Job Number: 170823

CROFT STRUCTURAL ENGINEERS

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Basement Impact Assessment

Property Details
6 Parsifal Road
London
NW6 1UH

Client Information
Mr & Mrs Wilcke

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) Francis Williams M.Geol. (Hons) FGS CGeol CEnv AG	(Separate Report) Francis Williams M.Geol. (Hons) FGS CGeol CEnv AG

Revision	Date	Comment
-	27/11/2017	First Issue
1	28/11/2017	Minor revisions
2	29/11/2017	Minor revisions to section 4

Contents

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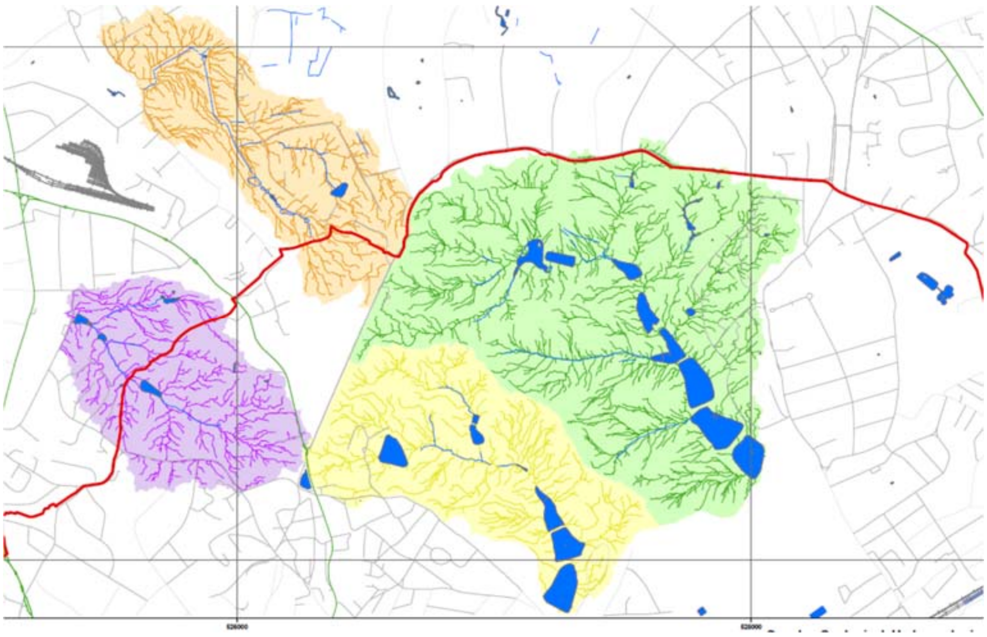
Executive (non-technical) Summary

	The London Borough of Camden requires a Basement Impact Assessment (BIA) to be prepared for developments that include basements and lightwells. This document forms the main part of the BIA and gives details on the impact of surface water flow. The scheme design for the proposed subterranean structure is also included. This document should be used in conjunction with the Ground Investigation and Basement Impact Assessment report (dated November 2017 and with Ref. GWPR2280/GIR/November 2017). This is a separate report and is referred to, where relevant, within this document. This BIA follows the requirements contained within Camden Council's planning guidance CGP4 – Basements and Lightwells (2015). In summary, the council will only allow basement construction to proceed if it does not: • cause harm to the built or natural environment and local amenity • result in flooding • lead to ground instability. In order to comply with the above clauses, a BIA must undertake five stages detailed in CPG 4. This report has been produced in line with Camden planning guidance and associated supporting documents such as CPG1, DP23, DP26, DP25 and DP27. Technical information from 'Camden geological, hydrogeological and hydrological study - Guidance for subterranean development', Issue 01, November 2010 (GSD, hereafter) was also used and is referred to in this assessment.
Existing Property	The site comprises a semi-detached four-storey residential building with lower ground, ground, first and second floor levels with paved off-street parking area and paved walkways to the front. There is a side access to the property on the north-eastern side of the property which leads to the private rear garden. The front paved off-street parking area is approximately ~2.00m lower than the rear garden level of the property.
Proposed Development	The proposed development involves a 300mm deepening of the existing basement beneath the front of the building and the construction of a new basement beneath the rear of the existing property. The new basement will be completely below the existing building.

	 *Figure 1: Map / Aerial view with approx. site area indicated*
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.
Stage 2 – Scoping	The Scoping stage identified the potential impacts and set the parameters required for further study of the areas of concern highlighted in the Screening phase. A desk survey was 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 desk survey was completed by an engineer. The information from this was utilised to confirm the requirements of a ground investigation. A structural engineer inspected the building to determine the current condition of the property. Visual inspections were completed of the adjacent properties to determine if there were signs of structural movement.

	The neighbouring land has not been excavated but an engineer has assessed the age of the adjacent properties and considered the type of foundations used for that period and assumed these in the design. A ground investigation was completed. This confirmed the presence of 1000mm of made ground, which overlies clay. Laboratory testing was undertaken on the soil samples. Ground water has been measured over repeat visits to determine water levels and flows. The highest groundwater reading was at 1.25m below ground level. This is not believed to be indicative of the ground water table but due to seepages of perched water.
Stage 4 – Impact Assessment	Engineering Considerations are presented within the Land Stability BIA and Groundwater BIA (dated November 2017 Ref. GWPR2280/GIR/November 2017). This section incorporates an Impact Assessment. A ground movement Assessment is included. The movement assessment of the basement and its construction 'Negligible' to 'Very Slight' on the Burland scale. Proposals for the temporary works are presented by Croft to ensure that any movement is within these limits.

1. Screening Stage

	This stage identifies any areas for concern that should be investigated further.
Land Stability	Refer to the assessment on Land Stability by Ground & Water.
Subterranean Flow	Refer to the assessment on Groundwater by Ground & Water.
Surface Flow and Flooding	The questions below are taken from the Camden CPG 4 – Basements and Lightwells.
	Question 1: Is the site within the catchment of the pond chains on Hampstead Heath? **No**. The Highgate chain catchment is indicated by coloured shaded areas on Figure 14 of the GSD. An extract of this is reproduced below. The site lies outside of these areas.  *Figure 2: Extract from Figure 14 of the GSD*

Question 2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route?

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

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

No. The amount of hard standing will remain unchanged

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

No – The surface water that flows from the proposed development will be routed the same way as before: water collected hard surfaced areas will enter the existing 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. At present surface water does no flow from the site to adjacent properties and downstream water courses. This will remain the case when the new basement is complete. The quality of water received by adjacent properties and downstream water courses will not be affected.

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?

No. The potential sources of flooding are summarised below:

Potential Source	**Potential Flood Risk at site?**	**Justification**
Fluvial flooding	No	EA Flood Mapping shows Flood Zone 1. Distance from nearest surface watercourse >1km

Tidal flooding	No	Site location is 'inland' and topography > 40mAOD.
Flooding from rising / high groundwater	No	The site is located on low permeability London Clay.
Surface water (pluvial) flooding	No	6 Parsifal Road, NW6 1UH is not noted on the flood street list and maps from 1975 or 2002
Flooding from infrastructure failure	Yes	Drainage at or near the site could potentially become blocked or cracked and overflow or leak. Drainage of the basement terrace areas may rely on pumping.
Flooding from reservoirs, canals and other artificial sources	No	There are no reservoirs, canals or other artificial sources in the vicinity of the site that could give rise to a flood risk.

The answers to Questions 1-5 above indicate that the issues related to surface water flow and flooding are not significant. These questions therefore do not have to be carried forward to Scoping Stage.

In answering Question 6, a flood risk assessment is not considered necessary: the property is not on a street that has flooded in 1975 or 2002 and there are no risks to flooding that are greater than those inherent with all subterranean structures. However, the risks associated with infrastructure failure should be investigated further. The assessment, with regards to Surface Water Flow, should be carried forward to Scoping Stage.

Carry forward to Scoping Stage.

2. Scoping Stage

	This stage identifies the potential impacts of the areas of concern highlighted in the Screening phase.
Land Stability	Refer to the assessment on Land Stability by Ground & Water Ltd.
Subterranean Flow	Refer to the assessment on Groundwater by Ground & Water Ltd.
Surface Flow & Flooding	**Conceptual Model** The basement is under the footprint of the property and will therefore not affect the above ground flow. It is evident from the screening study that the only significant flood risk associated with the development is due to the failure of existing sewers in the vicinity of the site. The flow paths of surface water around the property should be investigated further. Carry forward to Site Investigation & Desk Study

Ground Investigation

Ground Investigation Brief	A ground investigation is required. This should cover: • 2 boreholes • Stand pipe to be inserted to monitor ground water. • Indication of soil type • Site testing to determine in-situ soil parameters. • Laboratory testing to confirm soil make up and properties. • Report on soil conditions.

3. Site Investigation and Desk Study

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

Desk Study and Walkover Survey

A structural engineer from Croft visited the property on 7th September 2017

Site & Existing Property

The site comprises a semi-detached four-storey residential building with lower ground, ground, first and second floor levels with paved off-street parking area and paved walkways to the front. There is a side access to the property on the north-eastern side of the property which leads to the private rear garden. The front paved off-street parking area is approximately 2.00m lower than the rear garden level of the property.

Hardstanding

The site comprises a plot of paved off-street parking area at the front of the property.

Figure 3: Front paved parking area

Trees and Vegetation

No trees or any vegetation will be affected by the proposed development.

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Site Drainage

A structural engineer from Croft Structural Engineers visited the site on 7th September 2017. Rainwater pipes discharge water to external drainage channels and gullies.

Proposed Development

The proposed development involves a 300mm deepening of the existing basement beneath the front of the building and the construction of a new basement beneath the rear of the existing property. The basement is anticipated to be formed at ~2.30 – 2.50m below the rear garden level.

In addition to the basement area, this also includes areas that are likely to be temporarily occupied for construction purposes.

Figure 4: Aerial view with approx. site area indicated

The outline proposed construction sequence is appended to this report.

Listed Buildings and Conservation Areas

The existing building is not listed. Data from Historic England shows that there are no listed buildings close by.

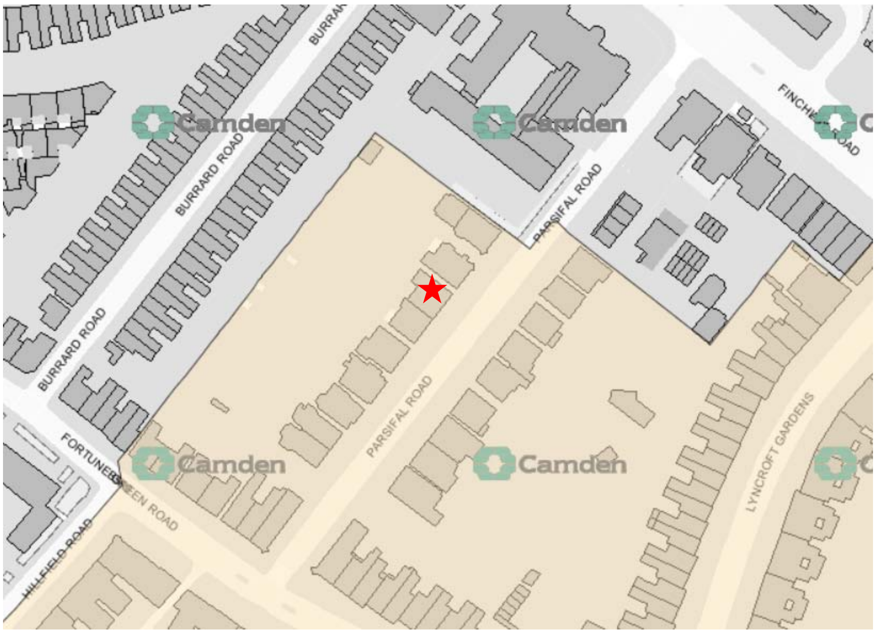
	 *Figure 5: Extract showing listed buildings* The site is within a conservation area.  *Figure 6: Extract showing building within conservation area*
Local topography & external features	The area surrounding the property has a general slope, downwards from north-east to south-west. The slope is gradual; there are no retaining walls for sudden changes in elevation.

Figure 7: Hard standing area

The walk over survey has confirmed that there are no surface water features (natural or man-made) within the site or on the adjacent sites.

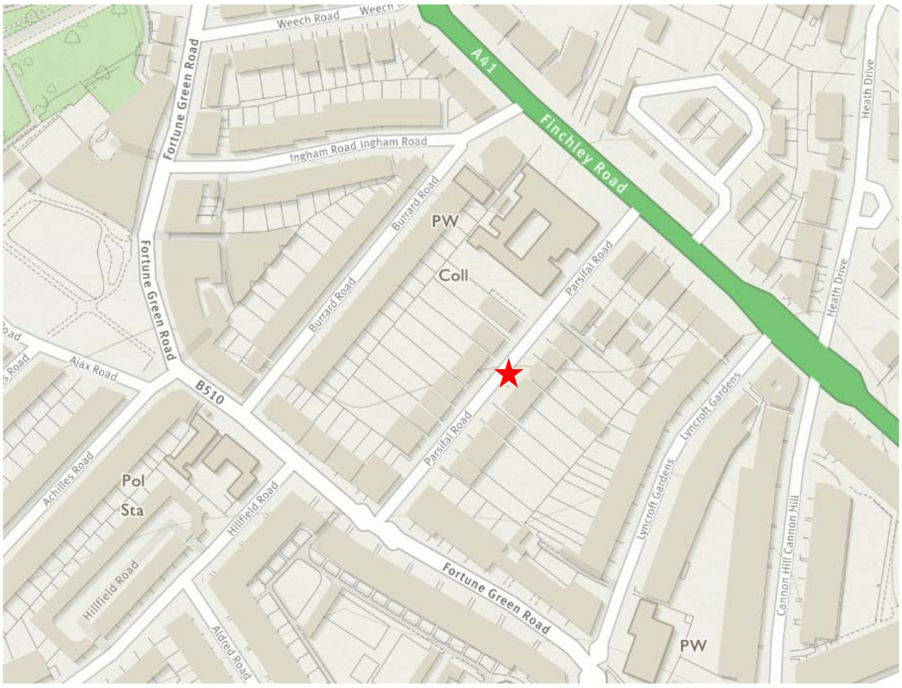


Figure 8: Extract from OS map, with location of site indicated

Geology

Site is underlain with made ground. Clay is present below this. More details are presented in the Ground Investigation report and the Hydrogeological

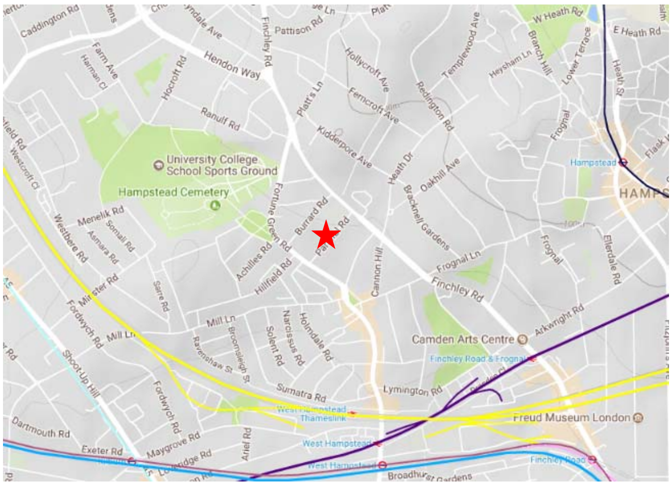
	and Land Stability assessment by Ground and Water Ltd.
Highways & public footpaths	The site is not within 5m of the public highway.
London Underground and Network Rail	 *Figure 9: Map showing rail line and location of property* The site is more than 100m away from the nearest national rail line and the nearest subterranean train line. These are unlikely to be affected by the new basement.
UK Power Networks	There are no significant items of electrical infrastructure (such as pylons or substations) in the immediate vicinity.
Proximity of Trees	There are trees close by, in the neighbouring land. These do not have tree preservation orders. The closest tree is more than 10m away from the outline of the proposed basement. This is located to the front of the property on the public pathway outside No. 4 Parsifal Road.
	## Adjacent Properties The external facades of the neighbouring properties have been inspected.

	 *Figure 10: Plan view of site (approx. area outlined in red) and the surrounding properties* Descriptions of the properties below are given in an anti-clockwise order starting from the neighbouring land to the north.
Nos 8 – Property 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? : an existing lower ground floor level is present as per the one already recorded for No. 6 Parsifal road.

	 *Figure 11: front of No 8 Parsifal Road* Structural Defects Noted: None noted Structural assessment of ongoing movement: from an observation of the external façade of the building, there were no visible signs of movement.
Nos 4 – Property to Right	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? : No basement is believed to be present in this property however there is an existing lower ground floor level as noted on the front of 6 Parsifal road.

Figure 12: front of No 4 Parsifal Road

Structural Defects Noted: None noted

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

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 by Ground and Water Ltd.

4. Basement Impact Assessment

Subterranean Flow and Land Stability	Impacts relating to Land Stability and Groundwater are described within the BIA produced by Ground & Water Ltd. Proposed measures to mitigate these, which should be developed further at detailed design stage, are presented in this section.
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.
Surface water flow and flooding	As described in previous sections, the only significant risk of flooding is from failure of infrastructure, such as flooding due to unexpected failure of the drainage, water mains, etc. This risk is inherent in the construction of all subterranean structures. In the detailed stage design this is taken into consideration by design the retaining walls to resist the hydrostatic pressures from such failures. There is a risk of flooding due to the failure of the pumping system but this can be reduced to acceptable levels with appropriate design and installation measures. Measures to mitigate this risk are described later under 'Initial Design Considerations'.

Ground Movement Assessment & Predicted Damage Category

	The design and construction methodology aims to limit damage to the existing building on the site, and to the neighbouring buildings, to Category 2 or lower as set out in Table 6.4 of CIRIA report C760. For this development, suitable temporary propping during the construction phase will limit the amount of movement due to the basement works. This is described in the Basement Method Statement (appended). The ground movement assessment (GMA) is contained within the Land Stability BIA by Ground & Water with reference GWPR2280/GIR/November 2017.

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Mitigation Measures Ground Movement

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

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

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

To reduce the risk to the development:

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

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

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Monitoring of Structures

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 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

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

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

Recommended levels are shown within the proposed monitoring statement (appended).

Basement Design & Construction Impacts and Initial Design Considerations

Foundation type	Reinforced concrete underpinning strip section will form the new foundations to the front part of the property and where the existing basement is present. This will allow for the existing basement to be lowered by approximately 300mm below the existing ground floor level. Reinforced concrete cantilevered retaining walls will form the new foundation of the property to the rear section of the property. The design of the retaining walls was calculated using software by TEDDS. The software is specifically designed for retaining walls and ensures that the construction 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 structure uses K_0 values. This approach minimises the level of movement from the concrete affecting the adjacent properties. The investigations highlight that perched water is present. The walls are designed to resist the hydrostatic pressure. The water table was recorded as low. The design of the walls considers long term scenarios. It is possible that a water main may break causing a local high water table. To account for this, the wall is designed for water approximately 1m from the top of the wall. The design also considers floatation as a risk. The design has accounted 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.
Intended use of structure and user requirements	Family/domestic use (1 dwelling)
Loading Requirements (EC1-1)	see table below
Part A3 Progressive collapse	Number of Storeys: 3 + lower ground floor **Is the Building Multi Occupancy?** No

Loading Requirements (EC1-1):

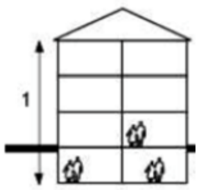
	UDL kN/m^2	Concentrated Load kN
Domestic Single Dwellings	1.5	2.0

Class 1	Single occupancy houses not exceeding 4 storeys

Change of use

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

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



3 storey over basement

Lateral Stability

Exposure and wind loading conditions

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

Stability Design

The cantilevered walls are suitable for carrying the lateral loading applied from above.

Lateral Actions

Below ground level, the reinforced concrete retaining walls are designed to carry the lateral loading applied from above.

The lateral earth pressure exerts a horizontal force on the retaining walls. The retaining walls will be checked for resistance to the overturning force this produces.

Lateral forces will be applied from:

- Soil loads
- Hydrostatic pressures
- Surcharge loading from behind the wall

These produce retaining wall thrust. This will be restrained by the opposing retaining wall.

Retained soil

Design overall stability to K_a & K_p values. Lateral movement necessary to

Parameters	achieve K_a mobilisation is height/500 (from Tomlinson). This is tighter than the deflection limits of the concrete wall.
Water Table	**Has a soil investigation been carried out?** Yes <u>Known water table from boreholes</u> Design temporary condition for water table level, If deeper than basement ignore. Design permanent condition for water table level: If deeper than existing, design reinforcement for water table at full basement depth to allow for local failure of water mains, drainage and storm water. Global uplift forces can be ignored when the water table is lower than the basement. BS8102 only indicates guidance.
Additional loading requirements	**Surcharge Loading** The following will be applied as surcharge loads to the front/ front lightwell retaining walls: • 10kN/m^2 if within 45° of road • 100kN point loads if under road or within 1.5m • 5kN/m^2 if within 45° of Pavement • Garden Surcharge 2.5kN/m^2 + 1 m of soil (if present above basement ceiling) 20kN/m^2 • Surcharge for adjacent property 1.5kN/m^2 + 4kN/m^2 for concrete ground bearing slab <u>Highways loading:</u> The basement is not within 5m of the pavement or the public highway. <u>Adjacent Properties:</u> All adjacent property footings within 45° to have additional geotechnical engineers input. A line at 45° from the base of the neighbours' wall footing would be intersected by the basement retaining wall. This should be accounted for in the design. The appended calculations show the design of one of the most heavily loaded retaining wall. The most critical parameters have been used for this.
Mitigation Measures - Internal Flooding	To mitigate the risks associated with flooding, Croft would recommend the following mitigation measures: • To reduce the likelihood of flooding into the lightwells, these should be designed (at detailed design stage) with upstands above

	ground level. • 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 was to occur, the build-up of water would be gradual and noticeable before it becomes a significant life-threatening hazard. • The pumping system should be a dual mechanism 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 (if practicable) should be considered at detailed design stage. • Route all electrical wiring at high level
Mitigation Measures - Drainage and Damp-proofing	The design of drainage and damp-proofing is not within the scope of this assessment and would not normally be expected to be part of the structural engineer's remit at detailed design stage. A common and anticipated detailed design stage approach is to use internal membranes (Delta or similar). These will be integral to the waterproofing of the basement. Any water from this will enter a drainage channel below the slab. This will be pumped and discharged into the exiting sewer system. It is recommended that a waterproofing specialist is employed to ensure all the water proofing requirements are met. The waterproofing specialist must name their structural waterproofer. The structural waterproofer must inspect the structural details and confirm that he is happy with the robustness. Due to the segmental construction nature of the basement, it is not possible to water proof the joints. All waterproofing must be made by the waterproofing specialist. He should review the structural engineer's design stage details and advise if water bars and stops are necessary. The waterproofing designer must not assume that the structure is watertight. To help reduce water flow through the joints in the segmental pins, the following measures should be applied: • All faces should be cleaned of all debris and detritus • Faces between pins should be needle hammered to improve key for bonding • All pipe work and other penetrations should have puddle flanges or hydrophilic strips

Mitigation Measures - Localised Dewatering	Monitor water levels 1 month prior to starting on site and throughout the construction process. Localised dewatering to pins may be necessary.
Temporary Works	Walls are designed to be temporarily stable. Temporary propping details will be required for the ground and this must be provided by the contractor. Their details should be forwarded to the design stage engineer. Particular attention should be paid to point loads from above. Critical areas where point loads are present from above include: Cross walls Chimney Stacks Door openings Water levels should be monitored for at least one month prior to starting on site and throughout the construction process. Localised dewatering to pin excavations may be necessary. <u>Construction Management</u> The site is in within a conservation area. Camden Council will require a management plan for construction, construction traffic and demolition. This should be developed at the detailed design stage. To demonstrate the feasibility of the works, a proposed basement construction method statement is appended.

Noise and Nuisance Control

The contractor is to follow the good working practices and guidance laid down in the 'Considerate Constructors Scheme'. This scheme commits construction sites to commit to care about appearance, respect the community, protect the environment and secure everyone's safety. The scheme will reinforce the measures described below

Figure 13: Examples of sites registered with the Considerate Constructors Scheme

Considerations that the contractor and the design team should account for in the construction management plan are described below.

<u>Noise Control</u>

- The hours of working will be limited to those allowed: 8am to 5pm Monday to Friday and Saturday, 8am to 1pm. The hours of working will further be defined within the Party Wall Act and the requirements of Camden Council.

- The site will be hoarded with 8' site hoarding to prevent access.

- Working in the basement generally requires hand tools to be used. The level of noise generally will be no greater than that of digging of soil. The noise is reduced and muffled by the works being undertaken underground. The level of noise from basement construction works is lower than typical ground level construction due to this.

- None of the construction practices cause undue noise greater than what is expected on a typical construction site (a conveyor belt typically runs at around 70dB). Site hoarding acts as a partial acoustic screen and will reduce the level of direct noise from the site.

<u>Dust and Vibration Control</u>

- Reduce the need to use vibrating and percussive machinery.
- Use well-maintained and modern machinery
- Plant/vehicles should be cleaned before exiting the site.
- Water should be applied to suppress dust
- Skips and storage of fine materials should be covered

Appendix A: Structural Calculations

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

The design must resist:

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

The final proposed scheme must:

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

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

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

2. Party Wall foundation and retaining wall

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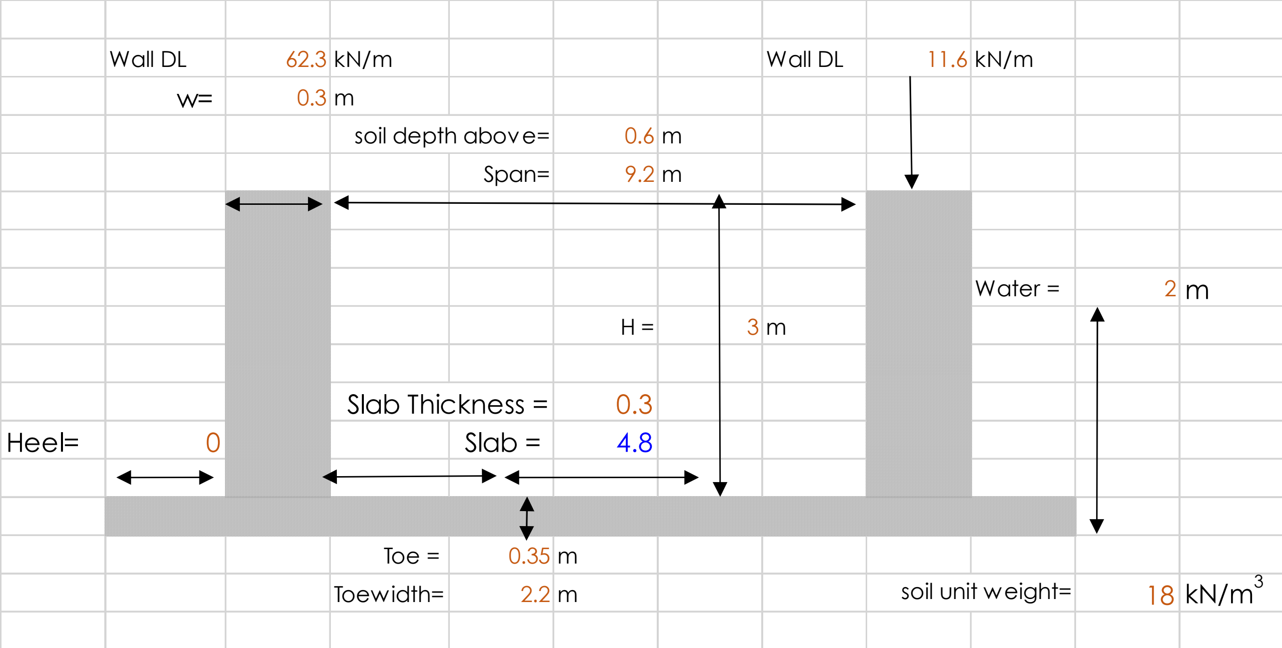

Uplift Calc							
Total Dead Load =	Slab=	36	kN/m				
	Toe and heel =	43.75	kN/m				
	Wall =	45					
	Soil=(	0	+	0) x 2 +	99.36	=	99.36 14
	Total Dead load =	298.01	kN/m				
Total Uplift Force=		196	kN/m	f.o.s.=	**1.52**	**No Global Uplift**	
Slab Uplift							
	Slab =	7.5	kN/m	Uplift =	20		
	Service Moment =	-132.25	kNm/m				
	Factored Design moment=	-161.345	kNm/m				
	Factored Design shear =	-70.15	kN/m				
Global Heave							
	Weight of building =	298.01	kN/m				
	Weight of soil removed =	529.2					
	% change	44%		place	44%	of Slab area as heave protection	
	width of heave protection =	4.281296	m	place	4.28	m of Slab area as heave protection	

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RETAINING WALL RC2 PRELIM DESIGN

RC2										
Pitched roof	1	1	1	g_k		0.93	0.9			
				q_k		0.60			0.6	
floor	6	1	6	g_k	3	0.63	11.3			
				q_k		1.50			27.0	
solid wall wall	10	1	10	g_k		5.00	50.0			
							62.3	kN/m	27.6	kN/m

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.6.11

Retaining wall details

Stem type Cantilever
Stem height h_{stem} = **2900** mm
Stem thickness t_{stem} = **300** mm
Angle to rear face of stem α = **90** deg
Stem density γ_{stem} = **25** kN/m^3
Toe length l_{toe} = **2500** mm
Base thickness t_{base} = **300** mm
Base density γ_{base} = **25** kN/m^3
Height of retained soil h_{ret} = **2900** mm Angle of soil surface β = **0** deg
Depth of cover d_{cover} = **0** mm
Height of water h_{water} = **2000** mm
Water density γ_w = **9.8** kN/m^3

Retained soil properties

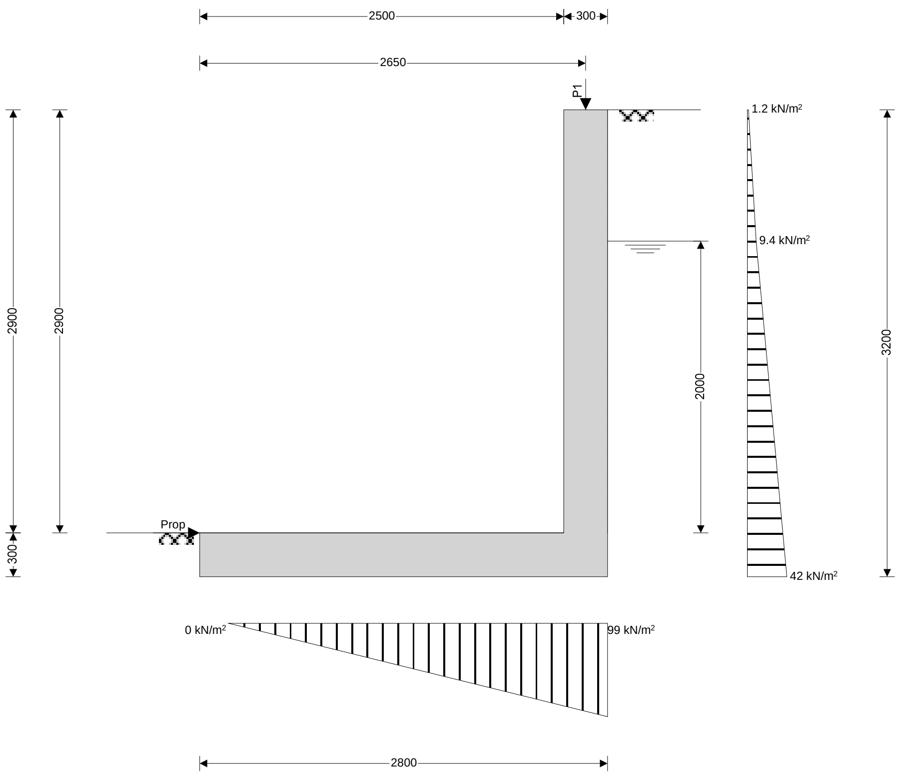
Soil type Stiff clay
Moist density γ_{mr} = **19** kN/m^3
Saturated density γ_{sr} = **19** kN/m^3
Characteristic effective shear resistance angle $\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle $\delta_{r.k}$ = **9** deg

Base soil properties

Soil type Stiff clay
Soil density γ_b = **19** kN/m^3
Characteristic effective shear resistance angle $\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle $\delta_{b.k}$ = **9** deg
Characteristic base friction angle $\delta_{bb.k}$ = **12** deg
Presumed bearing capacity $P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **2.5** kN/m^2
Vertical line load at 2650 mm	P_{G1} = **60** kN/m
	P_{Q1} = **26** kN/m



General arrangement

Calculate retaining wall geometry

Base length	l_{base} = **2800** mm		
Saturated soil height	h_{sat} = **2000** mm		
Moist soil height	h_{moist} = **900** mm		
Length of surcharge load	l_{sur} = **0** mm		
Vertical distance	x_{sur_v} = **2800** mm		
Effective height of wall	h_{eff} = **3200** mm		
Horizontal distance	x_{sur_h} = **1600** mm		
Area of wall stem	A_{stem} = **0.87** m^2	Vertical distance	x_{stem} = **2650** mm
Area of wall base	A_{base} = **0.84** m^2	Vertical distance	x_{base} = **1400** mm

Using Coulomb theory

Active pressure coefficient	K_A = **0.483**	Passive pressure coefficient	K_P = **2.359**

Bearing pressure check

Vertical forces on wall

Total $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} + F_{P\_v}$ = **128.8** kN/m

Horizontal forces on wall

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **61.8** kN/m

Moments on wall

Total $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P$ = **248.9** kNm/m

CROFT STRUCTURAL ENGINEERS

Check bearing pressure

Propping force	F_{prop_base} = **61.8** kN/m		
Bearing pressure at toe	q_{toe} = **0** kN/m^2	Bearing pressure at heel	q_{heel} = **99** kN/m^2
Factor of safety	FoS_{bp} = **1.01**		

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

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

Tedds calculation version 2.6.11

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

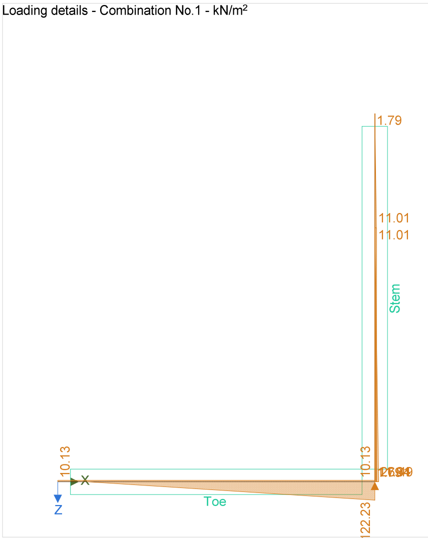
Concrete strength class	C30/37		
Char.comp.cylinder strength	f_{ck} = **30** N/mm^2	Mean axial tensile strength	f_{ctm} = **2.9** N/mm^2
Secant modulus of elasticity	E_{cm} = **32837** N/mm^2	Maximum aggregate size	h_{agg} = **20** mm
Design comp.concrete strength	f_{cd} = **17.0** N/mm^2	Partial factor	γ_C = **1.50**

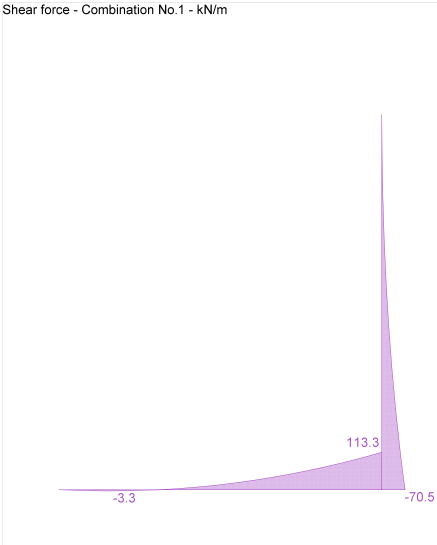
Reinforcement details

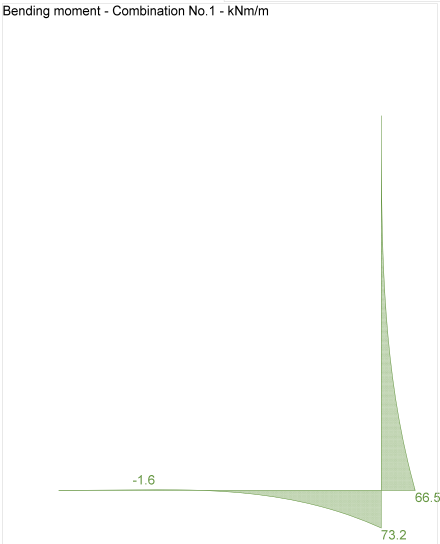
Characteristic yield strength	f_{yk} = **500** N/mm^2	Modulus of elasticity	E_s = **200000** N/mm^2
Design yield strength	f_{yd} = **435** N/mm^2	Partial factor	γ_S = **1.15**

Cover to reinforcement

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







Check stem design at base of stem

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **66.5** kNm/m K = **0.038** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{sr.req}$ = **671** mm^2/m

Tens.reinforcement provided 20 dia.bars @ 200 c/c Tens.reinforcement provided $A_{sr.prov}$ = **1571** mm^2/m

Min.area of reinforcement $A_{sr.min}$ = **361** mm^2/m Max.area of reinforcement $A_{sr.max}$ = **12000** mm^2/m

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

Deflection control - Section 7.4

Limiting span to depth ratio 26.1 Actual span to depth ratio 12.1

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.143** mm

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

Design shear force V = **70.5** kN/m Design shear resistance $V_{Rd.c}$ = **148.6** kN/m

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Min.area of reinforcement $A_{sx.req}$ = **393** mm^2/m Max.spacing of reinforcement s_{sx_max} = **400** mm

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

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

Check base design at toe

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **73.2** kNm/m K = **0.053** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{bb.req}$ = **824** mm^2/m

Tens.reinforcement provided 20 dia.bars @ 200 c/c Tens.reinforcement provided $A_{bb.prov}$ = **1571** mm^2/m

Min.area of reinforcement $A_{bb.min}$ = **324** mm^2/m Max.area of reinforcement $A_{bb.max}$ = **12000** mm^2/m

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.227** mm

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

Design shear force V = **113.3** kN/m Design shear resistance $V_{Rd.c}$ = **141.8** kN/m

PASS - Design shear resistance exceeds design shear force

Check base design at toe

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **1.6** kNm/m K = **0.001** K' = **0.207**

K' > K - No compression reinforcement is required

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

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

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

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

Crack control - Section 7.3

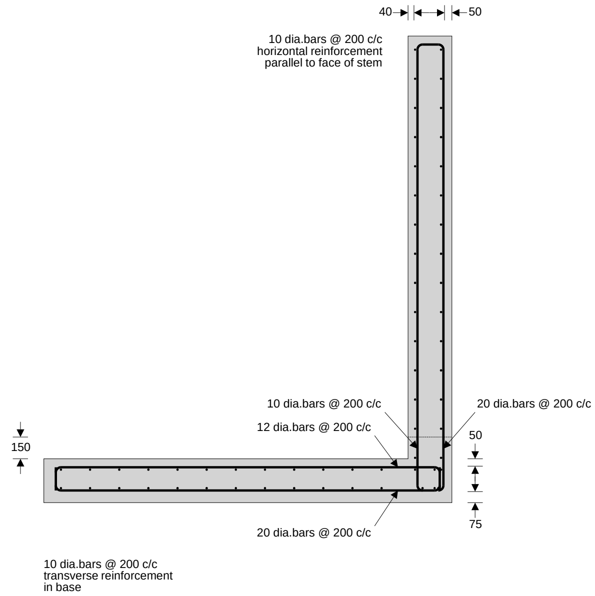
Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0** mm

PASS - Maximum crack width is less than limiting crack width**Secondary transverse reinforcement to base - Section 9.3**

Min.area of reinforcement $A_{bx.req}$ = **314** mm^2/m Max.spacing of reinforcement s_{bx_max} = **450** mm

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

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



Reinforcement details

RETAINING WALL RC2 PRELIM DESIGN

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.6.11

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **2600** mm
Stem thickness	t_{stem} = **300** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2000** mm
Base thickness	t_{base} = **300** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **2600** mm

Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **1800** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

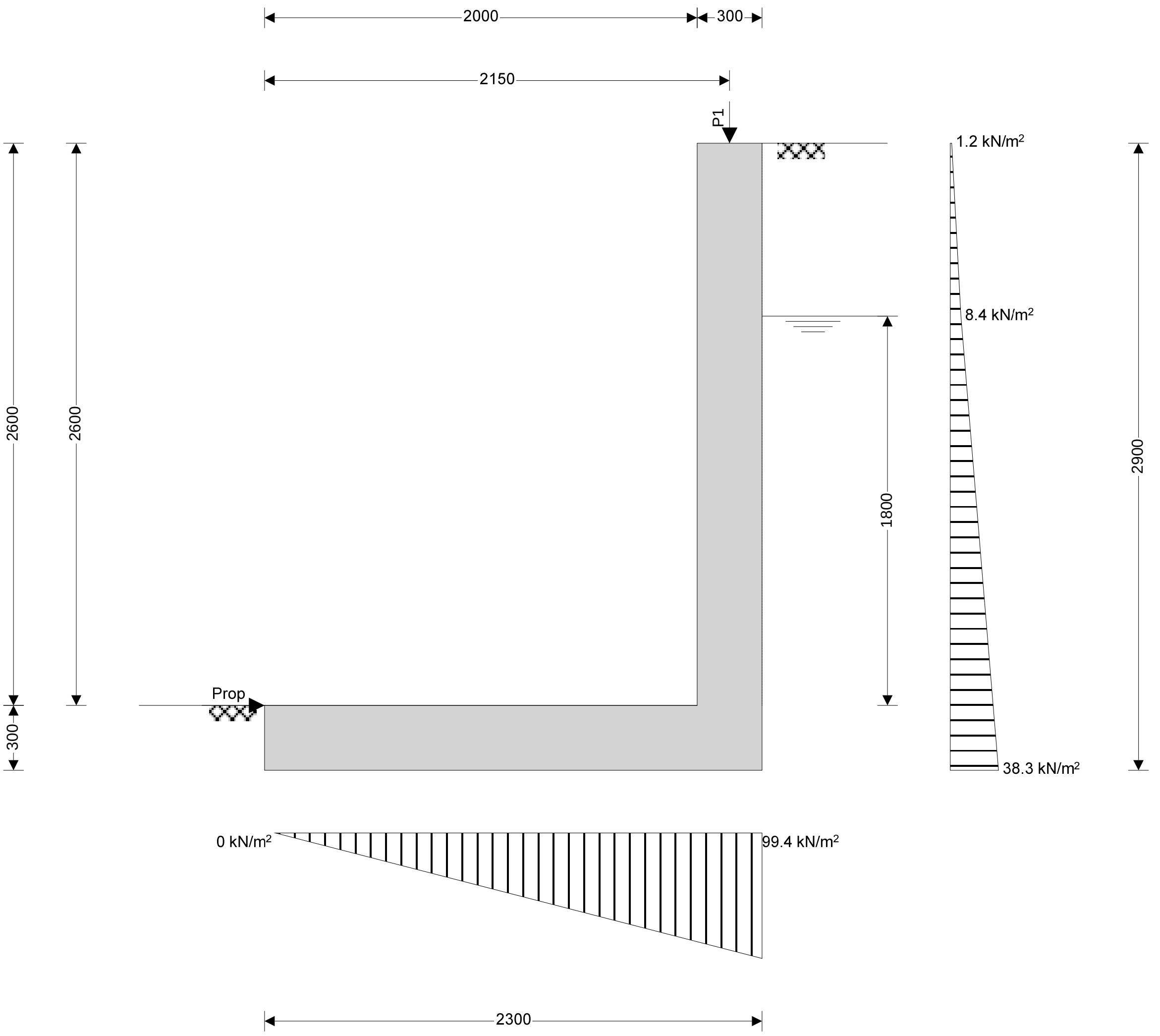
Soil type	Stiff clay
Moist density	γ_{mr} = **19** kN/m^3
Saturated density	γ_{sr} = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type	Stiff clay
Soil density	γ_b = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **2.5** kN/m^2
Vertical line load at 2150 mm	P_{G1} = **55** kN/m
	P_{Q1} = **20** kN/m



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} =$ **2300** mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} =$ **1800** mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} =$ **800** mm
Length of surcharge load	$l_{sur} = l_{heel} =$ **0** mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **2300** mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **2900** mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 =$ **1450** mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} =$ **0.78** m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 =$ **2150** mm
Area of wall base	$A_{base} = l_{base} \times t_{base} =$ **0.69** m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 =$ **1150** mm

Using Coulomb theory

Active pressure coefficient

$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[}\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]]^2) =$ **0.483**

Passive pressure coefficient | $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]]^2}) =$ **2.359**

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **19.5** kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **17.3** kN/m
Line loads	$F_{P\_v} = P_{G1} + P_{Q1} =$ **75** kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} + F_{P\_v} =$ **111.8** kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff} =$ **3.5** kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 =$ **9.7** kN/m
Water	$F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **21.6** kN/m
Moist retained soil	$F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **18.1** kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 =$ **-2** kN/m
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} =$ **50.9** kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} =$ **41.9** kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} =$ **19.8** kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h} =$ **-5** kNm/m
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1 =$ **161.3** kNm/m
Saturated retained soil	$M_{sat} = -F_{sat\_h} \times x_{sat\_h} =$ **-6.8** kNm/m
Water	$M_{water} = -F_{water\_h} \times x_{water\_h} =$ **-15.1** kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **-22.9** kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P =$ **173.2** kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h} =$ **50.9** kN/m
Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} =$ **1550** mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 =$ **400** mm
Loaded length of base	$l_{load} = 3 \times (l_{base} - \bar{x}) =$ **2249** mm
Bearing pressure at toe	$q_{toe} =$ **0** kN/m^2
Bearing pressure at heel	$q_{heel} = 2 \times F_{total\_v} / l_{load} =$ **99.4** kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **1.006**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

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

Tedds calculation version 2.6.11

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

Concrete strength class	C30/37
Characteristic compressive cylinder strength	$f_{ck} =$ **30** N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} =$ **37** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **38** N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **2.9** N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} =$ **2.0** N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **32837** N/mm^2

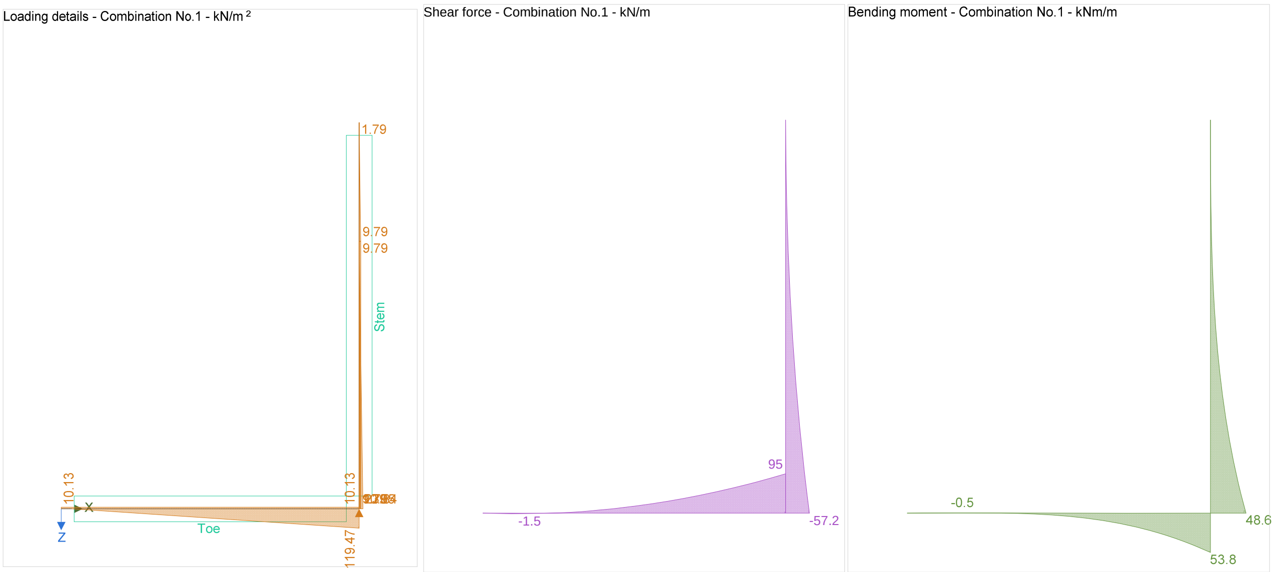
Partial factor for concrete - Table 2.1N	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1)	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **17.0** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm

Reinforcement details

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

Cover to reinforcement

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



Check stem design at base of stem

Depth of section	h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **48.6** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2$ = **240** mm
	$K = M / (d^2 \times f_{ck})$ = **0.028**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **228** mm
Depth of neutral axis	$x = 2.5 \times (d – z)$ = **30** mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z)$ = **491** mm^2/m
Tension reinforcement provided	20 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **1571** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **361** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h$ = **12000** mm^2/m
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.312**

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

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 =$ **0.005**
Required tension reinforcement ratio	$\rho = A_{sr.req} / d =$ **0.002**
Required compression reinforcement ratio	$\rho' = A_{sr.2.req} / d_2 =$ **0.000**
Structural system factor - Table 7.4N	$K_b =$ **0.4**
Reinforcement factor - exp.7.17	$K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) =$ **1.5**
Limiting span to depth ratio - exp.7.16.a	$K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}] =$ **42.7**
Actual span to depth ratio	$h_{stem} / d =$ **10.8**

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

Crack control - Section 7.3

Limiting crack width	$w_{max} =$ **0.3** mm
Variable load factor - EN1990 – Table A1.1	$\psi_2 =$ **0.6**
Serviceability bending moment	$M_{sls} =$ **34** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z) =$ **94.8** N/mm^2
Load duration	Long term
Load duration factor	$k_t =$ **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2) =$ **90000** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} =$ **2.9** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff} =$ **0.017**
Modular ratio	$\alpha_e = E_s / E_{cm} =$ **6.091**
Bond property coefficient	$k_1 =$ **0.8**
Strain distribution coefficient	$k_2 =$ **0.5**
	$k_3 =$ **3.4**
	$k_4 =$ **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} =$ **365** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k =$ **0.104** mm
	$w_k / w_{max} =$ **0.346**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V =$ **57.2** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**
	$k = \min(1 + \sqrt{(200\ mm / d)}, 2) =$ **1.913**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02) =$ **0.007**
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} =$ **0.507** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} =$ **148.6** kN/m
	$V / V_{Rd.c} =$ **0.385**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) =$ **393** mm^2/m
Maximum spacing of reinforcement – cl.9.6.3(2)	$s_{sx\_max} =$ **400** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) =$ **393** mm^2/m

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

Check base design at toe

Depth of section — $h = \mathbf{300}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{53.8}$ kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{215}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.039}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{204}$ mm

Depth of neutral axis — $x = 2.5 \times (d – z) = \mathbf{27}$ mm

Area of tension reinforcement required — $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{606}$ mm^2/m

Tension reinforcement provided — 20 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{1571}$ mm^2/m

Minimum area of reinforcement - exp.9.1N — $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{324}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max} = 0.04 \times h = \mathbf{12000}$ mm^2/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = \mathbf{0.386}$

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

Crack control - Section 7.3

Limiting crack width — $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1 — $\psi_2 = \mathbf{0.6}$

Serviceability bending moment — $M_{sls} = \mathbf{39.5}$ kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = \mathbf{123.2}$ N/mm^2

Load duration — Long term

Load duration factor — $k_t = \mathbf{0.4}$

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2) = \mathbf{91042}$ mm^2/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} = \mathbf{2.9}$ N/mm^2

Reinforcement ratio — $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.017}$

Modular ratio — $\alpha_e = E_s / E_{cm} = \mathbf{6.091}$

Bond property coefficient — $k_1 = \mathbf{0.8}$

Strain distribution coefficient — $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{452}$ mm

Maximum crack width - exp.7.8 — $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0.167}$ mm

$w_k / w_{max} = \mathbf{0.557}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — $V = \mathbf{95}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.964}$

Longitudinal reinforcement ratio — $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.007}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.528}$ N/mm^2

Design shear resistance - exp.6.2a & 6.2b — $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{141.8}$ kN/m

$V / V_{Rd.c} = \mathbf{0.670}$

PASS - Design shear resistance exceeds design shear force

Check base design at toe

Depth of section	h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **0.5** kNm/m
Depth to tension reinforcement	$d = h - c_{bt} - \phi_{bt} / 2$ = **244** mm
	$K = M / (d^2 \times f_{ck})$ = **0.000**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **232** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **31** mm
Area of tension reinforcement required	$A_{bt.req} = M / (f_{yd} \times z)$ = **5** mm^2/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **565** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **368** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bt.max} = 0.04 \times h$ = **12000** mm^2/m
	$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov}$ = **0.65**

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

Crack control - Section 7.3

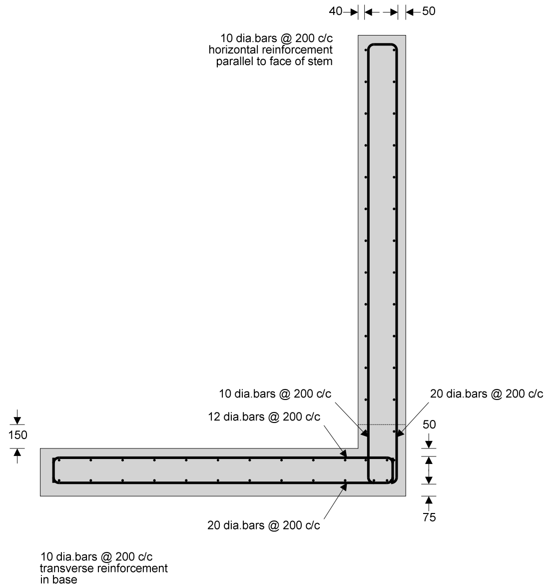
Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **0** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bt.prov} \times z)$ = **0** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **89833** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{bt.prov} / A_{c.eff}$ = **0.006**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.091**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff}$ = **494** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0** mm
	w_k / w_{max} = **0**

PASS - Maximum crack width is less than limiting crack width

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **314** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	s_{bx_max} = **450** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **393** mm^2/m

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



Reinforcement details

Appendix B: Construction Programme

<u>The Contractor is responsible for the final construction programme</u>

Outline construction Program

(For planning purposes only)

	Months															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Planning approval																
Derailed Design																
Tender																
Party Walls																
Monitoring of Adjacent structures																
Enabling works																
Basement Construction																
Superstructure construction																

Appendix C: Construction Sequence and Plans

1. Preamble

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

1.2. This method statement has been written by a Chartered Engineer. The sequencing has been developed using guidance from ASUC (Association of Specialist Underpinning Contractors). Croft Structural Engineers are an Associate Member of ASUC.

1.3. This method has been produced to allow for improved costings and for inclusion in the Party Wall Award. Final site conditions need there to be flexibility in the method statement: Should the site staff require alterations to the Method statement this is allowed once an alternative methodology, of the changes is provided, and an Addendum to the Party Wall Award will be required.

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

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

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1.6. Temporary props will be provided along the height of the pin in the temporary condition. Before the base is cast cross props are needed. The base/ground slab provides propping in the final condition. In the temporary condition, the edge of the slab is buttressed against the soil in the middle of the property. Also the skin friction between the concrete base and the soil provides further resistance. The central soil mass is to be removed in portions (thirds but no greater than 8m) and cross propping subsequently added as the central soil ass is removed

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1.7. A ground investigation has been undertaken. The soil present is made ground to a depth of 0.65-1m below ground level. The made ground generally comprised brown gravelly sandy clay. Sand was fine to coarse grained. Gravel was occasional, sub-angular, fine flint and brick fragments.Soils of the London Clay Formation were encountered underlying the Made Ground. The soils generally comprised orangish brown silty clay, becoming a dark grey brown silty clay at 8.00m bgl.

1.8. The bearing pressures have been limited to 125kN/m^2 as advised in the ground investigation report.

1.9. The water table is expected to encountered below the formation level at 6.3m BGL however, perched water was recorded during the return site visits.

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

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1.11. Provide engineers with concrete mix, supplier, delivery and placement methods two weeks prior to the first pour. Site mixing of concrete should not be employed apart from in small sections (less than 1m3). The contractor must provide a method on how to achieve site mixing to the correct specification. The contractor must undertake toolbox talks with staff to ensure site quality is maintained.

2. Enabling Works

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

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

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

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2.4. Dewater: Water is expected at 6.3m depths

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2.4.1. No significant dewatering is expected. Localised removal of water may be required to deal with rain from perched water or localised water. This is to be dealt with by localised pumping. Typically achieved by a small sump pump in a bucket.

2.5. 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

Front part of basement / Lower existing ground floor level

3.1. Begin by placing strip footings 1/2 noted on plans to the front part of the property. The existing lower ground floor will be lowered by approximately 300mm below the existing ground floor level.

3.2. The section pins to form the lowered area to the front of the basement will be limited to 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 wall has a lower risk of collapse to the heel face.

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3.3. Continue excavating section pins to form front part of lowered ground floor level following the numbering sequence on the drawings. (Follow methodology in Section 3.2 above)

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3.4. Excavation of adjacent pin to not commence until 48 hours after drypacking. (24hours possible due to inclusion of Conbextra 100 cement accelerator to dry pack mix). No more than

Rear part of basement

3.5. Begin by placing cantilevered wall 20 noted on plans. (Cantilevered walls to be placed in accordance with Section 4.)

3.6. Needle and prop the walls over.

3.7. Insert steel over and sit on cantilevered walls.

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

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

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

3.8.1. Excavation for the next numbered sequential sections of underpinning shall not commence until at least 8 hours after drypacking of previous works. Excavation of

adjacent pin to not commence until 48 hours after drypacking. (24hours possible due to inclusion of Conbextra 100 cement accelerator to dry pack mix). No more than

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

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

3.10. Excavate and cast floor slab

3.10.1. Excavate 1/3 of the middle section of basement floor. As excavation proceeds, place Slim Shore props at a maximum of 2.5m c/c across the basement. Locate props at a third of the height of the wall.

3.10.2. Continue excavating the next 1/3 and prop then repeat for the final 1/3.

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

3.10.4. Place reinforcement for basement slab.

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

3.10.6. Once inspected, pour concrete.

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

4. Underpinning and Cantilevered Walls

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

4.2. Provide propping to floor where necessary.

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

4.4. Excavation of pins involves working in confined spaces and the following measures should be applied:

- Operatives must wear a harness and there must be a winch above the excavation.
- An attendant must be present at all times, at ground level, while excavation is occupied.
- A rescue plan must be produced prior to the works as well as a task-specific risk and method statement.
- Working in the confined space should require a permit to work.

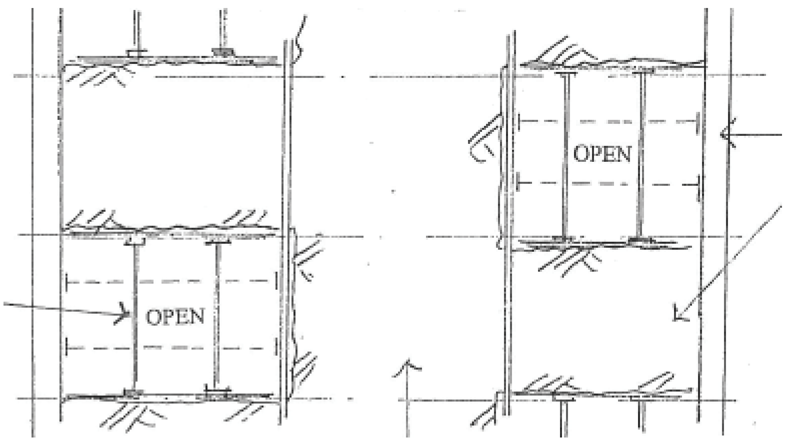

Figure 14 – Schematic Plan view of soil propping


Figure 15 Propping examples

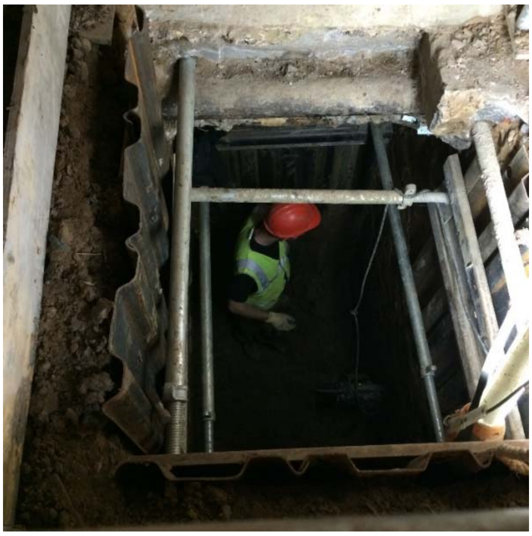


Figure 16 Examples of excavations of pins

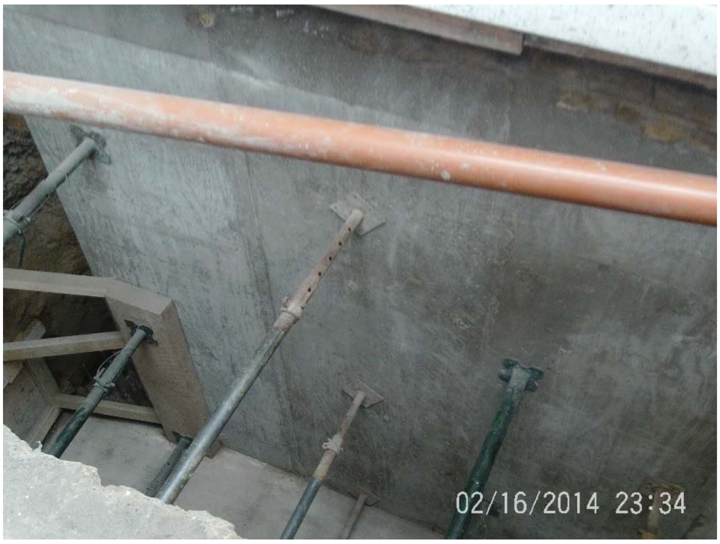

Figure 17 Examples of completed walls and back propping to central soil mass

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

Figure 18 Example of trench sheet back propping

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

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

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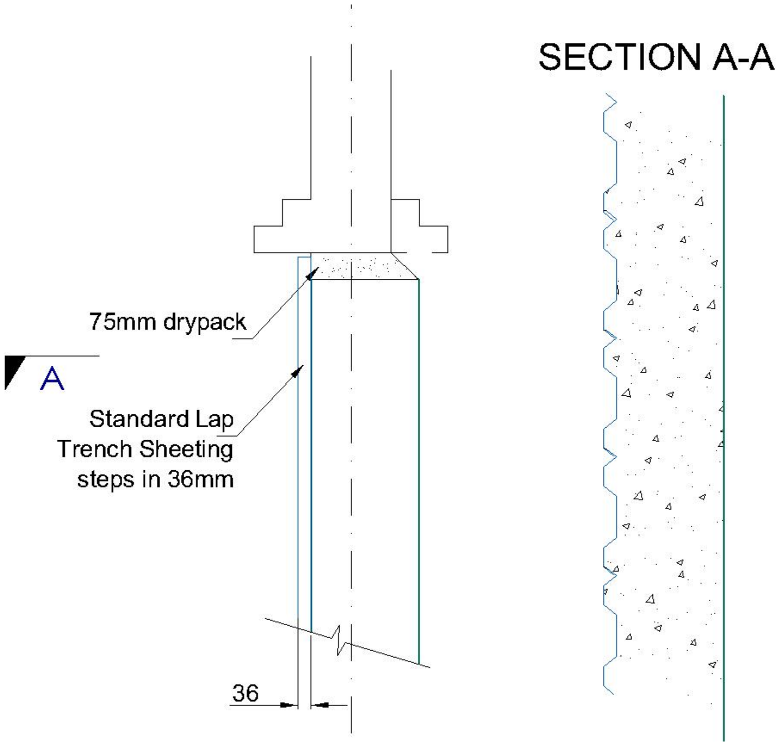

4.5.3. Where soft spots are encountered, leave in trench sheets or alternatively back prop with precast lintels or sacrificial boards. If the soil support to the ends of the lintels is insufficient, then brace the ends of the PC lintels with 150x150 C24 timbers and prop with Acrows diagonally back to the ground.

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

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

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

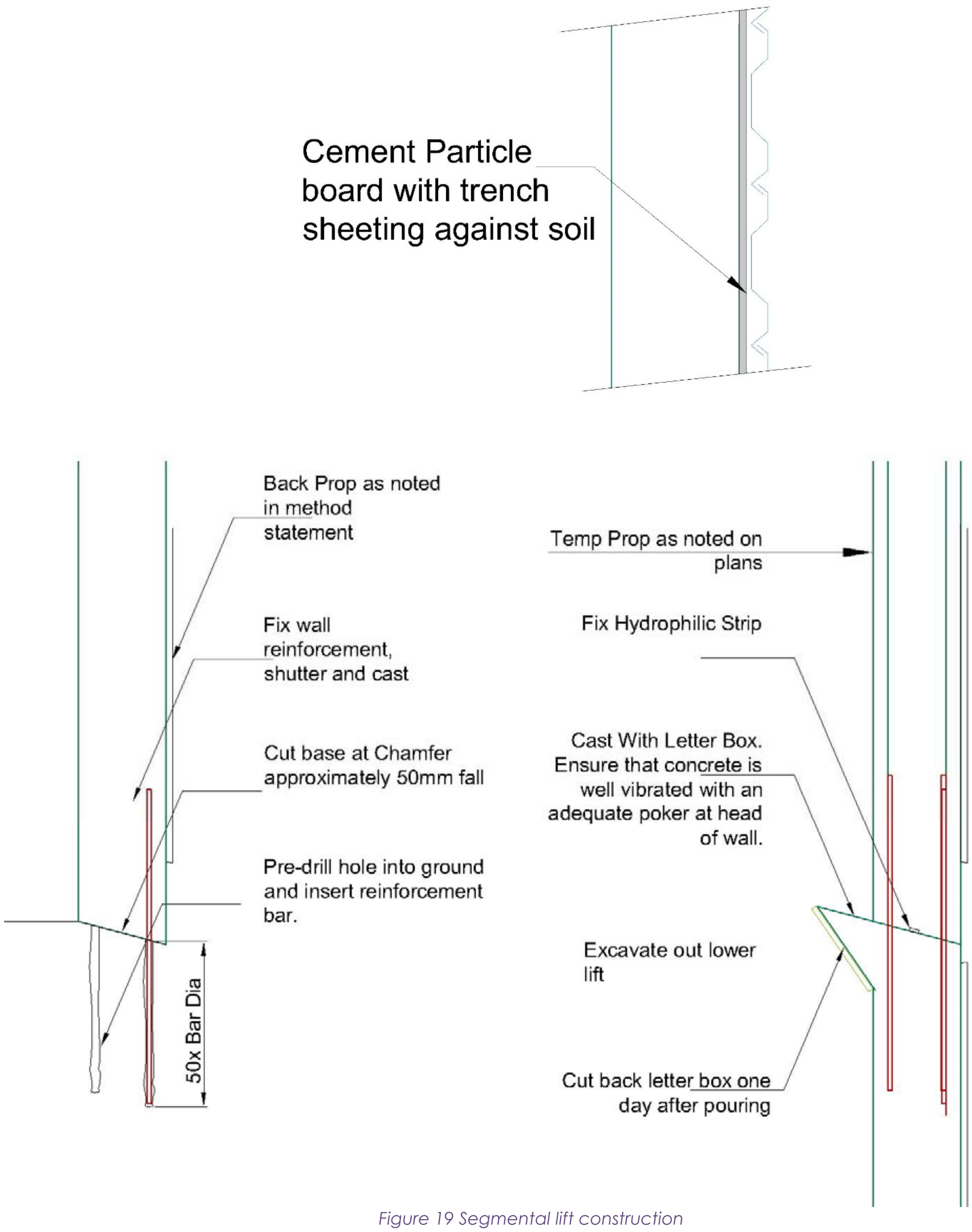

Figure 19 Segmental lift construction

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

-

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

4.9. Clear underside of existing footing.

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

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

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

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

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

4.14. Concrete Testing:

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

4.14.2. If results are good from first three pins, then from the 4^{th} pin onwards take 2 cubes of concrete from every third pin and store for testing. Test one at 28 days. If result is low, test second cube. Provide results to client and design team on request or if values are below those required.

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

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

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

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

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

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

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

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

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

5. Floor Support

Timber Floor

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

5.2. Position 100 x 100mm temporary timber beams, 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 beams as a replacement. Steel beams to bear onto concrete padstones built into the masonry walls (refer to Structural Engineer's details for padstone and beam sizes)

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

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 installation. Support comprises the temporary installation of steel needle beams at high level, supported on vertical props. This will enable safe removal of brickwork below and installation of the new beams and columns.

6.1.1. The condition of the brickwork 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 between windows should be given greater consideration.

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

6.2. Props are to be placed on sleepers on 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.

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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 of pins inspected and cast.

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

8. Basement Temporary Works Design Lateral Propping

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

Trench sheets should be placed at regular centres to deal with the ground. It is expected that the soil between the trench sheeting will arch. Looser soil will require tighter centres. It is typical for underpins to be placed at 1200c/c in this condition the highest load on a trench sheet is when 2 No.s trench sheets are used. It is for this design that these calculations have been provided.

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

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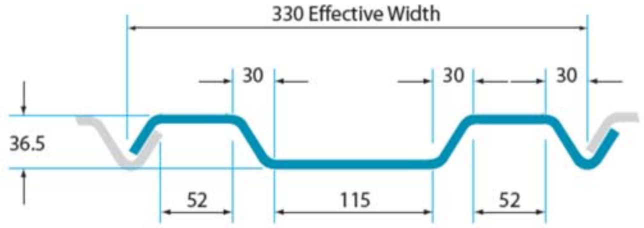
Trench Sheet Design

Soil Depth	Dsoil = 3000mm	
Surcharge	sur = $10kN/m^2$	
Soil Density	γ = $20kN/m^3$	
Angle of Friction	ϕ = 25°	
	$k_a = (1 - \sin(\phi)) / (1 + \sin(\phi))$	= **0.406**
	$k_p = 1 / k_a$	= **2.464**
Soil pressure bottom	soil = $k_a * \gamma *$ Dsoil	= **21.916**kN/m^2
Surcharge pressure	surcharge = sur * k_a	= **4.059** kN/m^2

STANDARD LAP

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



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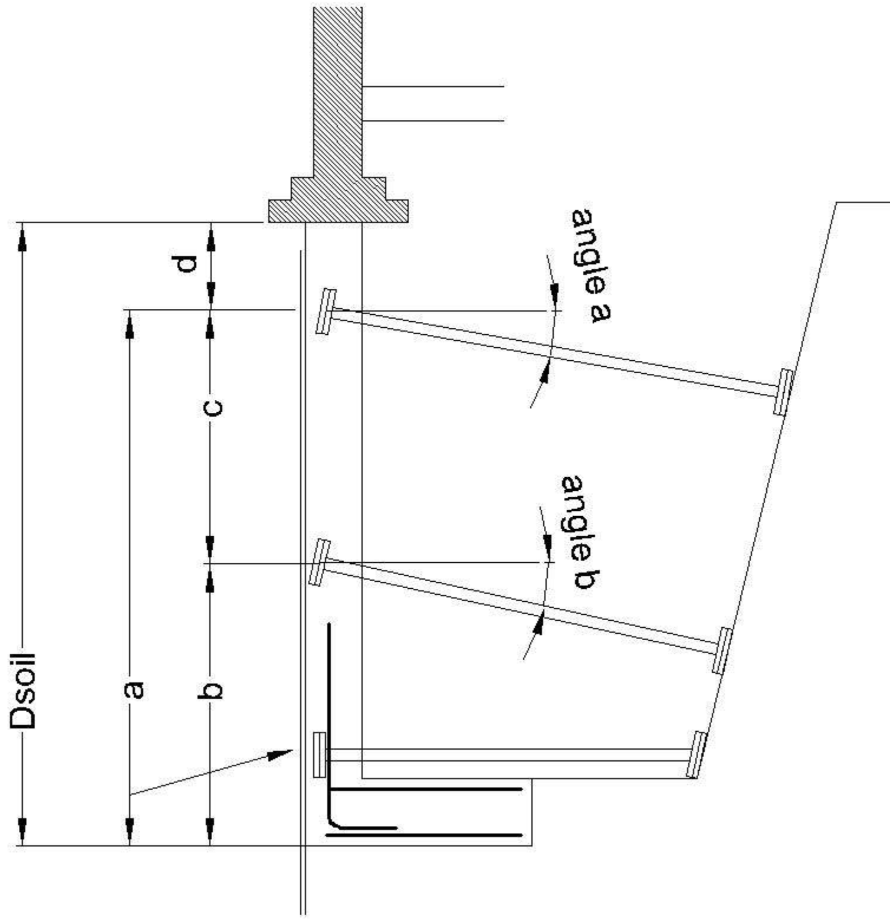
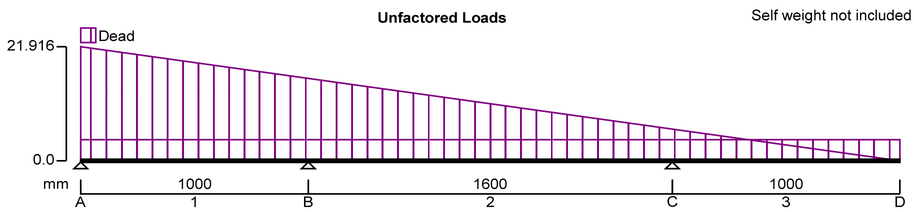
Technical Information

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

Sxx = 15.9 cm^3
py = 275N/mm^2
Ixx = 26.9cm^4
A = (1m * 32.9kg/m^2) / (7750kg/m^3) = **4245.161**mm^2





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 = **1000** mm Cross-sectional area = **4245** mm^2 Moment of inertia = **269.×10³** mm^4
Span 2 Length = **1600** mm Cross-sectional area = **4245** mm^2 Moment of inertia = **269.×10³** mm^4
Span 3 Length = **1000** mm Cross-sectional area = **4245** mm^2 Moment of inertia = **269.×10³** mm^4

LOADING DETAILS

Beam Loads:

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

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

LOAD COMBINATIONS

Load combination 1

Span 1 1.4×Dead

Span 2 1.4×Dead

Span 3 1.4×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Support Reactions - Combination Summary

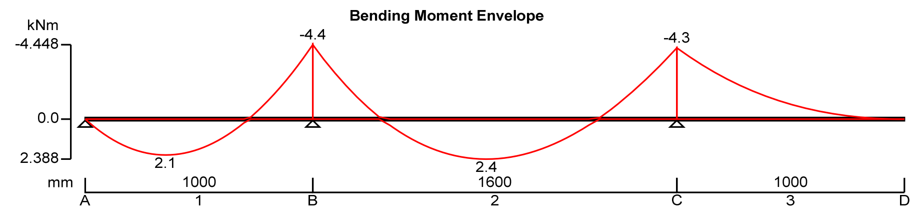
Support A	Max react = **-12.3** kN	Min react = **-12.3** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-38.5** kN	Min react = **-38.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-24.8** kN	Min react = **-24.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support D	Max react = **0.0** kN	Min react = **0.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm

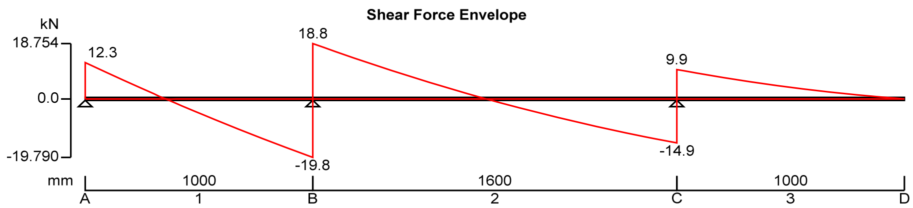
Beam Max/Min results - Combination Summary

Maximum shear = **18.8** kN Minimum shearF_{min} = **-19.8** kN

Maximum moment = **2.4** kNm Minimum moment = **-4.4** kNm

Maximum deflection = **17.1** mm Minimum deflection = **-0.1** mm





Number of sheets Nos = 3

Moment M_allowable = Sxx * py * Nos = **13.118**kNm

Deflection D = / Nos = **5.699**mm

Acro Load Acro = R_{max_B} / 2 = -19.272kN

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

Acrow Props A or B are acceptable placed 0.5m from top, middle and 1m from bottom

Cross Props

Props should be placed a third up the wall measured from the bottom slab.

Surcharge	sur = $10kN/m^2$
Soil Density	$\gamma = 20kN/m^3$

Angle of Friction $\phi = 25°$

Soil Depth Dsoil = 3000mm

$k_a = (1 - \sin(\phi)) / (1 + \sin(\phi))$ = **0.406**

$k_p = 1 / k_a$ = **2.464**

$1 - \sin(\phi)$ = **0.577**

Soil force bottomsoilforce = $k_a * \gamma *$ Dsoil * Dsoil / 2 = **36.527**kN/m

Surcharge Force Surchargeforce = k_a * sur * Dsoil = **12.176**kN/m

Place Props every other pin spacing = 2m

Propforce Propforce = spacing * (soilforce + Surchargeforce) = **97.406**kN

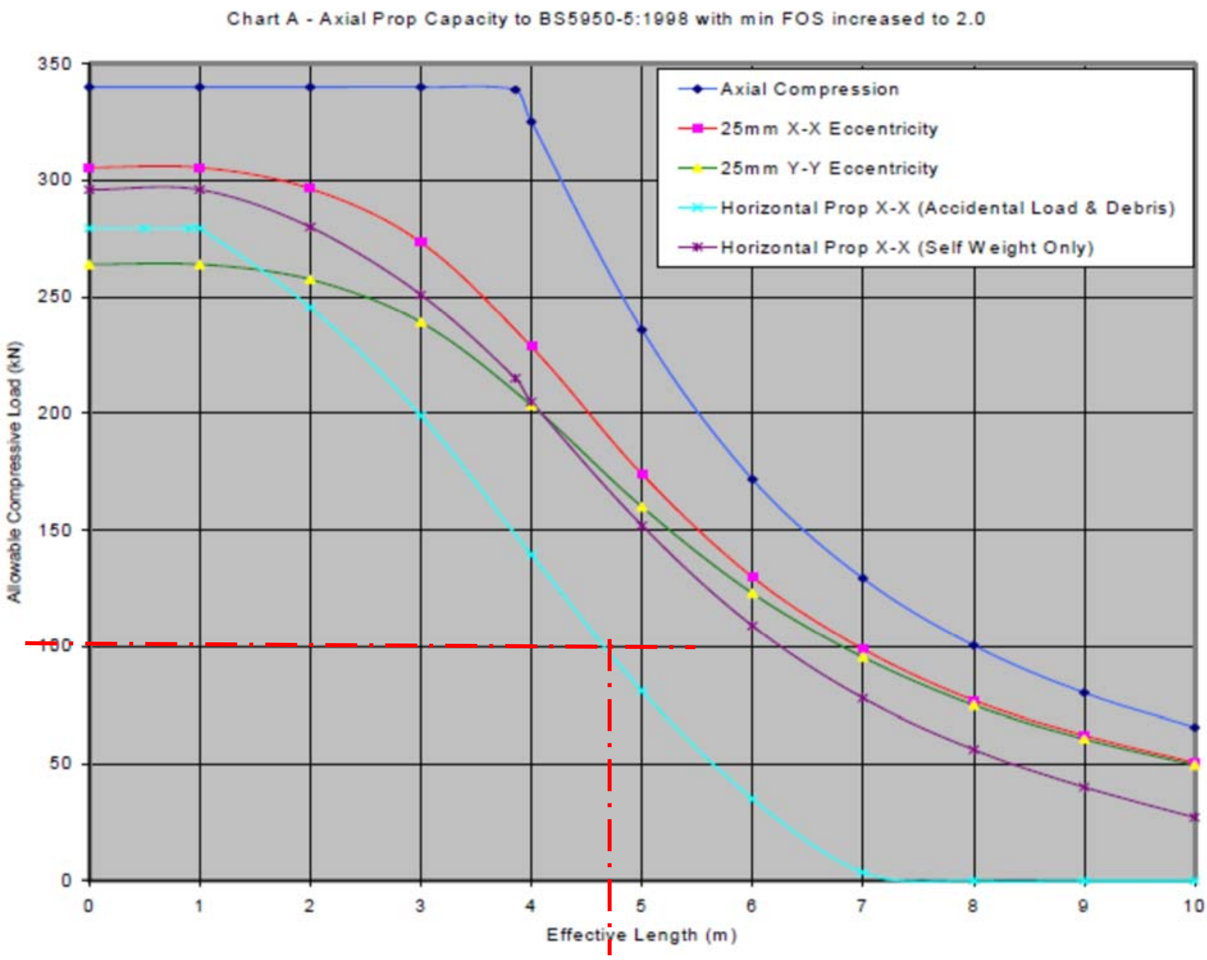


Figure 20 Mabey Mass 25 Load Chart

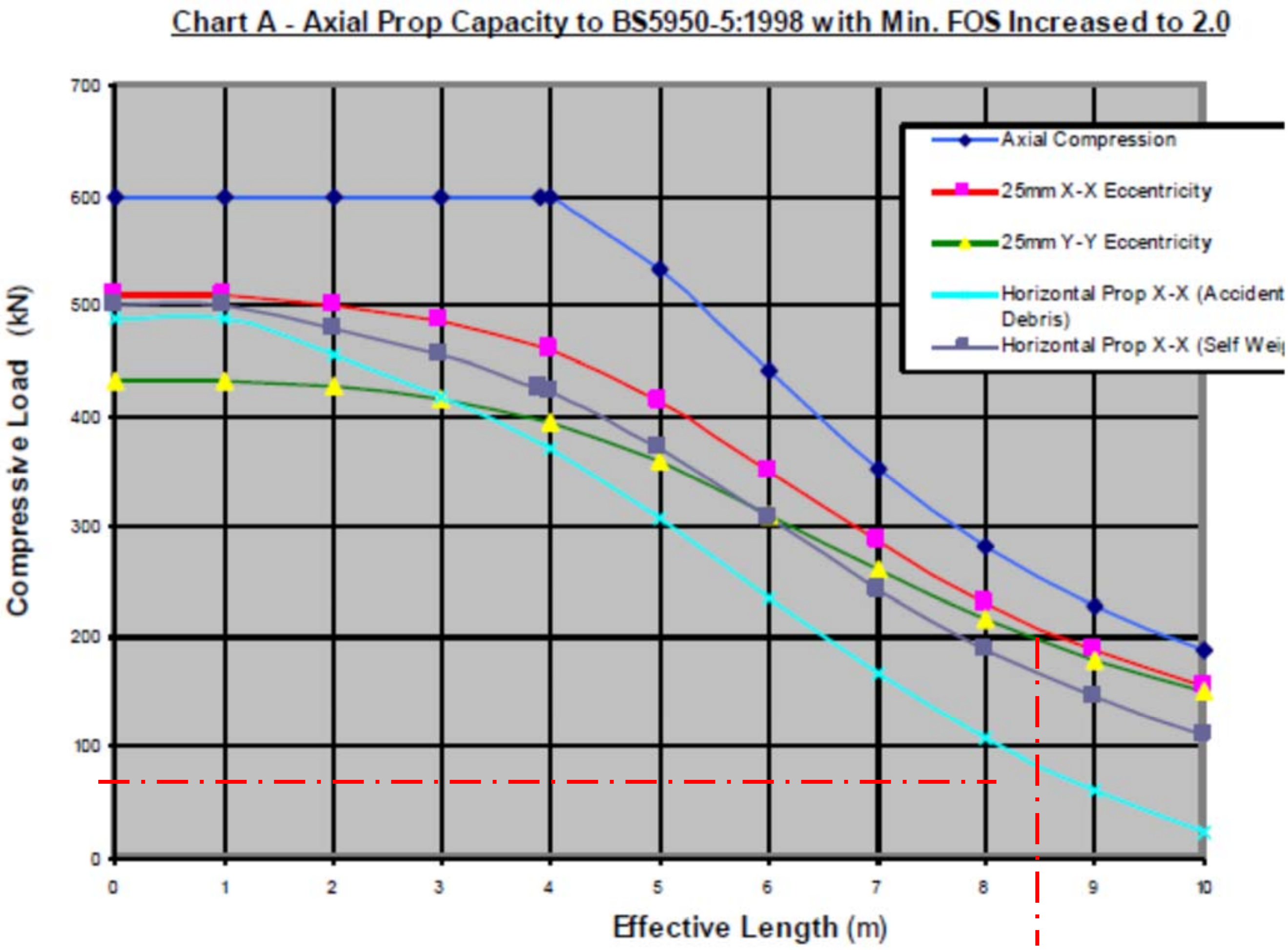
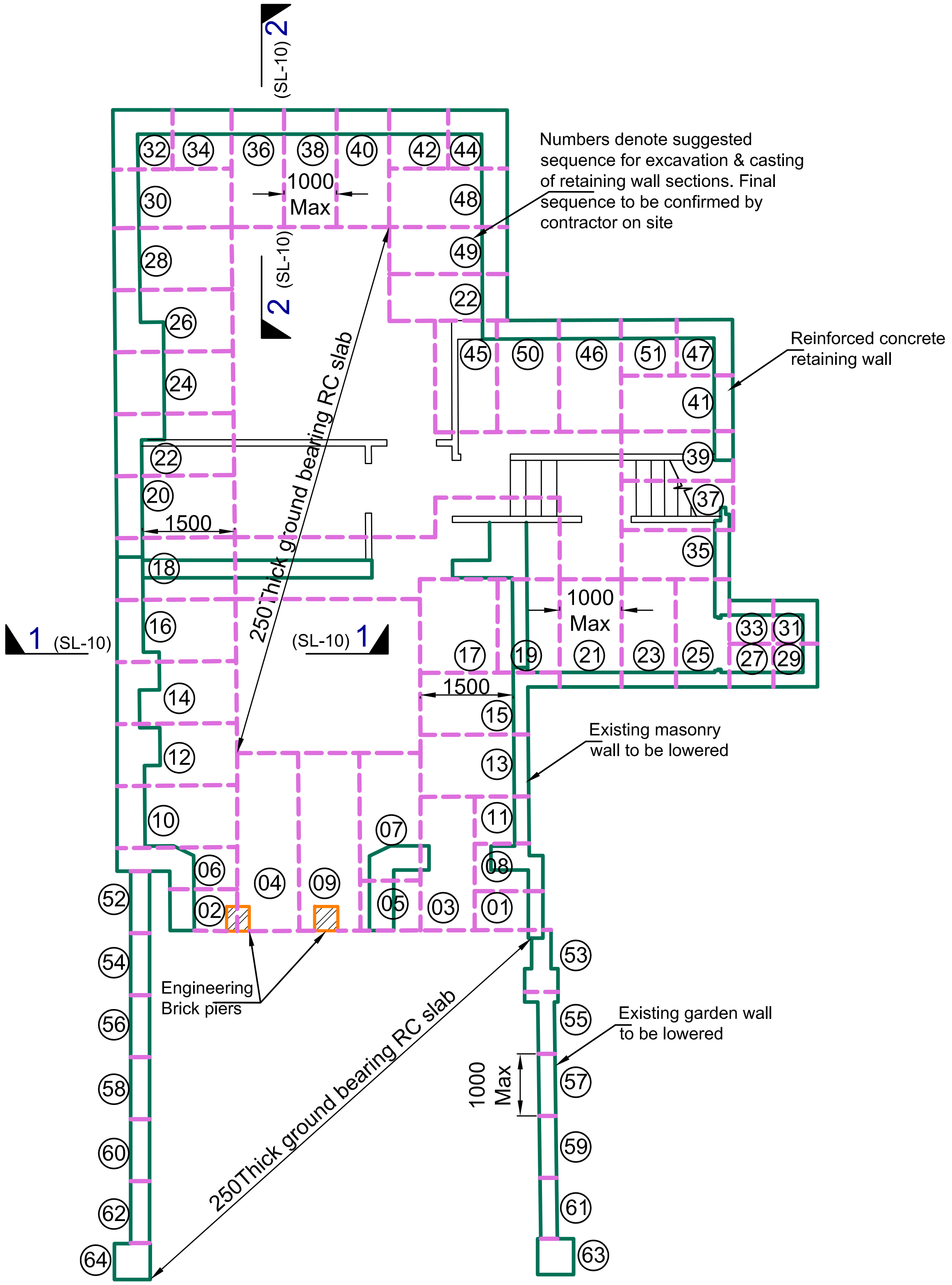


Figure 21 Mabey Mass 50 Load Chart

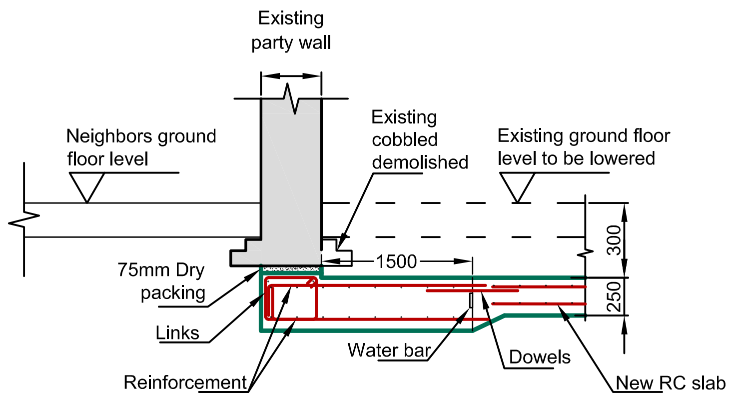
Provide Mabey Mass 50 at 2m Centres at 1/3 the height of the wall.

Appendix D: Structural Drawings



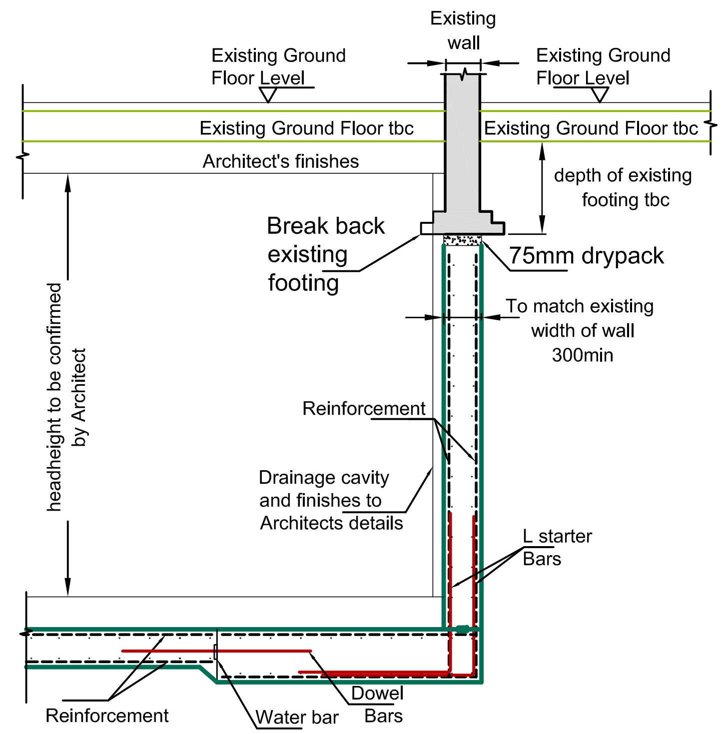
Basement Plan

Scale 1:100



Section 1-1

Scale 1:50



Section 2-2

(Scale 1:50)

Planning issue
Not for construction

Rev	Date	Amendments
1	16/10/17	minor amendments to Architects comments
-	06/10/17	First issue preliminary to Architect

Job Number 170823	Date Oct '17
Dwg Number SL-10	Rev 1
Drawn SB	Ch'kd EP
Scale As shown @ A3	

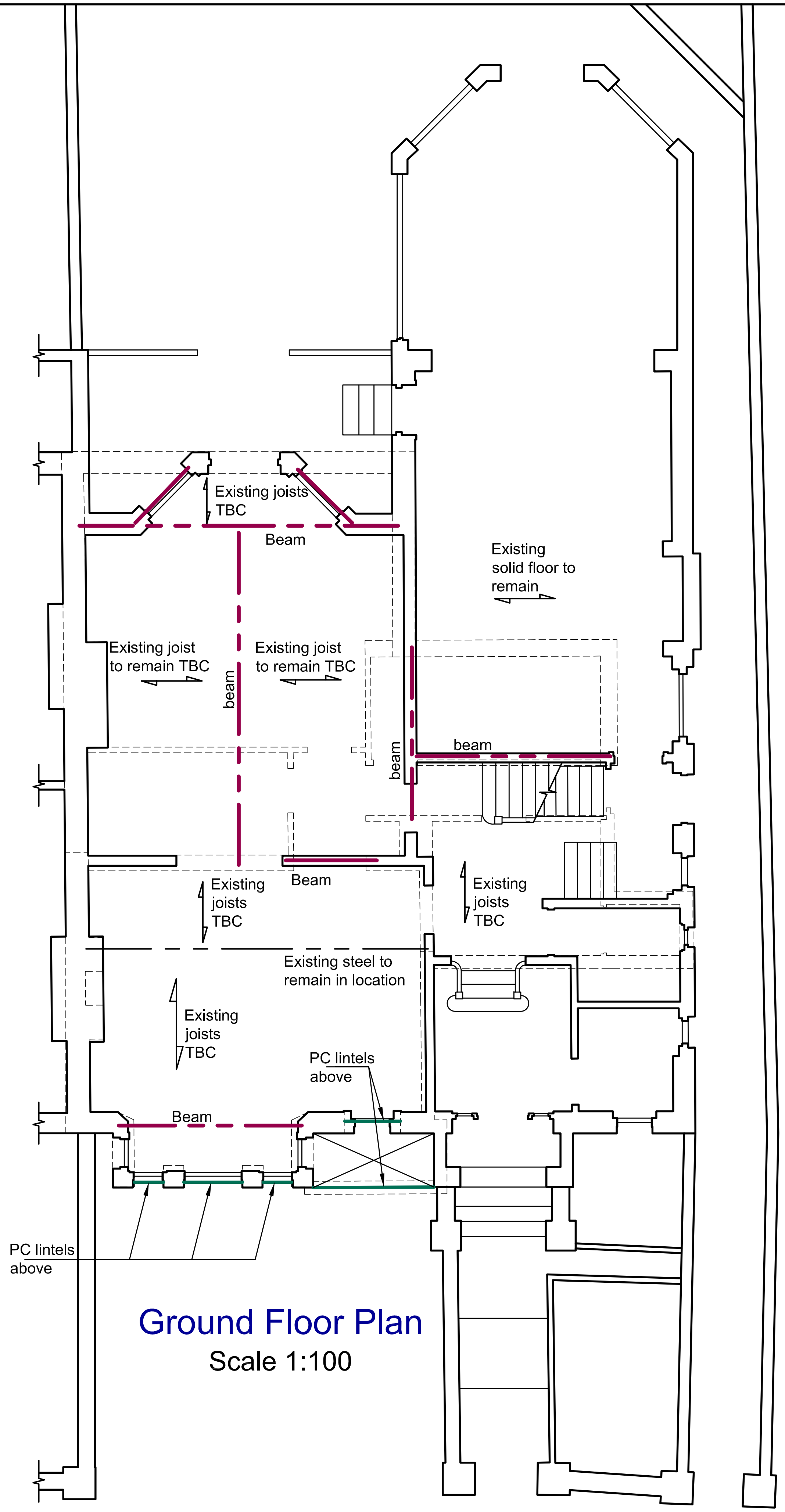
Client: Stephan Wilcke

Project: 6 Parsifal Rd

Title : Basement Plan and Section

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

Planning issue
Not for construction



Ground Floor Plan
Scale 1:100

Rev	Date	Amendments
1	16/10/17	minor amendments to Architects comments
-	06/10/17	First issue preliminary to Architect

Job Number 170823	Date Oct '17
Dwg Number SL-20	Rev 1
Drawn SB	Ch'kd EP
Scale As shown @ A3	

Client: Stephan Wilcke

Project: 6 Parsifal Rd

Title : Ground Floor Plan

Croft Structural Engineers
Clockshop Mews,
r/o 60 Saxon Rd,
London, SE25 5EH.
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CROFT STRUCTURAL ENGINEERS

Appendix E: Proposed Monitoring Statement

9. Introduction

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

10. Risk Assessment

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

Monitoring Level Proposed	Type of Works.
Monitoring 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 less than 2.50m Underpinning works less than 3.0m deep in clays Basements up to 2.0m deep in clays

11. Scheme Details

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

Scope of Works

The works comprise:

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

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

In addition, the PC will have responsibility for the following:
- Review of methods of working/operations to limit movements, and
- Implementation of any emergency remedial measures if deemed necessary by the results of the monitoring.

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

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

Table 1: Instrumentation

General

The site excavations and substructure works up to finished ground slab stage have the potential to cause vibration and ground movements in the vicinity of the site due to the following:
a) Removal of any existing redundant foundations / obstructions;
b) Installation of reinforced concrete retaining walls under the existing footings;
c) Excavations within the site

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

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

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

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

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

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

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

Applicable Standards and References

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

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

SPECIFICATION FOR INSTRUMENTATION

General

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

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

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

Monitoring of existing cracks

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

Instrument Installation Records and Reports

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

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

Installation

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

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

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

Monitoring

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

The following accuracies/ tolerances shall be achieved:

Party Wall settlement	$\pm$1.75mm
Crack monitoring	$\pm$0.75mm

CROFT STRUCTURAL ENGINEERS

REPORT OF RESULTS AND TRIGGER LEVELS

General

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

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

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

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

Between the trigger points, which are no greater than 2 m apart, there should be no more than:

Allowable movement to BS5950 for brittle finishes

Vertical = Span / 360 = 3000mm / 360 = 8.33mm

Croft proposes a tighter recommendation of 4mm

Horizontal = Height / 500 = 3000mm / 500 = 6mm

Croft proposes a tighter recommendation of 3mm

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

MOVEMENT		CATEGORY	ACTION
Vertical	Horizontal		
0mm-4mm	0-3mm	Green	No action required
4mm-7mm	3-6mm	AMBER	Detailed review of Monitoring: Check studs are OK and have not moved. Ensure site staff have not moved studs. If studs have moved reposition. Relevel to ensure results are correct and tolerance is not a concern. Inform Party Wall surveyors of amber readings. Double the monitoring for 2 further readings. If stable revert back. Carry out a local structural review and inspection. Preparation for the implementation of remedial measures should be required. Double number of lateral props
7mm-10mm	6-8mm		Implement remedial measures review method of working and ground conditions
>10mm	>8mm	RED	Implement structural support as required; Cease works with the exception of necessary works for the safety and stability of the structure and personnel; Review monitoring data and implement revised method of works

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

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

Standard Reporting

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

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

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

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

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

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

The reports are to include progress photographs of the works both general to the area of each instrument and globally to the main Works. In particular, these are to supplement annotated plans/sections described above. Wherever possible the global photographs are to be taken from approximately the same spot on each occasion. The locations of these points on site are to be determined by the Engineer at detailed design stage.

Erroneous Data

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

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

Trigger Values

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

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

Responsibility for Instrumentation

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

APPENDIX A
MONITORING FREQUENCY

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

APPENDIX B

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

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

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

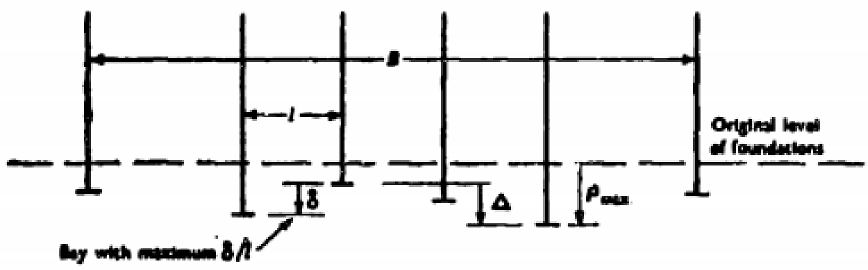


Figure 2.1 – Diagram illustrating the definitions of maximum angular distortion, δ/l, maximum settlement, ρ_{max}, and greatest differential settlement, Δ, for a building with no tilt (Skempton and MacDonald, 1956).

Figure 22: Diagram illustrating the definitions of maximum angular distortion, δ/l, maximum settlement, ρ_{max}, and greatest differential settlement ,Δ , for a building with no tilt (Skempton and MacDonald, 1956)

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

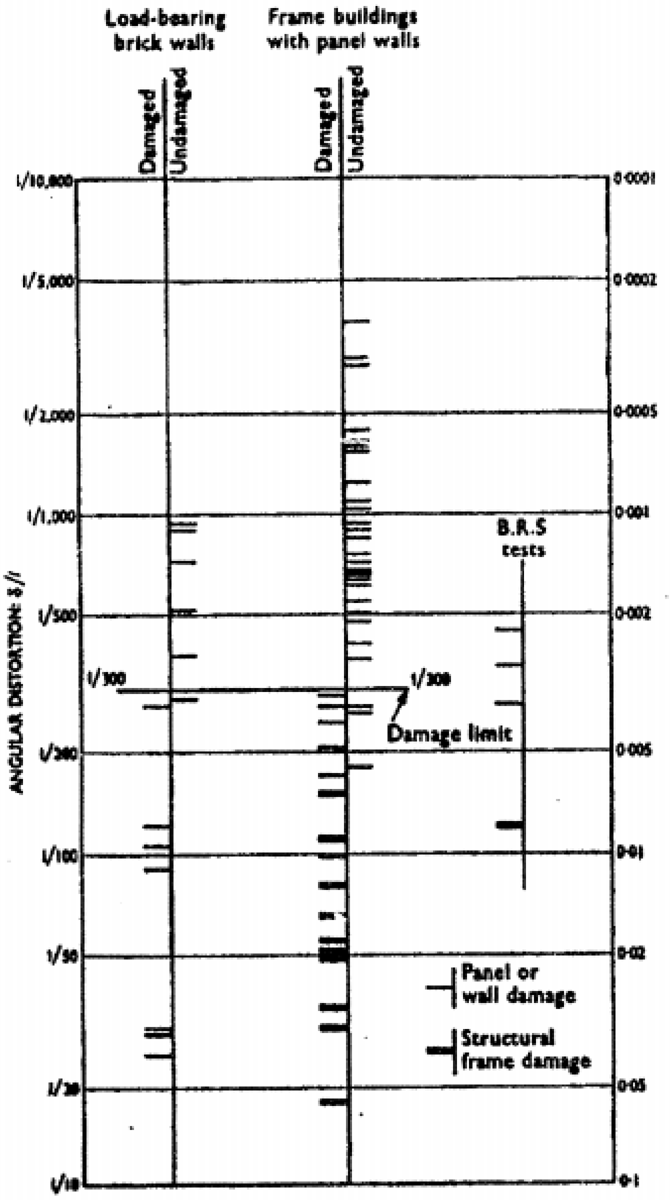
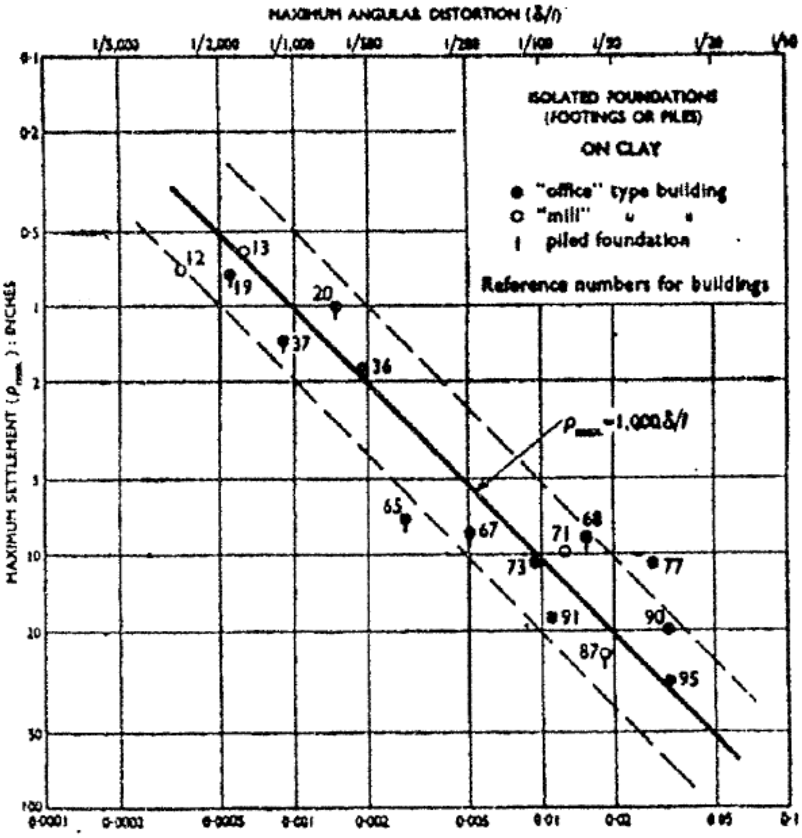
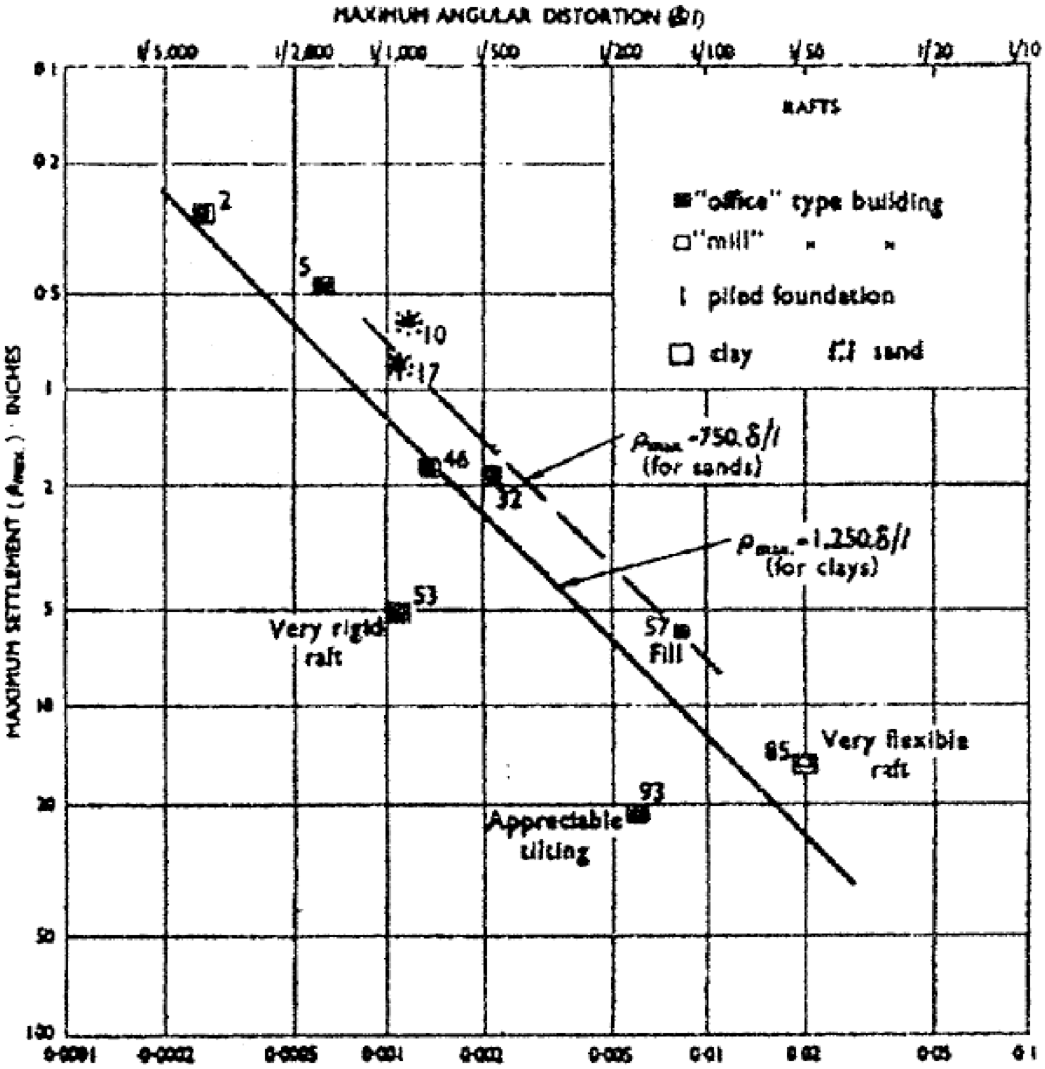


Figure 23: Skempton and MacDonald's analysis of field evidence of damage on traditional frame buildings and loadbearing brick walls

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

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





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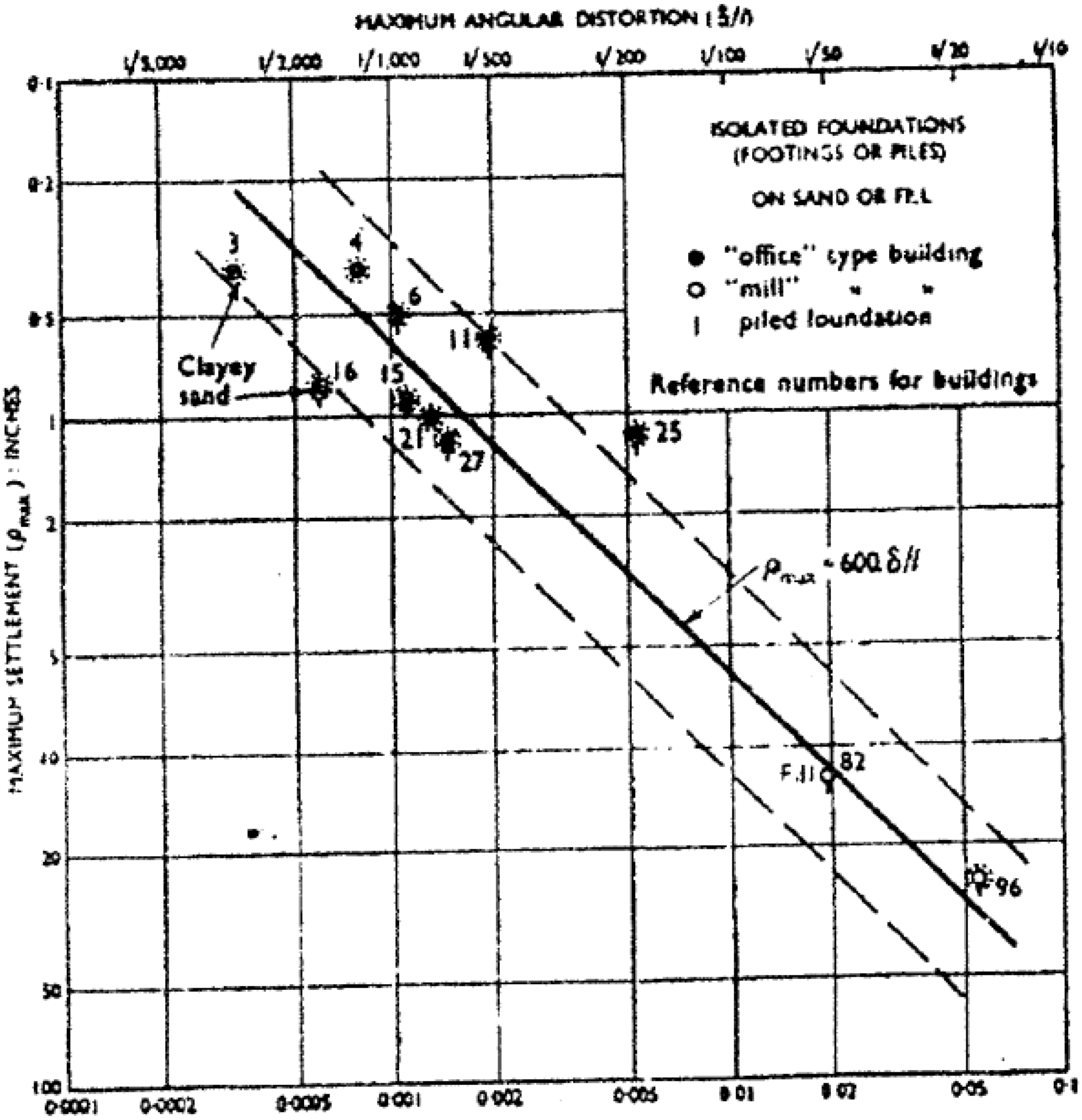

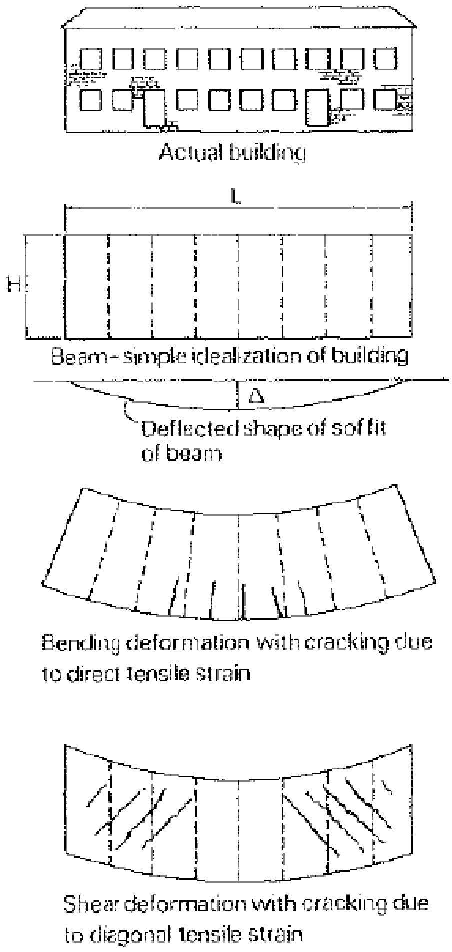

TABLE I

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