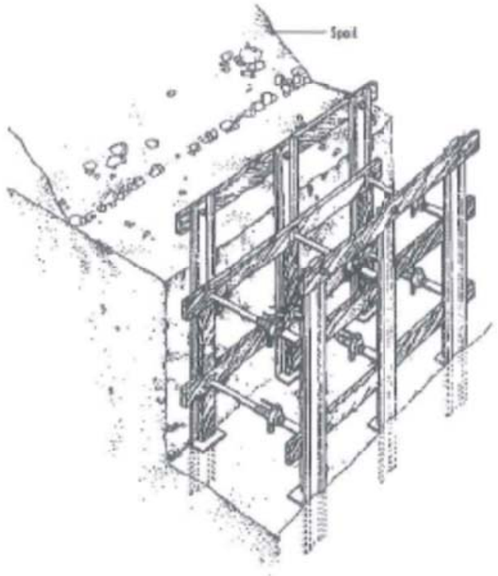
Sheeting requirements

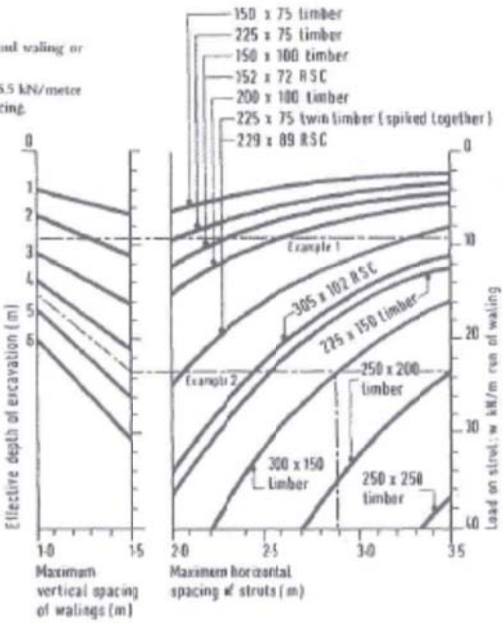


Quarter sheeting

Design to CIRIA 97

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

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



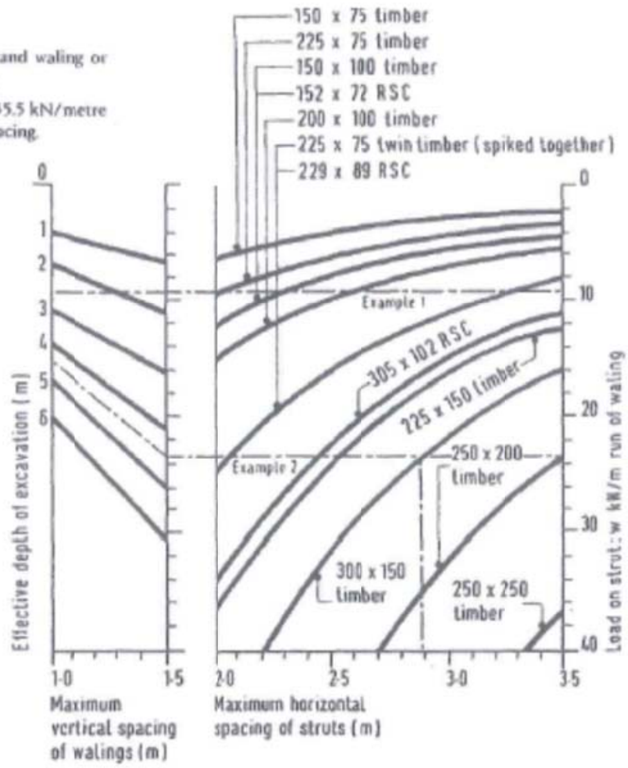
Use for:
Granular soils
Mixed soils
Short term trenches in clay
(see notes opposite)

CROFT STRUCTURAL ENGINEERS

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

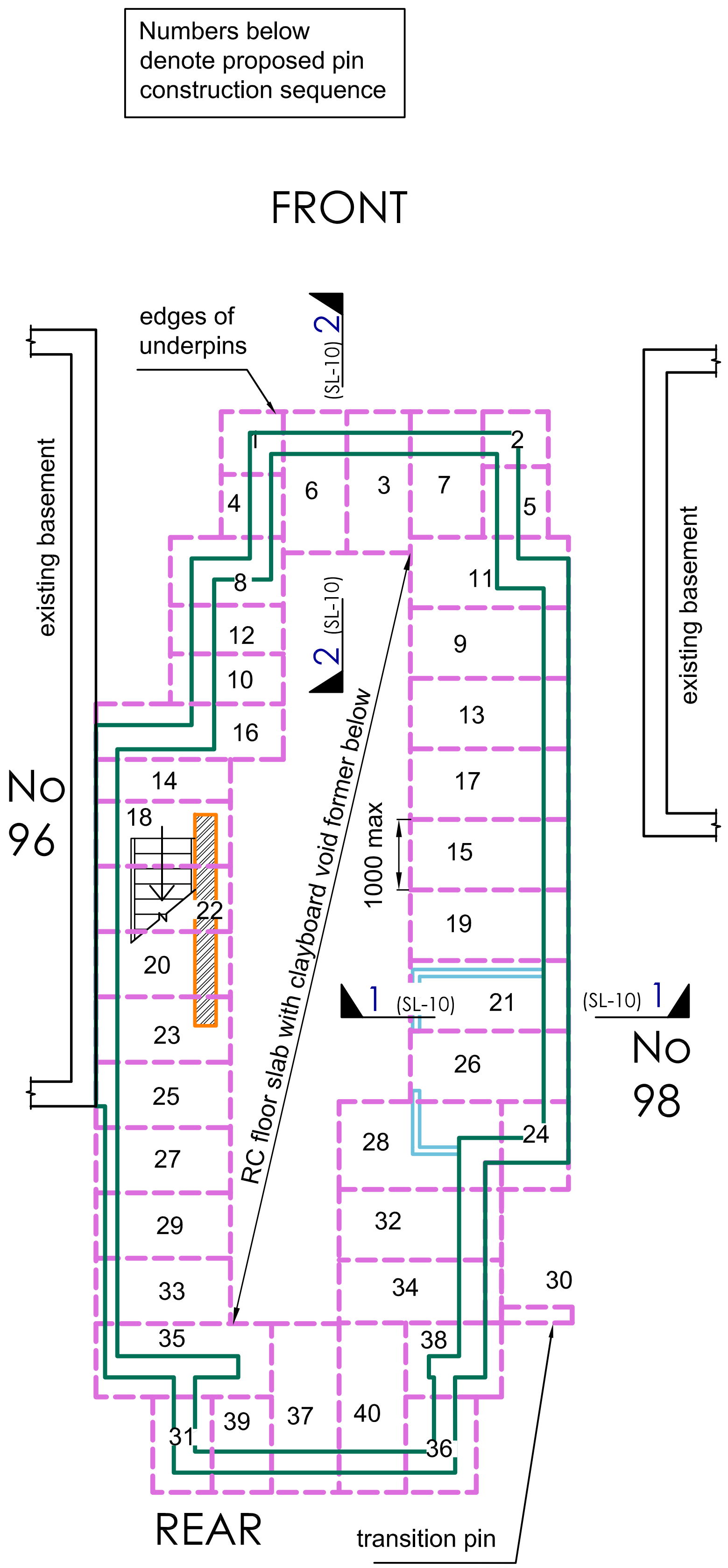
Heavy duty Speedshores have a capacity of 35.5 kN/metre run of waling at 3.2 m horizontal strut spacing.

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

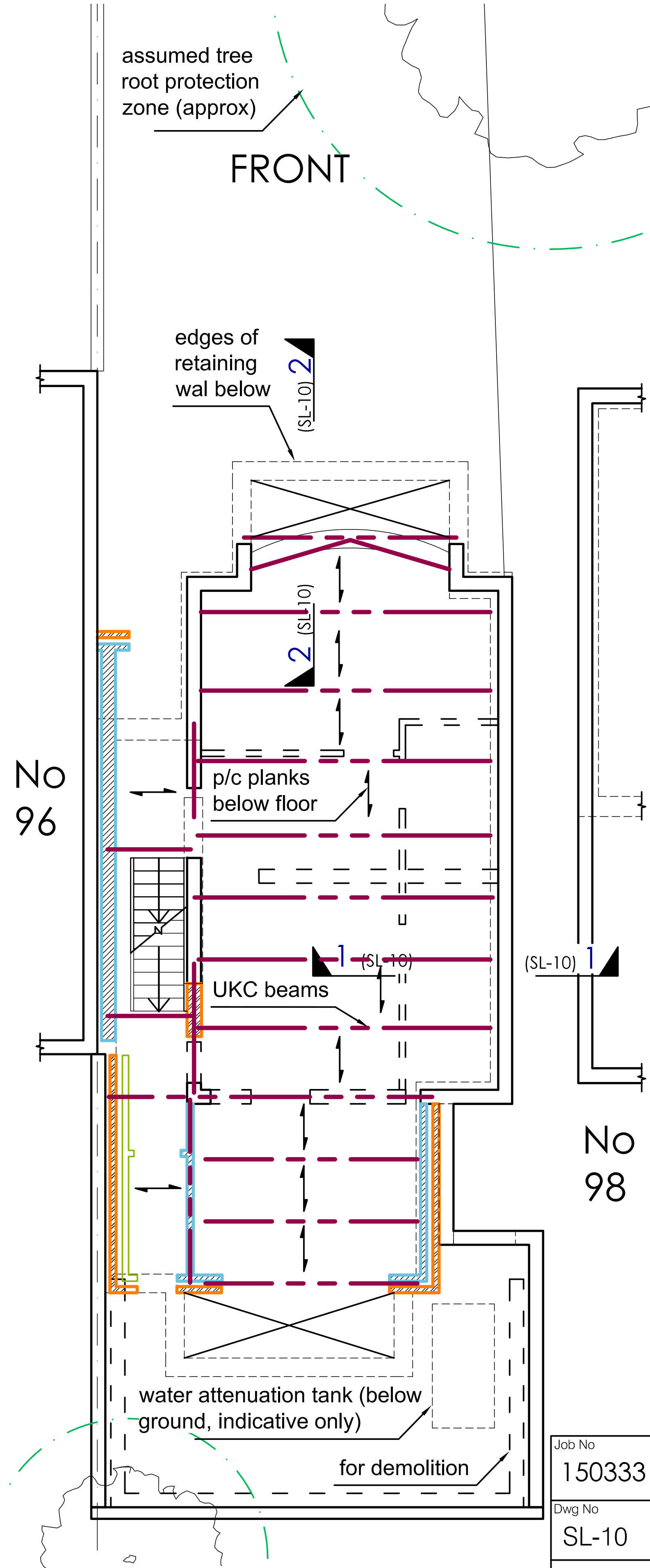


Use for:
Granular soils
Mixed soils
Short term trenches in clay
(see notes opposite)

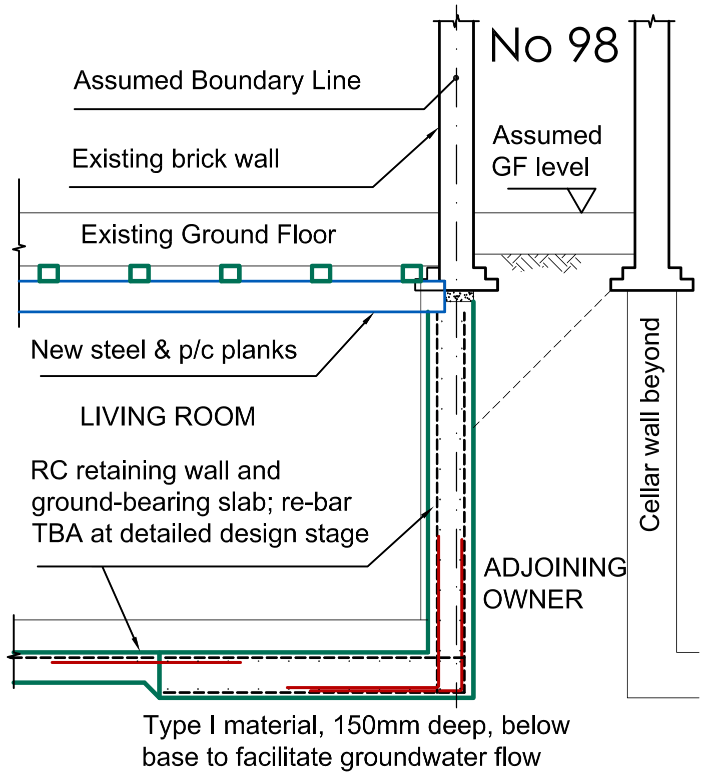
Appendix B: Structural Drawings



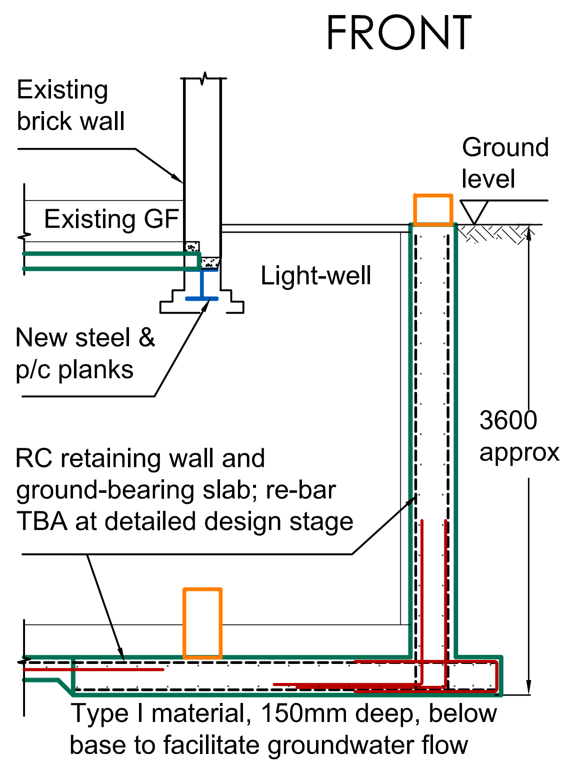
Basement Plan
Scale (1:100)



Ground Floor Plan
Scale (1:100)



Section 1-1: Below Party Wall
Scale (1:50)



Section 2-2: Front Lightwell
Scale (1:50)

PLANING ISSUE
NOT FOR CONSTRUCTION

Rev	Date	Amendments
-	19/6/15	First issue for planning

Job No: 150333	Client: Mr Michael Cohen
Dwg No: SL-10	Project: 98A Priory Road
Date: May 15; Drawn: SB; Chk'd: GW	Title: Proposed Basement Scheme Design
Scale: As shown @ A3; Rev: -	

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

Appendix C: Proposed pump mechanism

Final selection and design to be confirmed at detailed design stage

SUMPFLO TWIN BBPS

Twin discharge standard

The SumpFlo Twin BBPS™ is specially designed for the removal of groundwater from basement cavity drainage membrane systems. The system comprises of a polyethylene tank, locking access cover (pedestrian duty, not suitable for roadways), 2no. powerful submersible pumps and 24V backup pump. The system is very versatile, enabling the installer to locate inlets to their specifications.

The system comes complete with a battery back-up pump system, which is designed especially for where the possibility of primary/ secondary pump failure through either a pump fault or loss of mains power would be catastrophic. The system acts as a back-up that will alert the end user if the water rises above the normal operating level within the tank and will activate a 24V back-up pump. Comes standard with twin discharge.

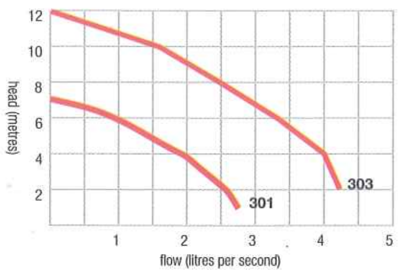
MODELS

- SumpFlo Twin BBPS (301)
- SumpFlo Twin BBPS (303)

TECHNICAL DATA

MODEL	301	303
Power Supply	230V AC	230V AC
Rated Current	1.9A	4.9A
Motor Rating	180W	500W
Frequency	50Hz	50Hz
Revs Per Minute	2720rpm	2800rpm
Max. Vert. Output	7m	12m
Max. Horiz. Output	50m	100m
Max. Flow Rate	168l/m	240l/m
Max. Liquid Temp.	<40°C	<40°C
Discharge Size	32mm	32mm
Cable Length	5m	5m
Weight	34kg	35kg
Colour	Yellow	Yellow

PUMP CURVE



DIMENSIONS

MODEL	SUMPFLO TWIN BBPS
Height / Diameter (mm)	600 x 600
Clear opening (mm)	350 x 350

KEY FEATURES

- Easy to install
- Odour tight locking access cover
- Variable inlet positions
- Integral non-return valve preventing back flow
- Durable polyethylene tank
- Pre-moulded flotation points preventing movement below ground
- Integral step for dual pump setup
- Powerful submersible pumps

Visit www.wykamol.com for full technical drawings

ACCESSORIES

Basement monitoring system

110mm rubber seal inlet

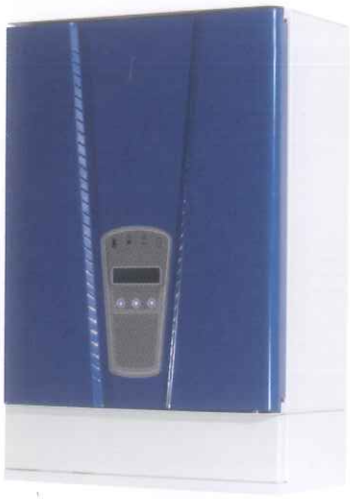
50mm rubber seal inlet

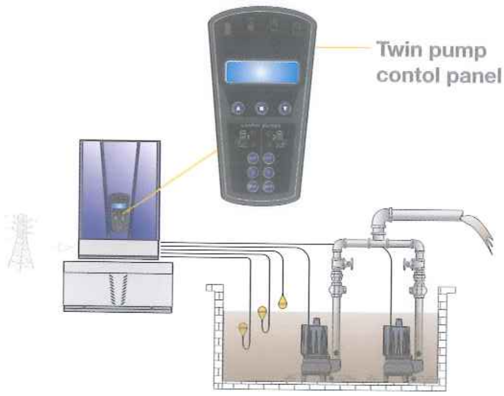
Inlay access cover

Battery back-up

Telemetric system

POWERSAFE





Available in single or twin pump configurations

The PowerSafe® stores power in case of a mains power loss, protecting your property from flooding. Don't worry, the PowerSafe® is completely automatic!

The PowerSafe range of fully automatic battery back up systems are designed to offer your customer peace of mind for their basement drainage. The system can be used on single or dual pump configurations and is suitable for both groundwater and foul waste. It comes complete with a built in GSM telemetry for complete monitoring of your pumped drainage system (not available on the PowerSafe ECO).

The PowerSafe is suitable for installing either at the initial building stage, or retrofitting to existing buildings. The system comprises of a PowerSafe control panel, cable cover, battery holder and batteries (no. of batteries to be specified at time of order).

SUITABLE FOR

- All pumping stations (below and above ground)

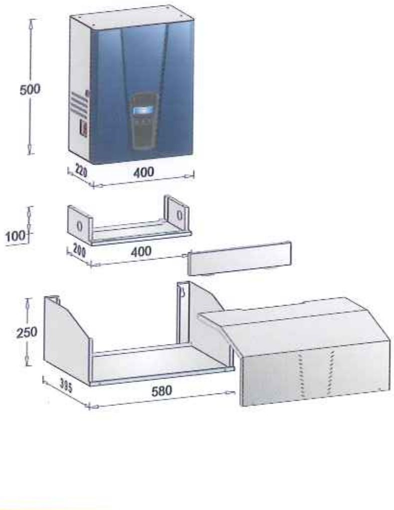
KEY FEATURES

- Hand/Off/Auto switches
- High Level Audible Alarm with Mute Button
- Duty/Assist Configuration (Alternates switching of pumps)
- Visual Indication for: (Supply On, Pumps Running, Pumps Tripped & HL)
- Available in both single & dual pump configuration.
- Single-phase power supply 230V
- Automatic battery charger
- LCD backlit display*
- Hour run meter*
- No. Start meter*
- Overload protection
- Reporting of scheduled maintenance*
- Automatic load TEST*
- General alarm output
- Removable front door
- Steel box painted with powder, protection IP31
- Integral telemetric GSM module*

* Not available on PowerSafe ECO.

† PowerSafe ECO: The battery is included in the control panel. For dimensions consider only the control panel.

DIMENSIONS †



HIGH LEVEL ALARM (MAINS & BATTERY OPERATED)

MODELS

- Microswitch float switch (suitable for ground water)
- Sewage float switch (suitable for foul water)

KEY FEATURES

- Visual lamp
- Loud 24V sounder
- Mute button
- Mechanical float switch

SUITABLE FOR

- SumpFlo
- SumpFlo Twin
- Sump Flo Pro
- Sump Flo Pro Twin
- DrainFlo 150
- DrainFlo 200

Should there be a power failure the trickle charge battery will automatically take over. This unit should be wall mounted and will notify clients of a high water level via an audio and visual signal. The unit can be supplied with either a microswitch float switch suitable for ground water systems, or a sewage float switch suitable for foul water systems.

DIMENSIONS

MODEL	HIGH LEVEL ALARM 9V
Height / Width / Length (mm)	200 / 250 / 90

BASEMENT MONITORING SYSTEMS & BATTERY BACK UP

KEY FEATURES

- Compact
- Multi-function
- Remote notification

SUITABLE FOR

- SumpFlo
- SumpFlo Twin
- SumpFlo BBPS
- SumpFlo Twin BBPS
- Sump Flo Pro
- Sump Flo Pro Twin

Designed to monitor automatic water sump pumps, it is wired in the power line to the pump. It also has the option to add a high level water alarm indicator. A built-in alarm relay can signal a home automation system, house alarm or a remote dialler.

DIMENSIONS

MODEL	BMS-BB
Height / Width / Length (mm)	90 / 158 / 60