85 CAMDEN MEWS,
LONDON NW1

TEMPORARY WORKS FOR RETRO -FIT BASEMENT CONSTRUCTION

CALCULATIONS

Read in conjunction with 18005-TW drawings and all other permanent works drawings and details.
Refer to 18005-TW-400 for sequence of works.

References and background information:
- British Standards Codes of Practice

<u>Key site notes and constrains:</u>

- There are existing walls that are to be underpinned in short sections in underpinning sequence. Adjacent building 83 and 87 Camden Mews surcharge the excavations. In other cases conservative surcharge load of 5kN/m2 is applied to temporary works to account for unknowns and intermediate walls.

- Existing walls that are to be retained and restrained at all times as per 18005-TW

Prepared by	Issued
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Introduction

Temporary works are required to enable construction of retro-fit basement and new rear lightwell at the above site.

Permanent works proposals are as per Axiom Structures drawings.

Substructure
- Basement propping design for DIG and DIG2 (To formation level)
- Temporary continuous pre-bent bars in bases below side wall check

Refer to next pages for calculation checks.

Loads Taken (Temporary Condition):

- We allowed that the building is to be partly demolished before basement works commence.

Timber Roof => Dead = 0.75 kN/m2, Imposed Load = 0.75 kN/m2
All timber floor areas => Dead = 0.5 kN/m2, Imposed load = 0.75 kN/m2
Solid Masonry walls at 20kN/m3

Supporting Documents:
London Geological Survey Maps, ground investigation report, CIRIA 111.

Lateral Stability, Load Path & Disproportionate Collapse:
Retained walls to be restrained by rakes shores as 18005-TW drawings.

Soil Type & Foundation:
Allowable ground bearing pressure is assumed to be **100kN/m2** + 50kN/m2 (unloading)) and to be reviewed on site in the first pin.

BASEMENT AND RETAINING WALLS DESIGN PARAMETERS AND REMARKS

Ground parameters (CLAY) as per ST Consult.

φ = 24 => Ka = 0.41

Weights:
Density of ground = 20kN/m3, Submerged density of retained mat = 13.3kN/m3,
Saturated Soil = 23kN/m3
Water = 10kN/m3, **but water is not considered as lateral load in calculations**
Concrete = 24kN/m3

Factors of safety = γf	= 1.2 Earth	
	= ground water not considered	
	= 1.4 Dead	
	= 1.6 Imposed	
Surface Surcharge =	Side properties Ps =>	Qk = 5kN/m2 (allowed for unknown condition)
	Road (Front and side) =>	Qk = 10kN/m2
	Adjacent Buildings =>	N/A

Overall Sliding – by inspection of fully buried structure cross props and braces, sliding is sustained by friction effect and passive pressure of new walls.
- refer to next pages for checks of localised overturning, sliding and bearing on soil

Loading Allowances

Thickesses of the walls as surveyed on site

Loading:

	Dead: kN/m2	Imposed	Total
Timber Floors:		(house)	
Finishes = allowance	0.10	0.75	
18mm T&G floor deck	0.10		
Floor Joists (200x47 at 400crs at 550kg/m3)	0.15		
12.5mm Ceiling p/board finish at 10kg/m3	0.15		
Partitions allowance	0.25		
SLS=	0.75	0.8	1.5
ULS=	1.1	1.2	2.3

	Dead: kN/m2	Imposed	Total
New Ceiling Timber:		L/W storage	
Insulation	0.02	0.3	
Joists	0.08		
Ceiling Plaster	0.15		
SLS=	0.3	0.3	0.6
ULS=	0.4	0.5	0.8

	Dead: kN/m2	Snow		Total
Roof Structure		on plan		
Slate Tiles	0.50	sb =	0.6	
Battens, roof underlay, boarding	0.07	pitch=	26	
Roof Rafters	0.08			
100mm Insulation Boards	0.02	0.6		
Plasterboard ceiling	0.13			
F=	0.8			
On plan = F / cos25°=	0.89			
SLS=	0.9	0.6		1.5
ULS=	1.2	1.0		2.2

	Dead: kN/m2	Imposed	Total
Timber Staircase:			
Finishes = Lightweight	0.25	0.75	
Timber Structure	0.25		
Ceiling and Services	0.25		
SLS=	0.75	0.8	1.5
ULS=	1.1	1.2	2.3

Wall Line Loads

	Dead: kN/m2	Height	Total
New Internal non-LB Walls:			DLxH
Framing 50x75 at 400c/c 0.10	0.1		
2x12.5mm plasterboard (10kN/m2)	0.2		
SLS=	0.3	3.0	0.9
ULS=	0.4	3.0	1.3

or distributed on plan = 0,25kN/m2

	Dead: kN/m2	Height	Total
Existing/New External Brick Walls:			DLxH
225 Brickwork (at 20kN/m3)	4.5		
Internal S/C/L finish plaster	0.5		
SLS=	5.0	3.0	15.0
ULS=	7.0	3.0	21.0

AXIOM
STRUCTURES

w1 -Side wall in towards neighbour's garage

	L		Dead		Live		DL+IL
BO= Roof x 3.2m	3.20 =	3.20	0.9	2.8	0.60	1.9	4.8
BO= 1st Floor x 6.7m/2	3.35 =	3.35	0.75	2.5	0.75	2.5	5.0
Wall (225) x 4.8m	4.8 =	4.80	5.0	24.0	0.00	0.0	24.0
			sum kN	29		4	
					SLS=	34	
					ULS=	48	
Basement Wall (300) x 1.0h	1.0 =	1.0	7.8	7.8	0.00	0.0	8
			sum kN	37		4	
					SLS=	42	
					ULS=	59	

Unloading soil pressure say = 18kN/m2 x 1.4m (h	25 kN/m2		
Allowable pressure = Stiff Clay =	100 kN/m2		
with unloading allowance =	125 kN/m2		
		0.33 m	**Use min. 0.4m temporary base**

Southern Testing
Environmental & Geotechnical

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Basement Impact Assessment: Site Investigation Report
Revised from J11954 September 2014

Desk Studies | Risk Assessments | Site Investigations | Geotechnical | Contamination Investigations | Remediation Design and Validation

Site: 85 Camden Mews, London NW1 9BU

Client: Whitehall Park Ltd

Report Date: January 2015

Project Reference: J12115

EXTRACT

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SUMMARY

The site comprises a two-storey mews building with an attached single garage. There is a garden area to the rear of the property. It is proposed to refurbish and extend the existing mews building, to provide a three storey residential property including a single level basement.

Geological records indicate the site to be underlain by London Clay.

Two phases of intrusive investigation were carried out.

The soils encountered comprised superficial made ground over clays presumed to be Head, over London Clay.

Groundwater was encountered associated with thin gravelly clays in two of the exploratory holes, and to a lesser extent in two other holes. The gravelly clay appears to occur as discrete bodies and it is uncertain whether this material will be encountered at all in the proposed basement excavation, though some allowance should be made for excavation dewatering.

The sulphate content of the fill and natural soil was found to fall within Class DS-2. The ACEC site classification is AC-2.

The development includes a basement which is anticipated to be constructed using conventional underpinning methods. Parameters for retaining wall design are given.

The design of the new basement foundation system should take account of the nature of the existing/adjacent foundations and their condition, the presence of trees, and heave across the base of the excavation from soil unloading. Consideration must also be given to the potential surface water flooding risk.

D Vooght MSc
(Countersigned)

L D Mockett PhD PGDip FGS
(Signed)

For and on behalf of Southern Testing Laboratories Limited

STL: J12115
23 January 2015

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A INTRODUCTION

1 Authority

Our authority for carrying out this work is contained in an STL Order from Mr B Frazer of Whitehall Park Ltd, dated 4th August 2014. A second phase of investigation was authorised by e-mail, dated 23rd December 2014.

2 Location

The site is located in a residential road about 0.75 km to the northeast of Camden Road railway station. The approximate National Grid Reference of the site is TQ 296 847.

3 Proposed Construction

It is proposed to refurbish and extend the existing mews building, to provide a three storey residential property including a single level basement. The work will include the demolition of the existing single garage and small single storey extensions to the rear of the main building, and construct a new two-storey extension on the site of the garage. A single level basement is to be installed across the whole of the new footprint, with a small extension to part of the rear elevation, to provide a small basement courtyard area.

4 Object

The object of the investigation was to assess foundation bearing conditions and other soil parameters relevant to the proposed development. An initial Basement Impact Assessment (screening & scoping) was undertaken and this report addresses some of the issues that arose from that exercise.

5 Scope

This report is a revision of our initial report produced for the site, ref J11954 dated September 2014, incorporating the findings of a supplementary phase of intrusive investigation. A thin layer of apparently water-bearing gravelly clay was found in the initial investigation but the origin and extent of this feature was uncertain and the supplementary boreholes were intended to provide more detailed information to resolve the uncertainties. This report presents our exploratory hole logs and test results and our interpretation of these data.

As with any site there may be differences in soil conditions between exploratory hole positions.

This report is not an engineering design and the figures and calculations contained in the report should be used by the Engineer, taking note that variations will apply, according to variations in design loading, in techniques used, and in site conditions. Our figures therefore should not supersede the Engineer's design.

The findings and opinions conveyed via this Site Investigation Report are based on information obtained from a variety of sources as detailed within this report, and which Southern Testing Laboratories Ltd believes are reliable. Nevertheless, Southern Testing Laboratories Ltd cannot and does not guarantee the authenticity or reliability of the information it has obtained from others.

The site investigation was conducted and this report has been prepared for the sole internal use and reliance of Whitehall Park Ltd and the appointed Engineers. This report shall not be relied upon or transferred to any other parties without the express written authorization of Southern Testing Laboratories Ltd. If an unauthorised third party comes into possession of this report they rely on it at their peril and the authors owe them no duty of care and skill.

The recommendations contained in this report may not be appropriate to alternative development schemes.

B DESK STUDY & WALKOVER SURVEY

5 Desk Study

A desk study has been carried out. Reference has been made to the following information sources.

- Geological Maps
- Online Historical Ordnance Survey Maps
- Environment Agency website
- Camden Borough Council website
- Bomb Maps
- BRE Radon Atlas[1]

The data compiled for this desk study comprises publicly available information together with data from third parties, some of which is under review. Accordingly, Southern Testing Laboratories Limited does not warrant its accuracy, reliability or completeness.

5.1 Geology

The British Geological Survey Map No 256 indicates that the site geology consists of London Clay.

London Clay

London Clay is a well-known stiff (high strength) blue-grey, fissured clay, which weathers to a brown colour near the surface. It contains thin layers of nodular calcareous mudstone - "claystone" - from place to place, and crystals of water clear calcium sulphate (selenite) are common.

5.2 Hydrology and Hydrogeology

Data from the Environment Agency and other information relating to controlled waters is summarised below.

Data		Remarks	Possible Hazard to/from Site Y/N
Aquifer Designation	Superficial Deposits	No superficial Deposits present.	N
	Bedrock	Unproductive Strata.	N

[1] BR 211 (2007) 'Radon: guidance on protective measures for new buildings'

Data	Remarks	Possible Hazard to/from Site Y/N
Groundwater Vulnerability	Non-Aquifer.	N
Abstractions	The site on the EA website on 21st August 2014 does not show any abstractions in the vicinity of the site area.	N
Source Protection Zones	The site on the EA website on 21st August 2014 is not shown within an area mapped as overlying a SPZ.	N
Surface Water Features	There are no surface water features near the site. The nearest is the Regents Canal, around 800m to the south west.	N
Marine/Fluvial Flood Risk	The site on the EA website on 21st August 2014 is not shown within an area mapped as being at risk.	N
Surface Water Flood Risk	The EA website on 21st August 2014 shows small areas of Camden Mews near the site mapped as being at low risk.	Y
Reservoir Flood Risk	The site on the EA website on 21st August 2014 is not shown within an area mapped as being at risk.	N

The site would appear to be at potential risk from surface flooding (also highlighted in BIA screening/scoping); this should be accommodated in the basement design.

5.3 Historical Map Search

A viewing of publicly available (online) historical Ordnance Survey maps indicates that the site was developed with a mews building prior to the earliest map (1873), and pre-dates the development of the mews buildings to either side and opposite, which were developed through the 20th Century. The surrounding area has a history of residential use.

5.4 Other Sources

Camden Borough Council's planning website indicates that one planning application for the subject property was conditionally granted in 1953, for the erection of a garage to be used for the storage of a private car only: ref G13/13/7/15918.

With reference to The London County Council 'Bomb Damage Maps 1939-1945'[2], this site was not subject to damage during WWII.

5.5 Radon Risk

With reference to HPA and BGS guidance: no radon protection is required on this site.

[2] London Topographical Society 2005.

6 Walkover Survey

A walkover survey was carried out on 26th August 2014.

6.1 General Description

The site consists of a two storey mews building, with an adjoining single storey single garage, located on Camden Mews. Camden Mews has similar properties, which consist of single and two storey garages and residential mews buildings. No properties in the vicinity of the site have basements, apart from No. 60 Camden Mews, immediately opposite the site, which has a single storey basement.

The subject property has two garages located on the ground floor, fronting onto Camden Mews, along with a further single storey garage located to the south west of the main building, bounding the property with No. 83 Camden Mews.

There is a small garden at the rear of the property. The garden is bounded by the gardens of neighbouring properties, with brick walls forming boundaries to the north east with No.87 and the southwest with No.83. The garden backs onto the garden of No. 236 Camden Road, with a 1m high wooden fence.

There are several shrubs in the garden, and two larger, semi-mature trees in neighbouring gardens, around 10m to 15m from the rear of the property, these comprise a Lime and a False Acacia, both around 10m to 12m high. There are also some smaller trees including a plum tree and a (possible) mimosa around 4m to 5m from the rear of the building. Along Camden Mews, there is a Birch tree (8-10m high) opposite the site, around 7m from the front of the property. To the NE and SW of the site, along Camden Mews are a Lime tree and a Sycamore tree, around 25m and 30m from the site respectively; both trees are around 12m high and appear to have been pollarded.

In terms of topography, the site is relatively level, with a slight slope to the west. In the surrounding area there is a gentle fall of around 2^o to 3^o to the south west. There is a similar fall along Camden Mews.

C SITE INVESTIGATION

11 Method

The strategy adopted for the intrusive investigation comprised the following:

- 2 No 6m deep boreholes were drilled using a light percussion window sampler (WS1 & WS2) in August 2014.

- 3 shallow hand excavated trial trenches were dug to expose the existing foundations.

- 2 No additional 5.6m to 6m deep boreholes were drilled using a light percussion window sampler (WS3 & WS4) in January 2015.

- Groundwater monitoring wells were installed in the boreholes.

Exploratory hole locations are shown in Figure 1 in Appendix A.

12 Weather Conditions

The fieldwork was carried out on 26th August 2014, at which time the weather was wet, during a period of changeable, showery weather, and on 8th January 2015, at which time the weather was also wet.

13 Soils as Found

The soils encountered are described in detail in the attached exploratory hole logs (Appendix A), but in general comprised a thin covering of made ground over clays over London Clay. A summary is given below.

Depth	Thickness	Soil Type	Description
GL to 0.4/0.65m	0.4m to 0.65m	Made Ground	Brown to black slightly sandy clay MADE GROUND with gravel size fragments of brick, concrete, ceramic, marble and oyster shell. Concrete surface in TP1, WS2 and WS4.
0.4/0.65m to 3.2/4.2m	2.8m to 3.7m	Clay	Firm to stiff, medium to high strength, orange brown slightly silty CLAY.
3.2/4.2m to 3.4/4.35m Seen in WS1 & WS2 only	0.15 to 0.2m	Gravelly Clay	Stiff to very stiff, high to very high strength, orange brown gravelly CLAY. The gravel comprises fine to medium sub-rounded to rounded flint.
3.4/4.35m to >5.6/6.0m Seen in WS1-4 only	Thickness unproven	Clay	Stiff to very stiff, high to very high strength orange brown CLAY. Sandy below 5.6m in WS1.

A thin layer of gravelly clay was found in the initial window sampler holes, at 3.2m below ground level in WS1 and 4.2m below ground in WS2. No gravelly clay layer was found in the supplementary holes, WS3 and WS4 and, therefore, it is thought that there is not a consistent gravelly clay deposit across the site. Rather, it appears that the gravelly clays encountered are discrete bodies.

The proposed basement excavation will likely extend to between 3m and 3.5m below the existing site levels and may encounter the gravelly clay as found in WS1, which is located immediately adjacent to the footprint of the proposed basement; the remaining boreholes are within the proposed basement footprint. No gravelly clay was found in the three boreholes within the basement area, within the anticipated depth of excavation.

In considering the engineering properties of the soils, the gravelly clay and the overlying clay are assumed to be a Head deposit.

13.1 Visual and Olfactory Evidence of Contamination

No obvious evidence of possible contamination was recorded during the fieldwork other than the presence of superficial made ground, which can contain elevated levels of some contaminants.

13.2 Existing Foundations

The existing foundations to boundary walls were exposed in hand dug trial pits. The arrangement of the foundations is shown in the sections in Appendix A; foundations are at 0.52m to 0.85m below ground level, formed in the natural clay soils.

14 Groundwater Strikes

Water was encountered in the exploratory holes as follows:

BH	Water Strikes
WS1	Sample tube wet at 3.4m depth. This is coincident with the gravelly clay.
WS2	Water on sample tubes from 5.1m.
WS3	None
WS4	None

The shallow pits were dry, although TP3 filled with rainwater.

D FIELD TESTING AND SAMPLING

The following in-situ test and sampling methods were employed. Descriptions are given in Appendix B.

- Disturbed samples
- Hand Penetrometer tests

E GEOTECHNICAL LABORATORY TESTS

The following tests were carried out on selected samples. Test method references and results are given in Appendix C.

- Moisture content & Atterberg Limit determinations
- Soluble sulphate & pH value determinations

F DISCUSSION OF GEOTECHNICAL TEST RESULTS AND RECOMMENDATIONS

15 Soil Classification and Properties

Soil Type	Depth	Compressibility	VCP	Permeability	Frost Susceptible	CBR	Remarks
Made Ground	GL to 0.4/0.65m	Potentially high	N/A	Variable	Potentially	Poor	Not suitable for foundations
Clay	0.4/0.65m to 3.4/4.35m	Medium to high	High	Very Low generally	No	Poor	Possible groundwater inflow from gravelly horizons
London Clay	3.4/4.35m to >6m	Low to medium	High	Very Low generally	No	Poor	Seepages on fissures possible

16 Swelling and Shrinkage

The Atterberg Limits tests carried out classify the clay soils as clays of very high plasticity (CV). The measured Plasticity Index values are in excess of 40% and fall within the NHBC High Volume Change classification.

Given the proximity of trees to the structure, particularly to the front and rear, moisture content and hand penetrometer profiles were taken, to check for the presence of desiccation.

16.1 Desiccation

No single factor can be used to assess the degree of desiccation of soils but some of the more commonly used criteria are listed below:-

1. If the soils are below a moisture content of 0.5 x liquid limit, measured by the cone method, they can be considered desiccated, but heave will not necessarily occur when the tree is removed.
2. If the soils are below a moisture content of 0.4 x liquid limit[3] then they are strongly desiccated and heave is likely after trees are removed.
3. Soils such as London Clay are usually found to have a moisture content that is close to the Plastic Limit, below a depth of about 4.0m. Above that depth softening occurs and the moisture content rises to Plastic Limit +2 to 4% where the soil is unaffected by trees. A typical profile would be a moisture content of PL + 3% at 1.0m reducing to PL + 1% at 3.0m.[4]

[4]R Driscoll - The influence of vegetation on the swelling and shrinkage of clay in Great Britain - Geotechnique, June 1983

[3]Samuels S.G. (1967) - The uplift of buildings on swelling clays BRS internal note IN40/67 BRE Watford

4. London Clay is usually considered to be significantly desiccated where the moisture content is less than 30%

Desiccation can also be assessed using hand penetrometer tests (after Pugh, Parnell & Parkes - January 1995), where the intact strength of clay is measured at intervals. By comparing the unconfined compression strength of the soil with the typical range of values for equilibrium conditions, the large increases in effective stress resulting from decreases in pore pressure (a direct result of desiccation) are identified graphically[5].

Plots of moisture content, Atterberg Limits parameters and hand penetrometer readings are given in Appendix D.

The measured moisture contents are above 30% and vary little over the test depth in either hole. The moisture content profiles are generally consistent with those expected for clays not affected by trees. In WS1, the moisture content results are below 0.5 of the Liquid Limit but do not fall below 0.4 of the Liquid Limit. In WS2, the moisture content results are also below 0.5 of the Liquid Limit, and straddle the 0.4 Liquid Limit profile below about 1m.

In considering all of the above observations, it is considered that the soils tested are not highly desiccated, and that the potential for the clays in WS2 to heave is marginal. This is consistent with the moderate water demand trees present in the vicinity of the site. However, the Engineer should check their influence using the guidance in NHBC Chapter 4.2 and make sure that the design caters for the potential effects of lateral pressure/heave from the trees in the future.

17 Groundwater Levels

Monitoring wells were installed in the four window sampler boreholes. Monitoring visits were undertaken following installation, as follows:

BH (Well Depth)	Water Level mbgl 26/08/2014	Water Level mbgl 03/09/2014	Water Level mbgl 15/09/2014	Water Level mbgl 08/01/15	Water Level mbgl 16/01/15
WS1 (5.9m)	Dry (at installation)	3.23	1.82	1.04	0.87
WS2 (6.0m)	Dry (at installation)	0.72	0.81	0.64	0.55
WS3 (4.9m)	-	-	-	Dry (at installation)	3.10
WS4 (5.9m)	-	-	-	Dry (at installation)	4.93

The four wells were dry at the time of installation. Whilst groundwater was observed during drilling of WS1 and WS2, inflows were not substantial. The subsequent monitoring shows differing responses between the wells. In WS1, the measured groundwater level appears to rise

[5] A rapid and reliable on-site method of assessing desiccation in clay soils. R. S. Pugh, P. G. Parnell, and R. D. Parkes Proc I.C.E. Geotech Engng 1995, 113,Jan., 25 - 30

very slowly during the early monitoring period, whereas in WS2 the measured groundwater level is significantly higher initially. This may indicate a significantly lower permeability in the gravelly clay in WS1. Much lower water levels were recorded in WS3 and WS4.

The two wells located to the rear of the existing building are at slightly lower topographic levels than the two inside the building. With regard to WS1 and WS2, this difference increases the apparent difference in water level between these two wells, and supports the idea that the gravelly clays encountered are discrete bodies.

Groundwater levels vary considerably from season to season and year to year, often rising close to the ground surface in wet or winter weather, and falling in periods of drought. Long-term monitoring from boreholes or standpipes is required to assess the ground water regime and this was not possible during the course of this site investigation.

On the basis of the measurements to date, some groundwater ingress should be anticipated during construction and some allowance should be made for dewatering. Flow rates are unlikely to be significant, and intermittent pumping from strategically placed collector sumps should be adequate.

For the longer term condition, the presence of groundwater should be allowed for in the design of the basement e.g. provision of drainage cavity/tanking, and also for hydrostatic uplift of the floor slab. Equilibrium standing water levels should be anticipated at around ground level for design purposes.

As noted above, the gravelly clay bodies encountered are likely to be of very limited lateral extent and, accordingly, there would not be any significant groundwater flow associated with them. Furthermore, the basement construction may not intercept these bodies. Therefore, bearing in mind the negligible permeability of the clay soils, there is minimal risk of the proposed basement construction causing a "damming effect" or mounding of water on the up-gradient side.

Similarly, and in terms of the potential cumulative effects of other basements being constructed in the future in the immediate area, these should have little influence on groundwater levels.

On the basis of the above, it is concluded that the proposed development is unlikely to result in any specific issues relating to the hydrogeology and hydrology of the site.

18 Sulphates and Acidity

The recorded pH values within the natural soils are in the range 6.9 to 7.8 being generally near neutral in reaction. The made ground sample gave a slightly acidic result of 5.7.

The Design Sulphate Class is DS-2. Groundwater should be assumed to be mobile due to the recorded seepages into the monitoring wells. The ACEC site classification is AC-2.

19 Bearing Capacity & Foundations

The anticipated formation level of the proposed basement will be at around 3m to 3.5m below current ground level. At this depth, the base of the excavation and basement floors will be formed within the firm to stiff or stiff clay, at or above the level at which the gravelly clay was observed in WS2. For any foundations proposed at this depth a net allowable bearing pressure of 100 kPa would be available. Excavation of the basement will result in both immediate and long-term soil displacements associated with unloading of the clay soils. Heave precautions will be required in the design of the basement slab.

It is anticipated that the basement will be formed by conventional underpinning techniques.

20 Heave

Due to stress relief following the removal of the existing soils to form the basement structure, both immediate (undrained) and long term (drained) heave displacements can be expected to occur in the underlying clay.

The immediate (undrained) heave displacements will occur as excavation of the basement takes place and before the construction of basement elements e.g. slabs etc. Accordingly, only the long term (drained) heave displacements will need to be catered for in design, to overcome the problem of uplift pressures forming. This is normally overcome by installing appropriate void forming materials beneath the basement elements.

It is anticipated that the heave will be dominated by the underlying London Clay. For the analysis of heave movements the following stiffness parameters after Burland and Kalra (1986)[6] are suggested for the London Clay:

Undrained Young's Modulus (E_u) = (10+5.2z) (MN/m^2)

Undrained Poisson Ratio (ν_u) =0.5

Drained Young's Modulus (E_d) = (7.5+3.9z) (MN/m^2)

Drained Poisson Ratio (ν_d) =0.2

Where z (m) is taken from the surface of the London Clay

Calculations of the magnitude of any movements could be undertaken once design proposals and loading have been finalised.

21 Basement Construction

The following soil parameters are suggested for design of retaining walls:

Soil Type	Bulk density γ_b (kN/m^3)	Undrained Shear Strength (Temporary Condition) kN/m^2	Long Term Drained Condition	
			c' (kN/m^2)	φ^o
Made Ground	19	N/A	0	27
Clay (assumed) Head	20	60	0	25

[6] Burland J.B. and Kalra J.C. (1986) Queen Elizabeth Conference Centre: geotechnical aspects, Proc. Inst. Civ. Engnrs, Part 1,80,1479-1503

London Clay	20	125	0	25

22 Excavations and Trenching

Statutory lateral earth support will be required in all excavations where men must work. Instability of the sides of any open excavations carried out must be expected. Accordingly, measures should be taken at all times to ensure that excavations are adequately supported. Groundwater seepages into excavations should be anticipated, until suitable waterproofing measures have been employed.

Given the presence of the existing adjacent foundations, close attention in design of temporary and permanent propping is required at all times to prevent settlement or excessive lateral yielding of the excavation/foundations.

Southern Testing | ST Consult

Tel: 01342 333100

Project No.	Hole Type	Borehole No
J11954	WS	WS1
		Sheet 1 of 1

Project Name: 85 Camden Mews (London NW1)

Dates: 26/08/2014

Location: London NW1

NGR: -

Client: Whitehall Park Ltd

Level: -

Logged By AW

Water Strikes	Depth (m)	Type	Results	Level (m AOD)	Thickness	Depth (m)	Stratum Description
					0.40	0.40	Brown to black, slight sandy CLAY MADE GROUND with frequent fine to medium gravel sized, sub rounded to angular brick, concrete and occasional ceramic fragments.
	0.30	D					
	0.35	ES					
	0.40	D					
	0.50		UCS = 150		2.80	3.20	Firm to stiff, medium to high strength, orange brown slightly silty CLAY.
	0.50	ES					
	0.75		UCS = 140				
	1.00		UCS = 140				
	1.00	D					
	1.25		UCS = 160				
	1.50		UCS = 150				
	1.50	D					
	1.75		UCS = 130				
	2.00		UCS = 160				
	2.00	D					
	2.25		UCS = 150				
	2.50		UCS = 150				
	2.50	D					
	2.75		UCS = 130				
	3.00		UCS = 130				
	3.00	D					
▽	3.25		UCS = 190		0.20	3.40	Stiff, high strength, orange brown gravelly CLAY. Gravel is fine to medium sized rounded to sub rounded flint.
	3.50		UCS = 200		2.20	5.60	Very stiff, very high strength, orange brown CLAY
	3.50	D					
	3.75		UCS = 200				
	4.00		UCS = 250				
	4.00	D					
	4.25		UCS = 300				
	4.50		UCS = 250				
	4.50	D					
	4.75		UCS = 340				
	5.00		UCS = 290				
	5.00	D					
	5.25		UCS = 300				
	5.50		UCS = 270				
	5.50	D					
	5.75		UCS = 300		0.40	6.00	Very stiff, very high strength, orange brown, Sandy CLAY.
	6.00		UCS = 250				
	6.00	D					End of Borehole at 6.00 m

Hole Diameters

Depth (m)	Hole (mm)	Casing (mm)

Water Strikes

Date	Water (m)	Casing (m)	Time (mins)	Rose to (m)	Sealed (m)
25/08/2014	3.40	-	-	-	-

General Remarks:

Water in sample from 3.40m

PT = Equivalent Standard Penetration Test , UCS = Unconfined Compressive Strength (kN/m2) by Hand Penetrometer , HV = Hand Vane Result (kPa)

AXIOM
STRUCTURES

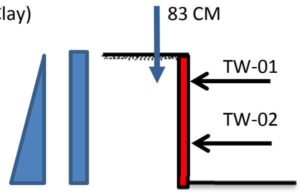
Introduction:
Permanent retaining wall is not sufficient to resist sliding and overturning without
connection with the basement slab, hence propping is to be adopted until base slab is constructed.

Refer to TW-400 for sequence of works.

Temporary Struts to Retaining Wall - 83/85 Camden Mews

D I G 1

g =	20	kN/m3	(unit weight)
fi=	25	degree	(Angle of internal friction)
Ko =	0.58		(Stiff Clay)
a =	0.4	m	
b =	0.6	m	
c=	0.45	m	
H =	1.45	m	
0.25H =	0.36	m	



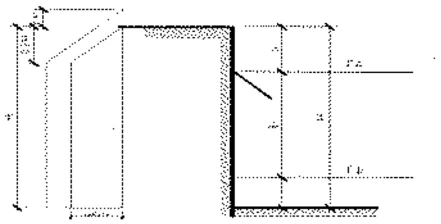
Active Pressure on Wall

Earth:

Pe at H =	k0 x g x H =	16.8	kN/m/m
Pe T =	PeH x H x 0.5 =	12.2	kN/m (conventional triangular shape)

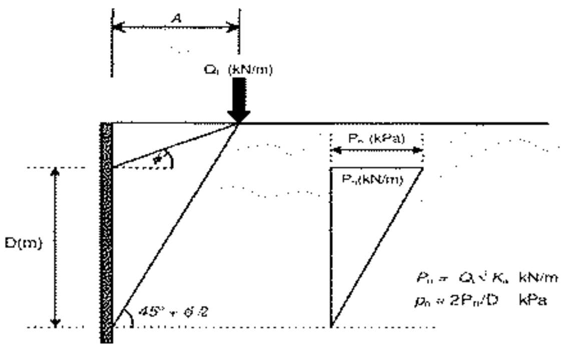
Surcharge

S =	(Party wall)	5	kN/m2
Ps =	S x k0 =	2.9	kN/m/m
Ps T =	Ps x H =	4.2	kN/m
he =		0.15	m



Surcharge from adjacent footings - from calculations refer to previous page - applied 0.85m BGL

Q =	43	kN/m
A =	0.27	m
θ =	25	degree
Ka =	0.41	
D(0) = A x tanθ =	0.1	m
D(1) = A x tan(45+θ/2)	0.4	m
D = D(1)-D(0) =	0.3	m
Pn = Q x (Ka)^0.5=	27	kN/m
pn = 2Pn / D	184	kN/m/m



Extract from Ciria C580

Water:
not considered in temporary condition

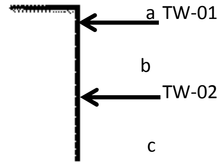
at 0	2.9	kN/m/m	
Pe+Ps at H=	19.7	kN/m/m	(short term temporary works)

Loads in line of struts:

Fa =	4	kN /m
Fb=	40	kN /m

TW-01=TW-02 (critical Fb):

Load to diagonal prop:



Lb=Max. spacing of props=	2.5	m
Axial = Fb x Lb =	**100.0**	kN

Load to raking strut TW-01 at angle =	45	degree	
Axial SLS = Fsls/ cos(90-angle) =	**141.4**	kN	
sideway to face = Fuls x tan(90 - angle) =	**100.0**	kN	Note that stiff corner effects not taken into account <-- conservatively

Use 4M20 R-kex resin bolts (Spacing= 150mm):

Ps,1=	85.1	kN/bolt	
f= factor for spacing=	0.87		
Ps=	296.1	kN	> Sideway=100kN OK

Check Raking Props TW-02:

Fuls =	212.1	kN
Mnom = 0.05 x Fuls	10.6	kNm

Try 152x152x37UC S355

Le =	**4**	m	
Pc =	630.0	kN	(Blue book)
Mb =	77.1	kNm	

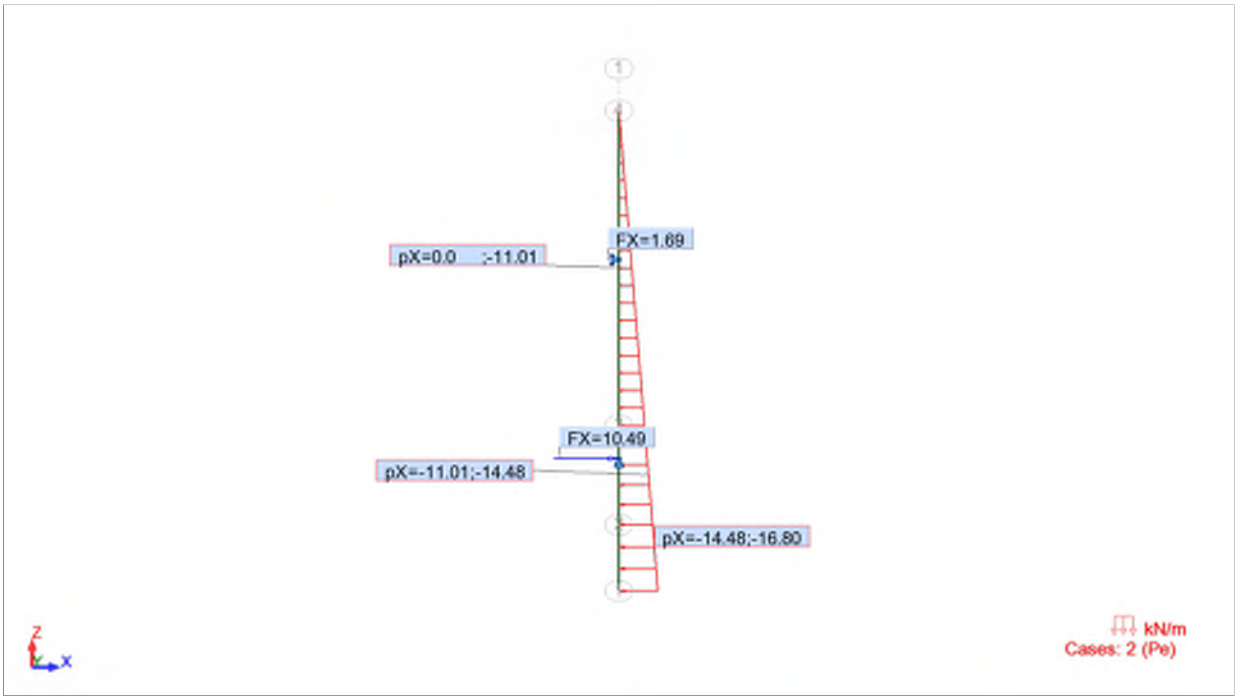
<u>Interaction check compression with bending</u>

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs	0.14				
	Myy/Mcy	0.00 =				
	F/Pcy =	0.34 =	0.47	<	1	**OK**

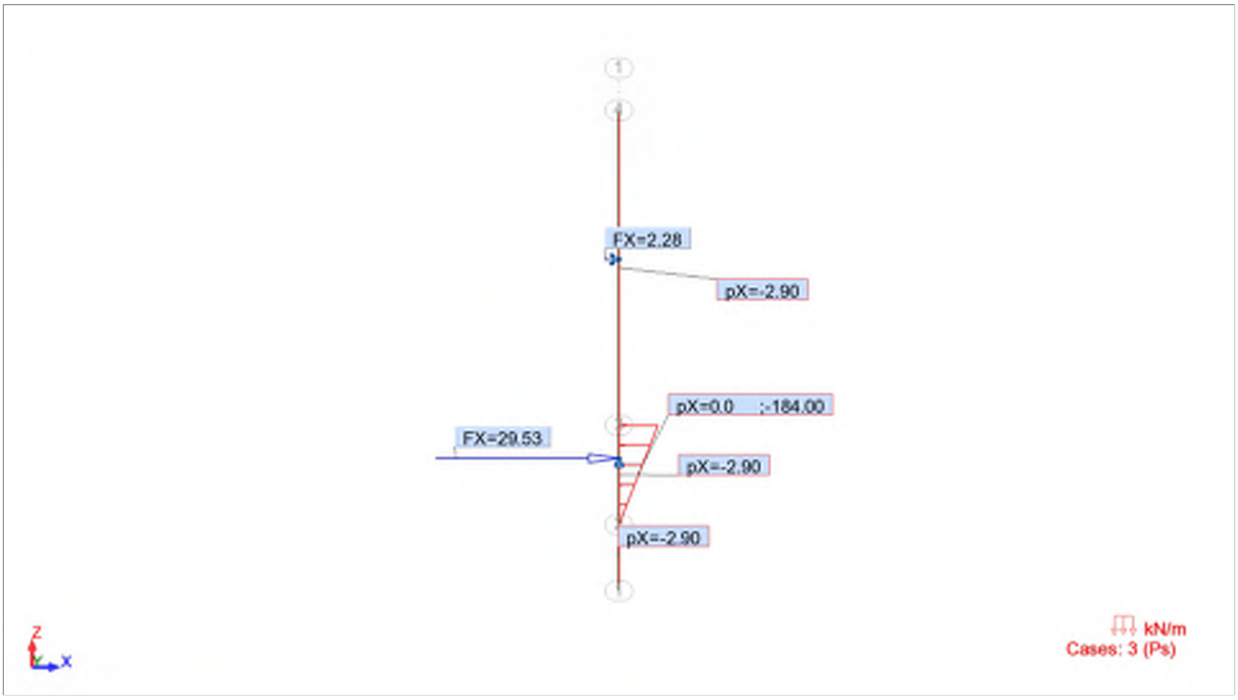
152UC37 is sufficient

Bearing on concrete face = F_v / 0.3 x 0.3 = 2.4 N/mm2 < 40N/mm2 / 1.5 = 26N/mm2

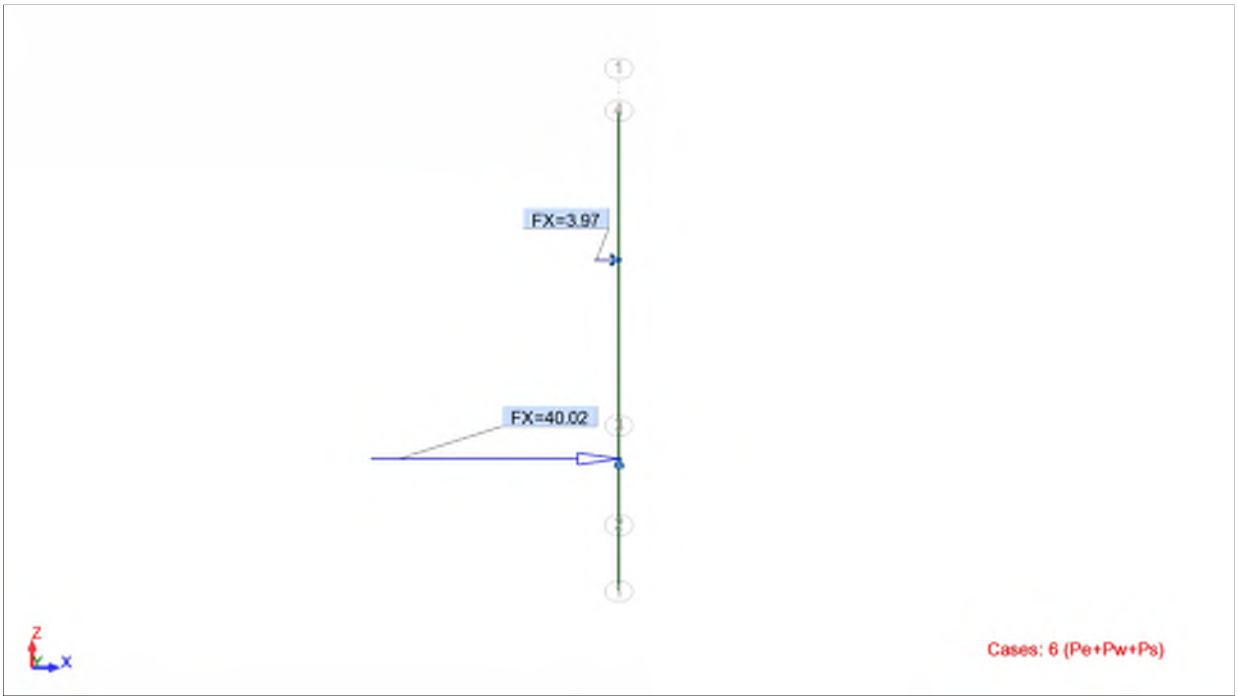
View - Reaction forces(kN,kN/m), Cases: 2 (Pe)



View - Reaction forces(kN,kN/m), Cases: 3 (Ps)



View - Reaction forces(kN,kN/m), Cases: 6 (Pe+Pw+Ps)

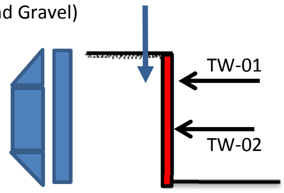


Introduction:
Permanent retaining wall is not sufficient to resist sliding and overturning without connection with the basement slab, hence propping is to be adopted until base slab is constructed.

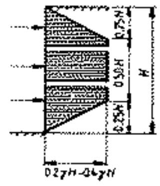
Refer to TW-400 for sequence of works.

Temporary Struts to Retaining Wall - 83/85 Camden Mews
D I G 1
According to BS8002

g =	20 kN/m3	(unit weight)
fi=	25 degree	(Sand and Gravel)
Ko =	0.58	
a =	0.4 m	
b =	0.6 m	
c=	0.45 m	
H =	1.45 m	
0.25H =	0.36 m	



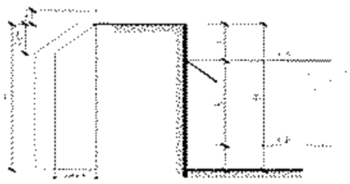
BS8002



Active Pressure on Wall

Earth according to BS8002:

Pe at 0.25H =	0.4 x g x H =	11.6 kN/m/m
Pe T =	Pe0.25H x 3/4H =	12.6 kN/m

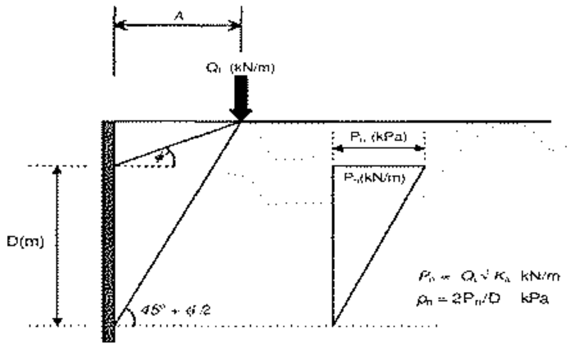


Surcharge

S =	(Party wall)	5 kN/m2
Ps =	S x k0 =	2.9 kN/m/m
Ps T =	Ps x H =	1.3 kN/m
he =		0.15 m

Surcharge from adjacent footings - from calculations refer to previous page - applied 0.85m BGL

Q =	43 kN/m
A =	0.27 m
θ =	0 degree
Ka =	1.00
D(0) = A x tanθ =	0.0 m
D(1) = A x tan(45+θ/2)	0.3 m
D = D(1)-D(0) =	0.3 m
Pn = Q x (Ka)^0.5=	43 kN/m
pn = 2Pn / D	319 kN/m/m



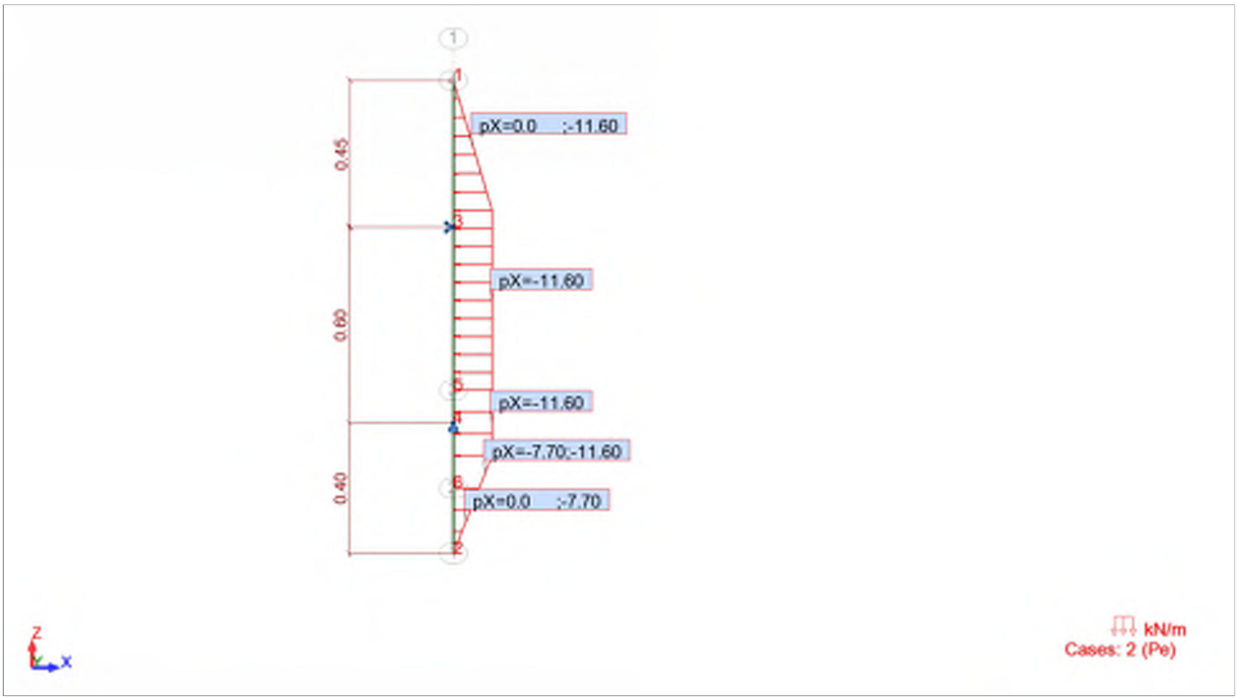
Water:
not considered in temporary condition

at 0	2.9 kN/m/m	
Pe+Ps at H=	14.5 kN/m/m	(short term temporary works)

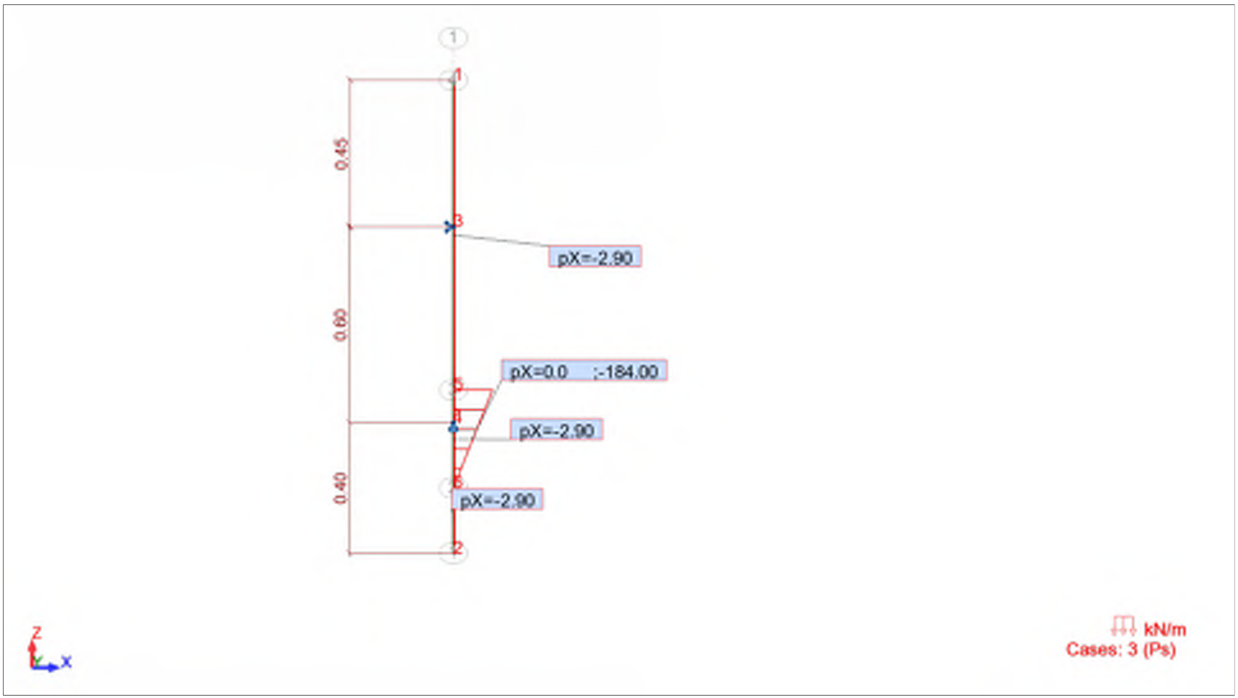
Loads in line of struts:

Fa =	8 kN /m	**TW-01 Shores designed for crtical load Fb, hence OK**
Fb =	36 kN /m	**Not critical**

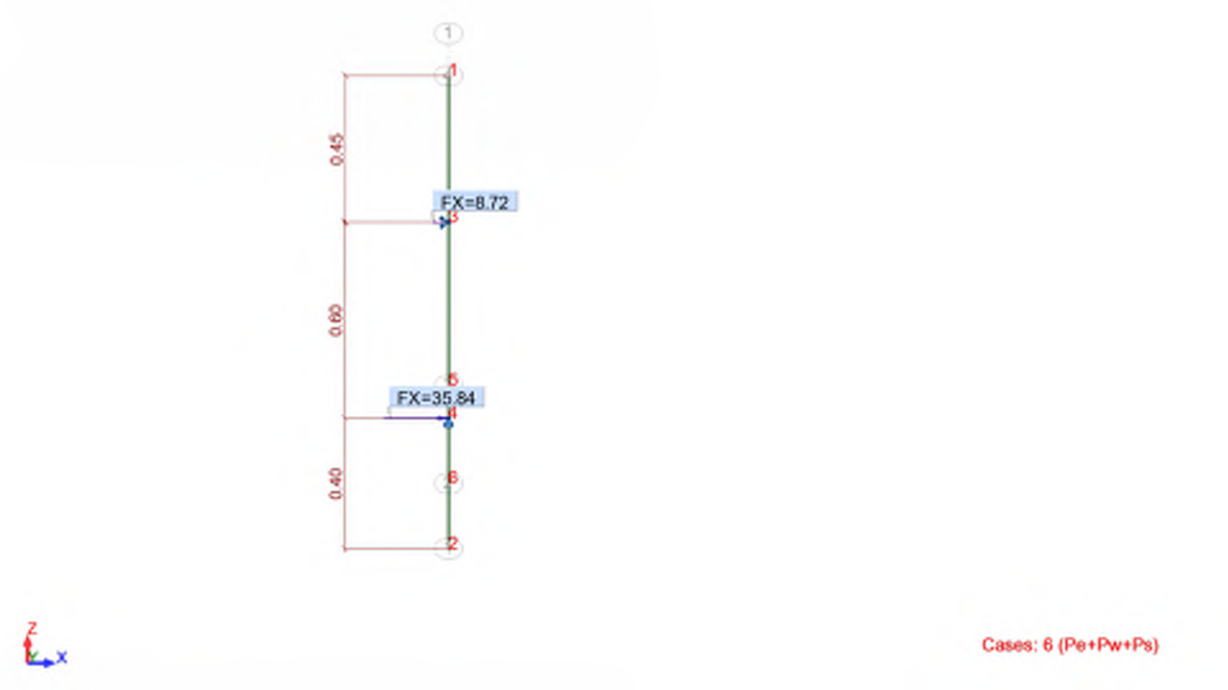
View - Cases: 2 (Pe)



View - Cases: 3 (Ps)



View - Reaction forces(kN,kN/m), Cases: 6 (Pe+Pw+Ps)



AXIOM STRUCTURES

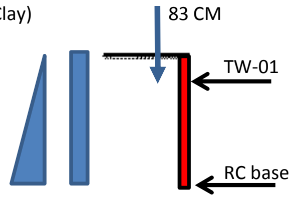
Introduction:
Permanent retaining wall is not sufficient to resist sliding and overturning without connection with the basement slab, hence propping is to be adopted until base slab is constructed.

Refer to TW-400 for sequence of works.

Temporary Struts to Retaining Wall - 83/85 Camden Mews

DIG 2

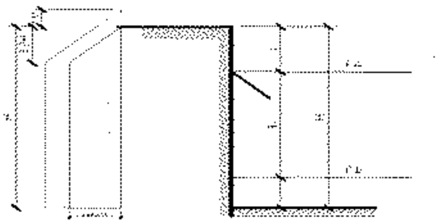
g =	20	kN/m3	(unit weight)
fi=	25	degree	(Angle of internal friction)
Ko =	0.58		(Stiff Clay)
a =	0.4	m	
b =	2.7	m	
c=	0.5	m	
H =	3.6	m	
0.25H =	0.90	m	



Active Pressure on Wall

Earth:

Pe at H =	k0 x g x H =	41.8	kN/m/m	
Pe T =	PeH x H x 0.5 =	75.2	kN/m	(conventional triangular shape)

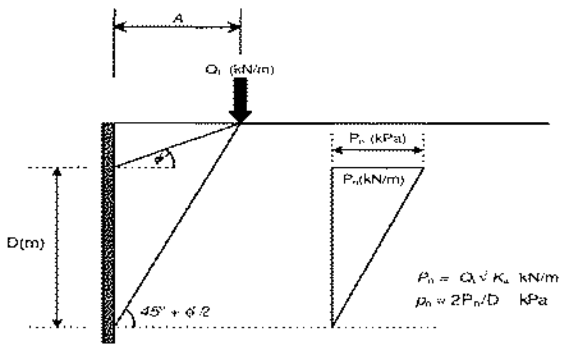


Surcharge

S =	(Party wall)	5	kN/m2
Ps =	S x k0 =	2.9	kN/m/m
Ps T =	Ps x H =	10.4	kN/m
he =		0.15	m

Surcharge from adjacent footings - from calculations refer to previous page - applied 0.85m BGL

Q =	43	kN/m
A =	0.27	m
θ =	25	degree
Ka =	0.41	
D(0) = A x tanθ =	0.1	m
D(1) = A x tan(45+θ/2)	0.4	m
D = D(1)-D(0) =	0.3	m
Pn = Q x (Ka)^0.5=	27	kN/m
pn = 2Pn / D	184	kN/m/m



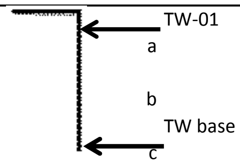
Water:
not considered in temporary condition

Extract from Ciria C580

at 0	2.9	kN/m/m	
Pe+Ps at H=	44.7	kN/m/m	(short term temporary works)

Loads in line of struts:

Fa =	57	kN /m
Fb=	56	kN /m



TW-01 prop check:

Load to diagonal prop:

Lb=Max. spacing of props= 2.5 m

Axial = Fb x Lb = **140.8** kN

Load to raking strut TW-01 at angle = 45 degree

Axial SLS = Fsls/ cos(90-angle) = **199.1** kN

sideway to face = Fuls x tan(90 - angle) = **140.8** kN Note that stiff corner effects not taken into account <-- conservatively

Use 4M20 R-kex resin bolts (Spacing= 150mm):

Ps,1= 85.1 kN/bolt

f= factor for spacing= 0.87

Ps= 296.1 kN > Sideway=167.3kN OK

Check Raking Props TW83:

Fuls = 298.6 kN

Mnom = 0.05 x Fuls 14.9 kNm

Try 152x152x37UC S355

Le = **4** m

Pc = 630.0 kN (Blue book)

Mb = 77.1 kNm

<u>Interaction check compression with bending</u>

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs	0.19				
	Myy/Mcy	0.00 =				
	F/Pcy =	0.47 =	0.67	<	1	**OK**

152UC37 is sufficient

Bearing on concrete face = F_v / 0.3 x 0.3 = 3.3 N/mm2 < 40N/mm2 / 1.5 = 26N/mm2

Load to Bottom of Pin

Design of Basement Toes to Work as Ground Beam between Walls:

From analysis = w_sls = 56.3 kN/m

L = Span bet'n footings to act as struts = **6.6 m**

Muls = FOS x Ruls x L^2 / 8 = **460** kNm (FOS=1.5)

INPUT	Location	<u>RC TW base</u>					
	Design moment, M	<u>460.0</u> kNm	fcu	35	N/mm²	gc = <u>1.50</u>	
	ßb	<u>1.00</u>	fy	500	N/mm²	gs = <u>1.15</u>	
	Span	<u>6600</u> mm	steel class	<u>A</u>			
	Height, h	**<u>1800</u> mm**	Comp cover		<u>50</u> mm to main reinforcement		
	Breadth, b	<u>350</u> mm	Tens cover		<u>50</u> mm to main reinforcement		
	Tens Ø	12 mm	Side cover		<u>50</u> mm to main reinforcement		
	Comp Ø	12 mm	Section location				

OUTPUT

RC Ground Beam

d = 1800 - 50 - 12/2 = 1744.0 mm

(3.4.4.4) K' = 0.156 > K = 0.006 ok

(3.4.4.4) z = 1,744.0(0.5 + (0.25 - 0.006/0.893)^½) = 1,732.8 > 1,656.8 mm

(Fig 3.3) fst = 434.8 N/mm²

As = 1E6 x 214 /1,656.8 /434.8 = 297 mm²

PROVIDE 6H12 tension steel = 678 mm²

fs = 2/3 x 500 x 297 /905 = 109.4 N/mm²

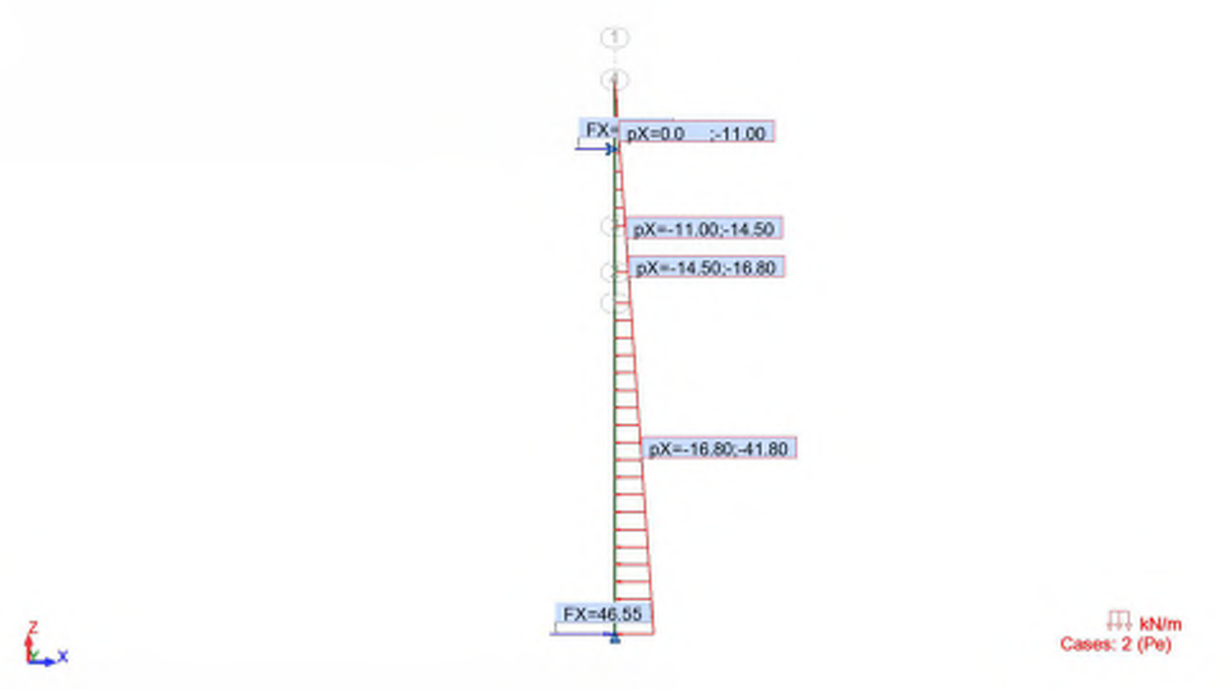
Comp mod factor = 1 + 0.037 /(3 + 0.037) = 1.012 < 1.5

Tens mod factor = 0.55 + (477 - 109.4) /120 /(0.9 + 0.201) = 3.332>2

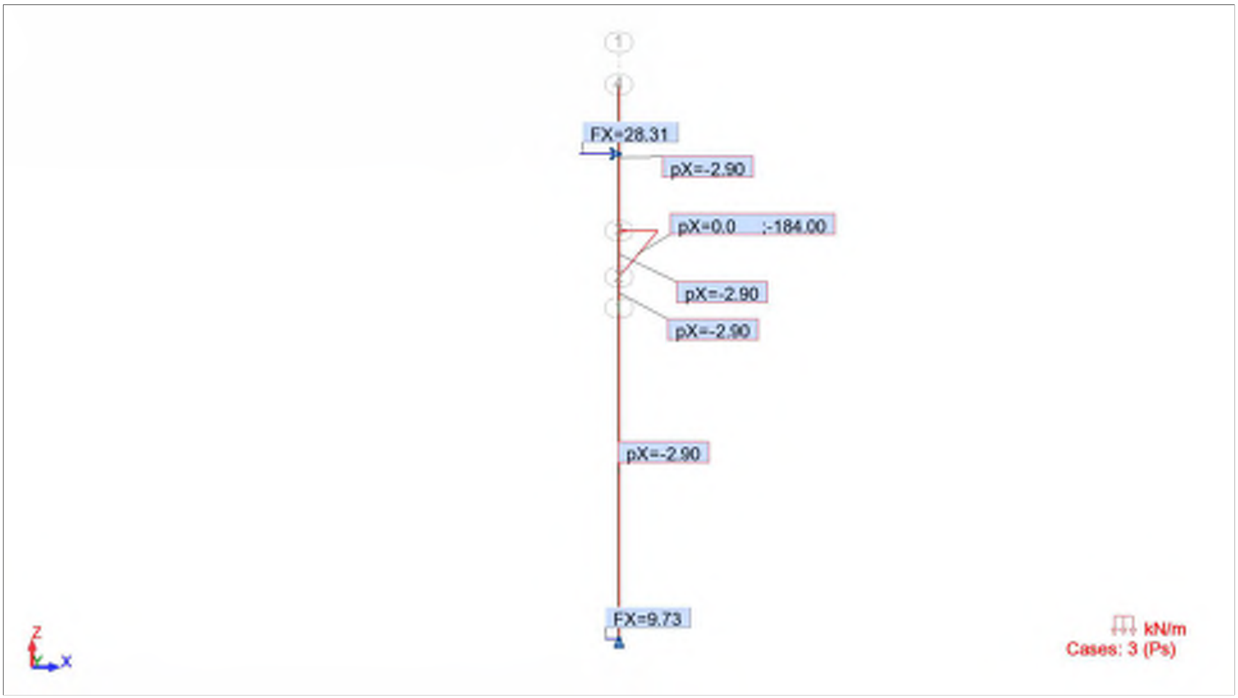
Permissible L/d = 26.0 x 1.012 x 2.000 = 52.634

Actual L/d = 4500 /1744.0 = 2.580 ok

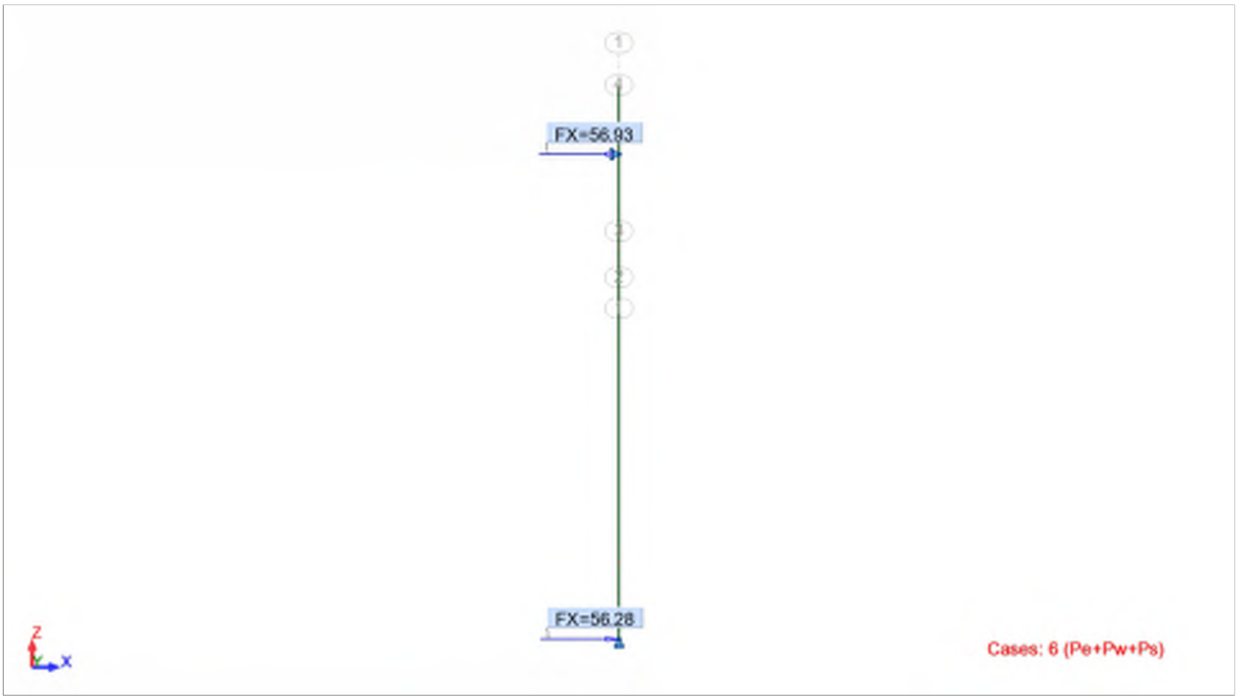
View - Reaction forces(kN,kN/m), Cases: 2 (Pe)



View - Reaction forces(kN,kN/m), Cases: 3 (Ps)



View - Reaction forces(kN,kN/m), Cases: 6 (Pe+Pw+Ps)



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According to BS8002

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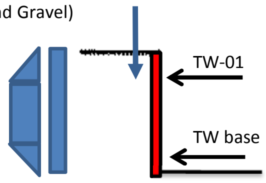
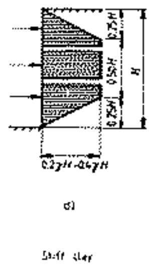
fi=	25 degree	(Sand and Gravel)	BS8002
Ko =	0.58		
a =	0.4 m		
b =	3.05 m		
c=	0.15 m		
H =	3.6 m		
0.25H =	0.90 m		

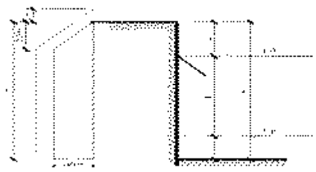




Active Pressure on Wall

Earth according to BS8002:

Pe at 0.25H =	0.4 x g x H =	28.8 kN/m/m
Pe T =	Pe0.25H x 3/4H =	77.8 kN/m

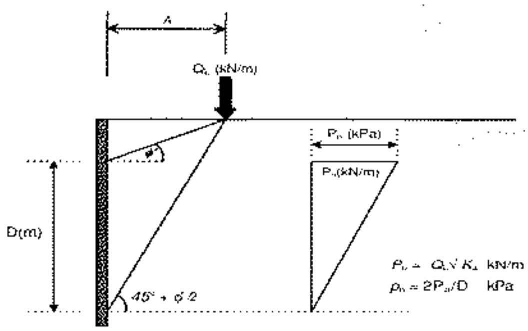


Surcharge

S =	(Party wall)	5 kN/m2
Ps =	S x k0 =	2.9 kN/m/m
Ps T =	Ps x H =	0.4 kN/m
he =		0.15 m

Surcharge from adjacent footings - from calculations refer to previous page - applied 0.85m BGL

Q =	43 kN/m
A =	0.27 m
θ =	0 degree
Ka =	1.00
D(0) = A x tanθ =	0.0 m
D(1) = A x tan(45+θ/2)	0.3 m
D = D(1)-D(0) =	0.3 m
Pn = Q x (Ka)^0.5=	43 kN/m
pn = 2Pn / D	319 kN/m/m



Water:
not considered in temporary condition

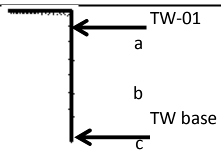
at 0	2.9 kN/m/m	
Pe+Ps at H=	31.7 kN/m/m	(short term temporary works)

Loads in line of struts:

Fa =	72 kN /m	
Fb =	43 kN /m	**Not critical**

TW-01 prop check:
Load to diagonal prop:

Lb=Max. spacing of props=	2.5 m
Axial = Fb x Lb =	**180.0** kN



Load to raking strut TW-01 at angle =	45 degree	
Axial SLS = Fsls/ cos(90-angle) =	**254.6** kN	
sideway to face = Fuls x tan(90 - angle) =	**180.0** kN	Note that stiff corner effects not taken into account <-- conservatively

Use 4M20 R-kex resin bolts (Spacing= 150mm):

Ps,1=	85.1 kN/bolt	
f= factor for spacing=	0.87	
Ps=	296.1 kN	> Sideway=180kN OK

Check Raking Props TW83:

Fuls =	381.8 kN	
Mnom = 0.05 x Fuls	19.1 kNm	

Try 152x152x37UC S355

Le =	**4** m	
Pc =	630.0 kN	(Blue book)
Mb =	77.1 kNm	

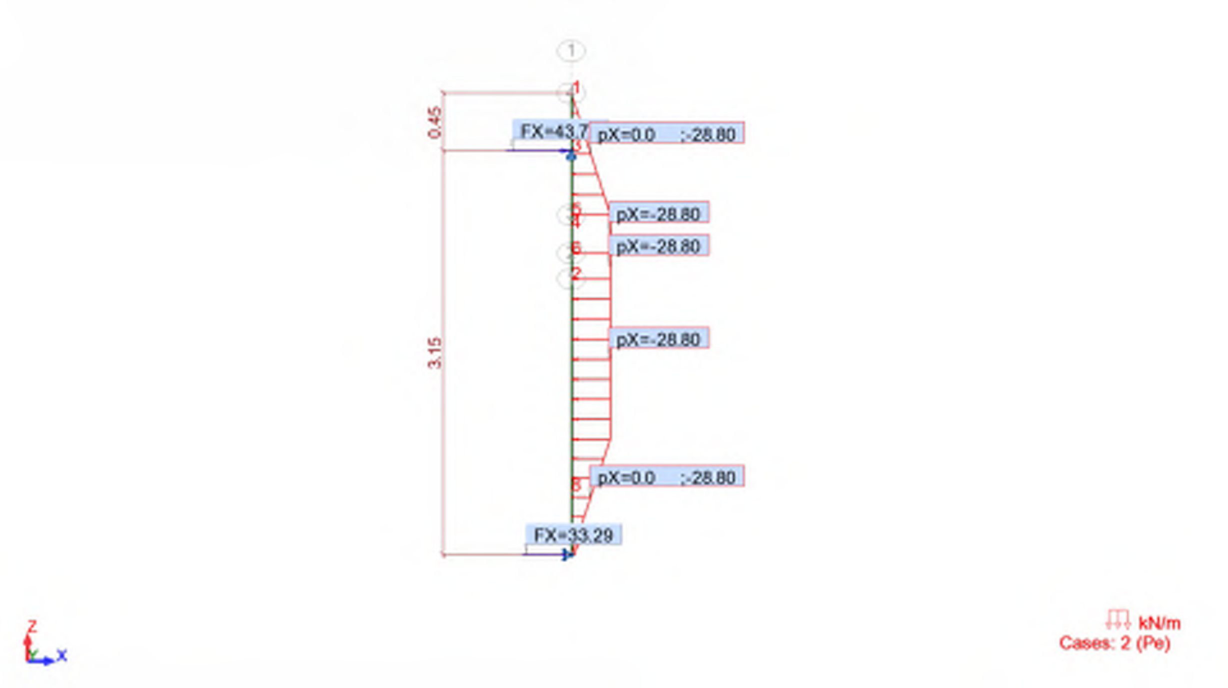
<u>Interaction check compression with bending</u>

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs	0.25				
	Myy/Mcy	0.00 =				
	F/Pcy =	0.61 =	0.85	<	1	**OK**

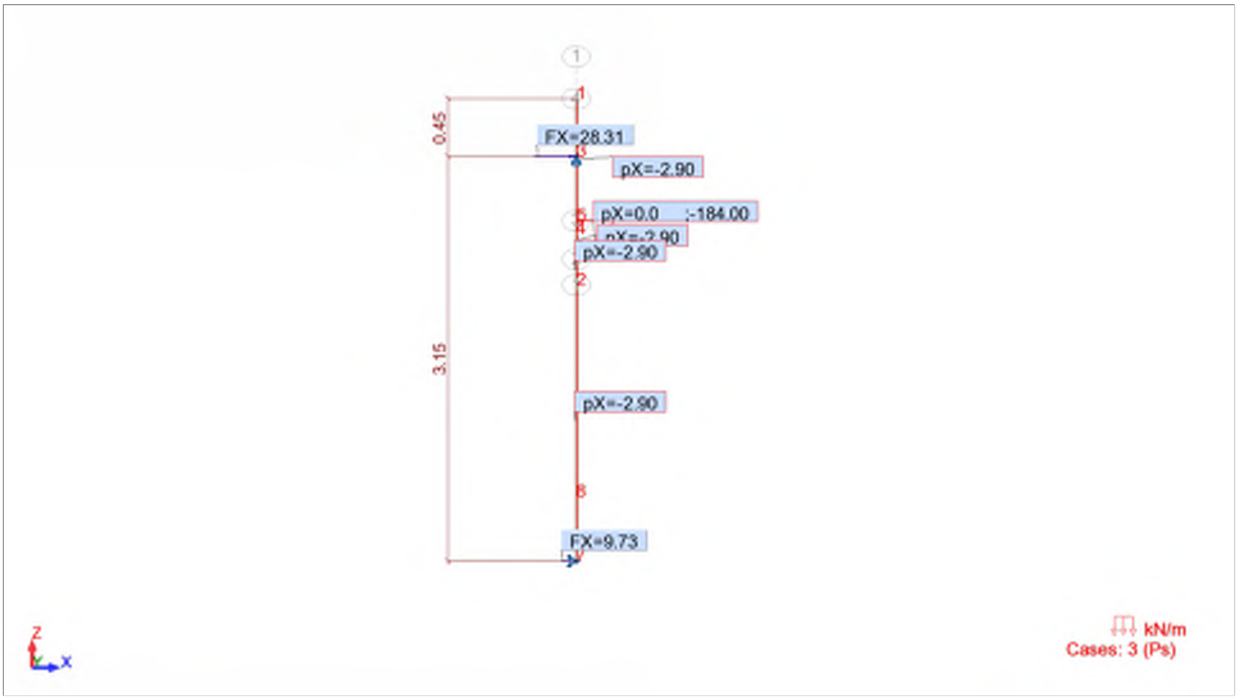
152UC37 is sufficient

Bearing on concrete face = F_v / 0.3 x 0.3 = 4.2 N/mm2 < 40N/mm2 / 1.5 = 26N/mm2

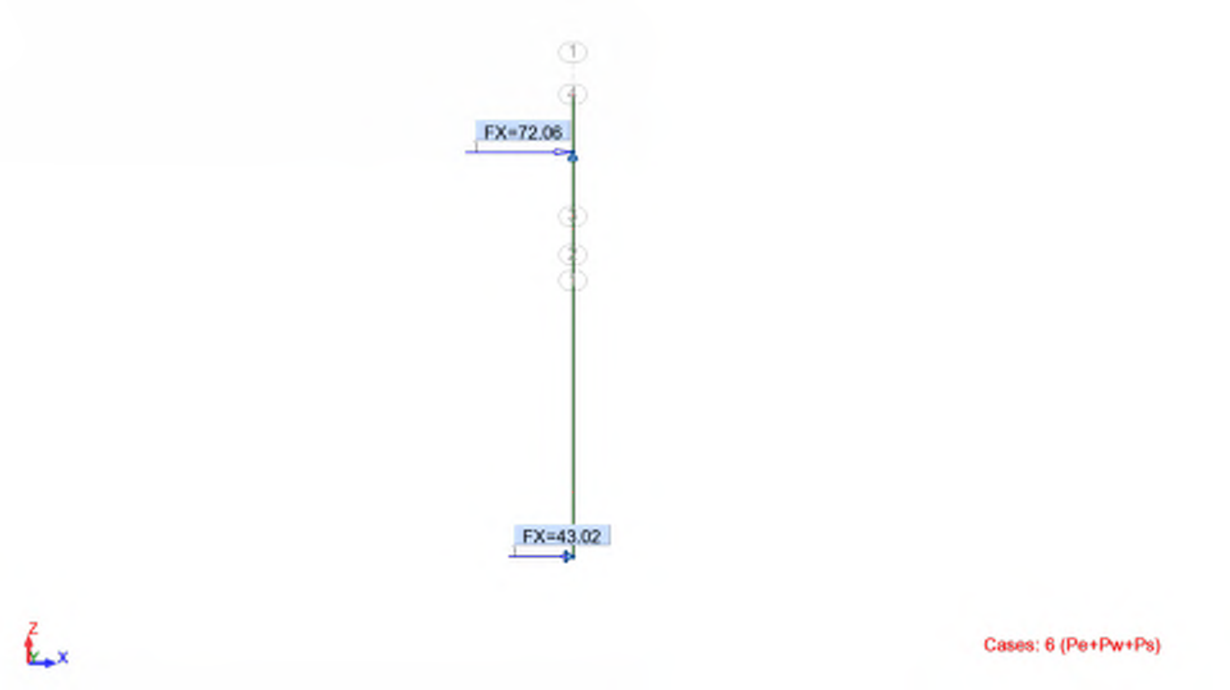
View - Reaction forces(kN,kN/m), Cases: 2 (Pe)



View - Reaction forces(kN,kN/m), Cases: 3 (Ps)



View - Reaction forces(kN,kN/m), Cases: 6 (Pe+Pw+Ps)



AXIOM STRUCTURES

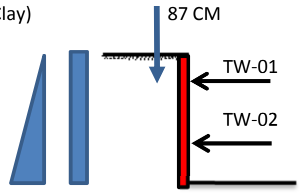
Introduction:
Permanent retaining wall is not sufficient to resist sliding and overturning without connection with the basement slab, hence propping is to be adopted until base slab is constructed.

Refer to TW-400 for sequence of works.

Temporary Struts to Retaining Wall - 85/87 Camden Mews

D I G 1

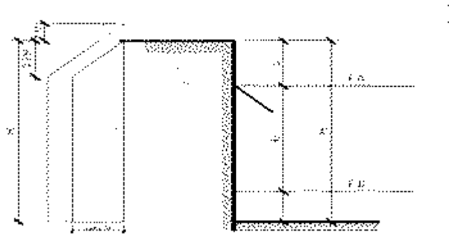
g = 20 kN/m3 (unit weight)
fi= 25 degree (Angle of internal friction)
Ko = 0.58 (Stiff Clay)
a = 0.4 m
b = 0.6 m
c= 0.45 m
H = 1.45 m
0.25H = 0.36 m



Active Pressure on Wall

Earth:
Pe at H = k0 x g x H = 16.8 kN/m/m
Pe T = PeH x H x 0.5 = 12.2 kN/m (conventional triangular shape)

Surcharge
S = (Garagel) 10 kN/m2
Ps = S x k0 = 5.8 kN/m/m
Ps T = Ps x H = 8.4 kN/m
he = 0.29 m

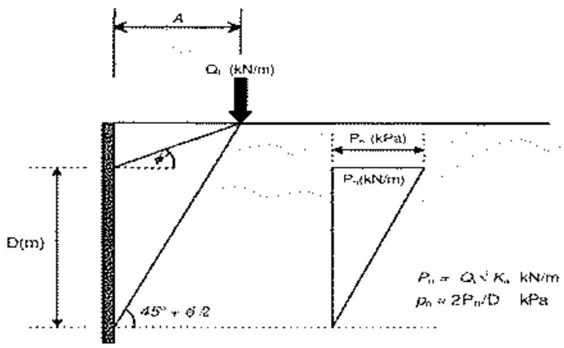


Surcharge from adjacent footings - from calculations refer to previous page - applied 0.5m BGL
Q = 17 kN/m
A = 0.1 m
θ = 25 degree
Ka = 0.41

D(0) = A x tanθ = 0.05 m
D(1) = A x tan(45+θ/2) 0.16 m

D = D(1)-D(0) = 0.1 m

Pn = Q x (Ka)^0.5= 11 kN/m
pn = 2Pn / D 196 kN/m/m



Extract from Ciria C580

Water:
not considered in temporary condition

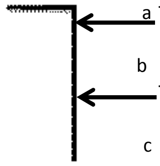
at 0 5.8 kN/m/m
Pe+Ps at H= 22.6 kN/m/m (short term temporary works)

Loads in line of struts:

Fa = **15** kN /m
Fb= **21** kN /m

AXIOM STRUCTURES

TW-01=TW-02 (critical Fb):



Load to diagonal prop:

Lb=Max. spacing of props= 2.2 m

Axial = Fb x Lb = **47.1** kN

Load to raking strut TW-01 at angle = 45 degree

Axial SLS = Fsls/ cos(90-angle) = **66.6** kN

sideway to face = Fuls x tan(90 - angle) = **47.1** kN

Note that stiff corner effects not taken into account <-- conservatively

Use 4M20 R-kex resin bolts (Spacing= 150mm):

Ps,1= 85.1 kN/bolt

f= factor for spacing= 0.87

Ps= 296.1 kN > Sideway=53.5kN OK

Check Raking Props TW-02:

Fuls = 99.9 kN

Mnom = 0.05 x Fuls 5.0 kNm

Try 152x152x37UC S355

Le = **7** m

Pc = 246.0 kN (Blue book)

Mb = 53.4 kNm

<u>Interaction check compression with bending</u>

Mxx/Mbs + Myy/Mcy + F/Pcy = Mxx/Mbs 0.09

Myy/Mcy 0.00 =

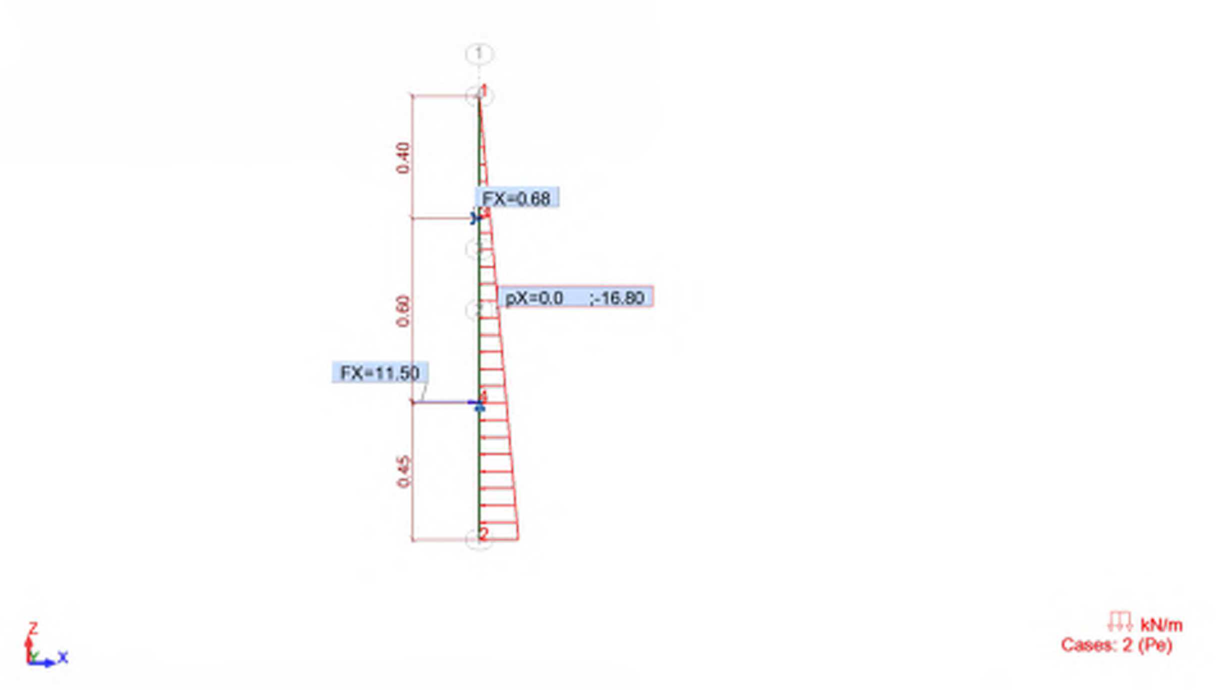
F/Pcy = 0.41 = 0.50 < 1 **OK**

152UC37 is sufficient

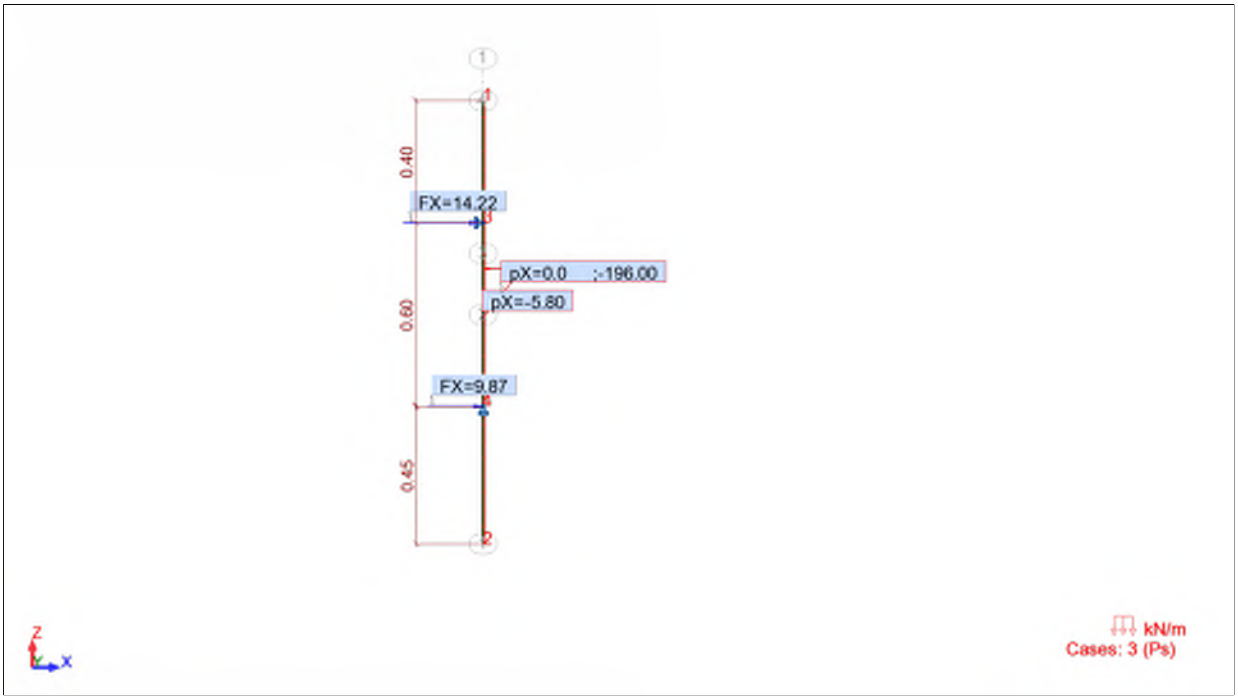
Bearing on concrete face = F_v / 0.3 x 0.3 = 1.1 N/mm2 < 40N/mm2 / 1.5 = 26N/mm2

By inspection of TW props for retaining wall to adjacent building 83 Camden Mews, calculations for loadings according to BS8002 are not critical.

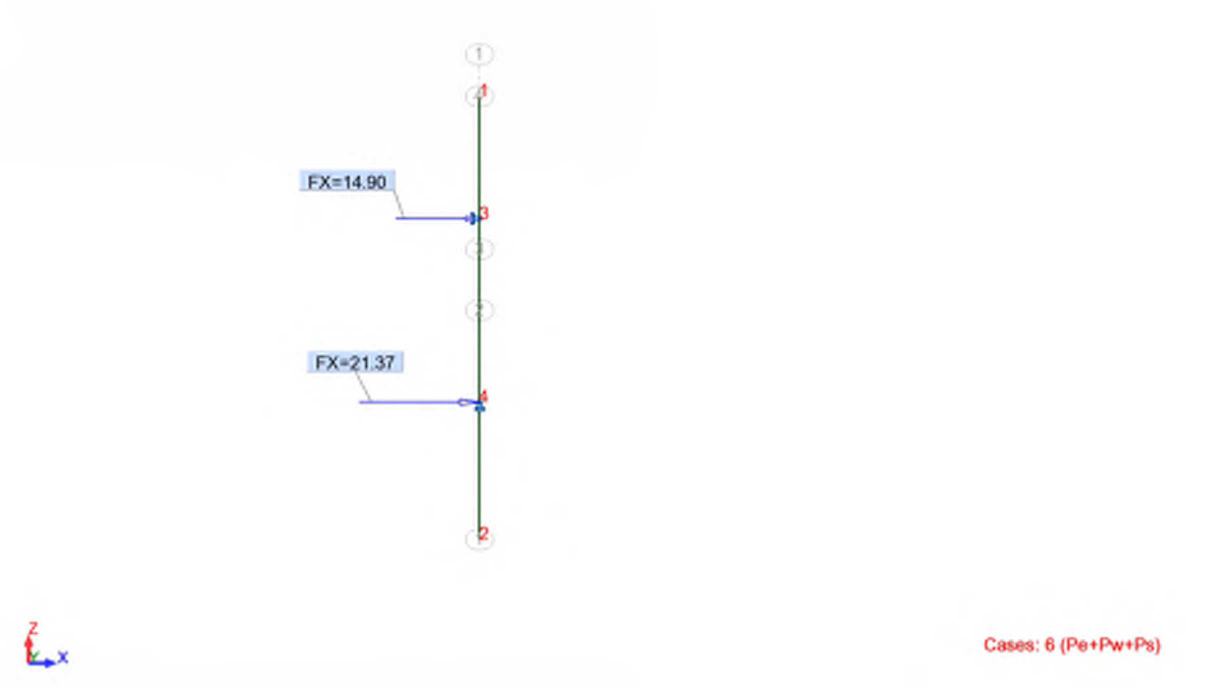
View - Reaction forces(kN,kN/m), Cases: 2 (Pe)



View - Reaction forces(kN,kN/m), Cases: 3 (Ps)



View - Reaction forces(kN,kN/m), Cases: 6 (Pe+Pw+Ps)



AXIOM STRUCTURES

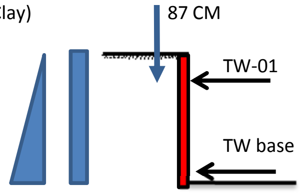
Introduction:
Permanent retaining wall is not sufficient to resist sliding and overturning without connection with the basement slab, hence propping is to be adopted until base slab is constructed.

Refer to TW-400 for sequence of works.

Temporary Struts to Retaining Wall - 85/87 Camden Mews

D I G 2

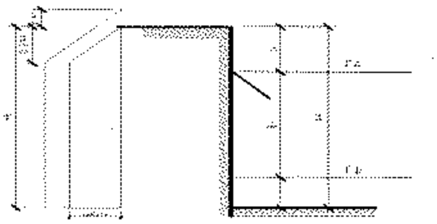
g =	20 kN/m3	(unit weight)
fi=	25 degree	(Angle of internal friction)
Ko =	0.58	(Stiff Clay)
a =	0.4 m	
b =	3.05 m	
c=	0.15 m	
H =	3.6 m	
0.25H =	0.90 m	



Active Pressure on Wall

Earth:

Pe at H =	k0 x g x H =	41.8 kN/m/m	
Pe T =	PeH x H x 0.5 =	75.2 kN/m	(conventional triangular shape)

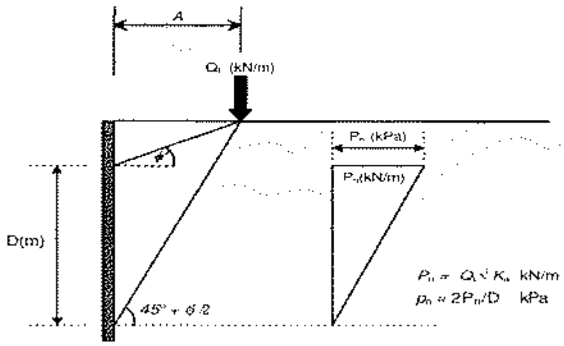


Surcharge

S =	(Garage)	10 kN/m2
Ps =	S x k0 =	5.8 kN/m/m
Ps T =	Ps x H =	20.9 kN/m
he =		0.29 m

Surcharge from adjacent footings - from calculations refer to previous page - applied 0.5m BGL

Q =	17 kN/m
A =	0.1 m
θ =	25 degree
Ka =	0.41
D(0) = A x tanθ =	0.05 m
D(1) = A x tan(45+θ/2)	0.16 m
D = D(1)-D(0) =	0.1 m
Pn = Q x (Ka)^0.5=	11 kN/m
pn = 2Pn / D	196 kN/m/m



Extract from Ciria C580

Water:
not considered in temporary condition

at 0	5.8 kN/m/m	
Pe+Ps at H=	47.6 kN/m/m	(short term temporary works)

AXIOM STRUCTURES

Loads in line of struts:

Fa = 55 kN /m
Fb= 57 kN /m — Not critical, see 83/85 Camden Mews Retaining Wall

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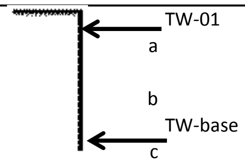
TW-01 check :



Load to diagonal prop:

Lb=Max. spacing of props= 2.2 m
Axial = Fa x Lb = **120.1** kN

Load to raking strut TW-01 at angle = 45 degree
Axial SLS = Fsls/ cos(90-angle) = **169.9** kN
sideway to face = Fuls x tan(90 - angle) = **120.1** kN — Note that stiff corner effects not taken into account <-- conservatively

Use 4M20 R-kex resin bolts (Spacing= 150mm):
Ps,1= 85.1 kN/bolt
f= factor for spacing= 0.87
Ps= 296.1 kN > Sideway=110.3kN OK

Check Raking Props TW01:

Fuls = 254.8 kN
Mnom = 0.05 x Fuls 12.7 kNm

Try 203x203x46UC S355

Le = **7** m
Pc = 503.0 kN (Blue book)
Mb = 88.3 kNm

Interaction check compression with bending

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs	0.14				
	Myy/Mcy	0.00 =				
	F/Pcy =	0.51 =	0.65	<	1	**OK**

203x203x46UC is sufficient

Bearing on concrete face = F_v / 0.3 x 0.3 = 2.8 N/mm2 < 40N/mm2 / 1.5 = 26N/mm2

AXIOM STRUCTURES

Introduction:
Permanent retaining wall is not sufficient to resist sliding and overturning without connection with the basement slab, hence propping is to be adopted until base slab is constructed.

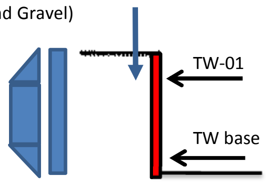
Refer to TW-400 for sequence of works.

Temporary Struts to Retaining Wall - 85/87 Camden Mews

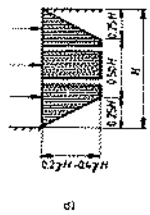
DIG 2

According to BS8002

g =	20	kN/m3	(unit weight)
fi=	25	degree	(Sand and Gravel)
Ko =	0.58		
a =	0.4	m	
b =	3.05	m	
c=	0.15	m	
H =	3.6	m	
0.25H =	0.90	m	



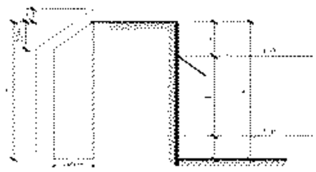
BS8002



Active Pressure on Wall

Earth according to BS8002:

Pe at 0.25H =	0.4 x g x H =	28.8	kN/m/m
Pe T =	Pe0.25H x 3/4H =	77.8	kN/m

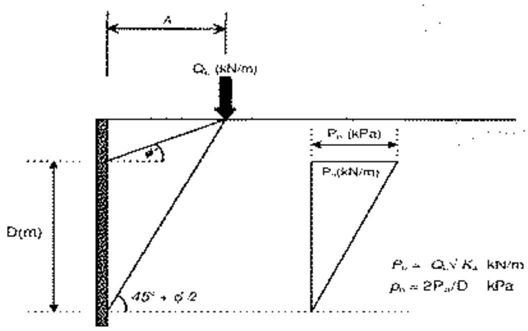


Surcharge

S =	(Party wall)	5	kN/m2
Ps =	S x k0 =	2.9	kN/m/m
Ps T =	Ps x H =	0.4	kN/m
he =		0.15	m

Surcharge from adjacent footings - from calculations refer to previous page - applied 0.5m BGL

Q =	17	kN/m
A =	0.1	m
θ =	25	degree
Ka =	0.41	
D(0) = A x tanθ =	0.05	m
D(1) = A x tan(45+θ/2)	0.16	m
D = D(1)-D(0) =	0.1	m
Pn = Q x (Ka)^0.5=	11	kN/m
pn = 2Pn / D	196	kN/m/m



Water:
not considered in temporary condition

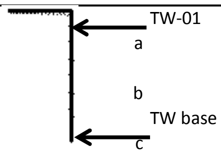
at 0	2.9	kN/m/m	
Pe+Ps at H=	31.7	kN/m/m	(short term temporary works)

Loads in line of struts:

Fa =	**70**	kN /m	
Fb =	**44**	kN /m	**Not critical**

TW-01 prop check:
Load to diagonal prop:

Lb=Max. spacing of props=	2.2	m
Axial = Fb x Lb =	**154.2**	kN



Load to raking strut TW-01 at angle =	45	degree	
Axial SLS = Fsls/ cos(90-angle) =	**218.1**	kN	
sideway to face = Fuls x tan(90 - angle) =	**154.2**	kN	Note that stiff corner effects not taken into account <-- conservatively

Use 4M20 R-kex resin bolts (Spacing= 150mm):

Ps,1=	85.1	kN/bolt	
f= factor for spacing=	0.87		
Ps=	296.1	kN	> Sideway=180kN OK

AXIOM STRUCTURES

Check Raking Props TW01:

Fuls =	327.2	kN
Mnom = 0.05 x Fuls	16.4	kNm

Try 203x203x46UC S355

Le =	**6**	m	
Pc =	503.0	kN	(Blue book)
Mb =	88.3	kNm	

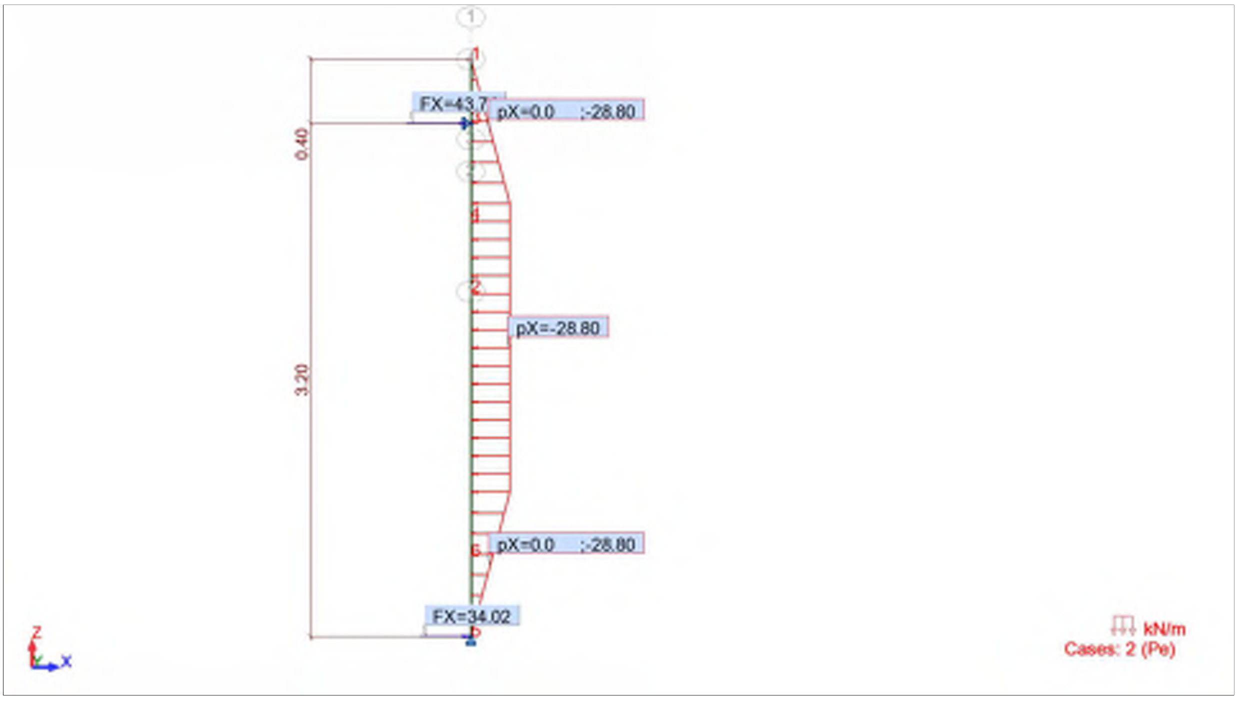
Interaction check compression with bending

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs	0.19				
	Myy/Mcy	0.00 =				
	F/Pcy =	0.65 =	0.84	<	1	**OK**

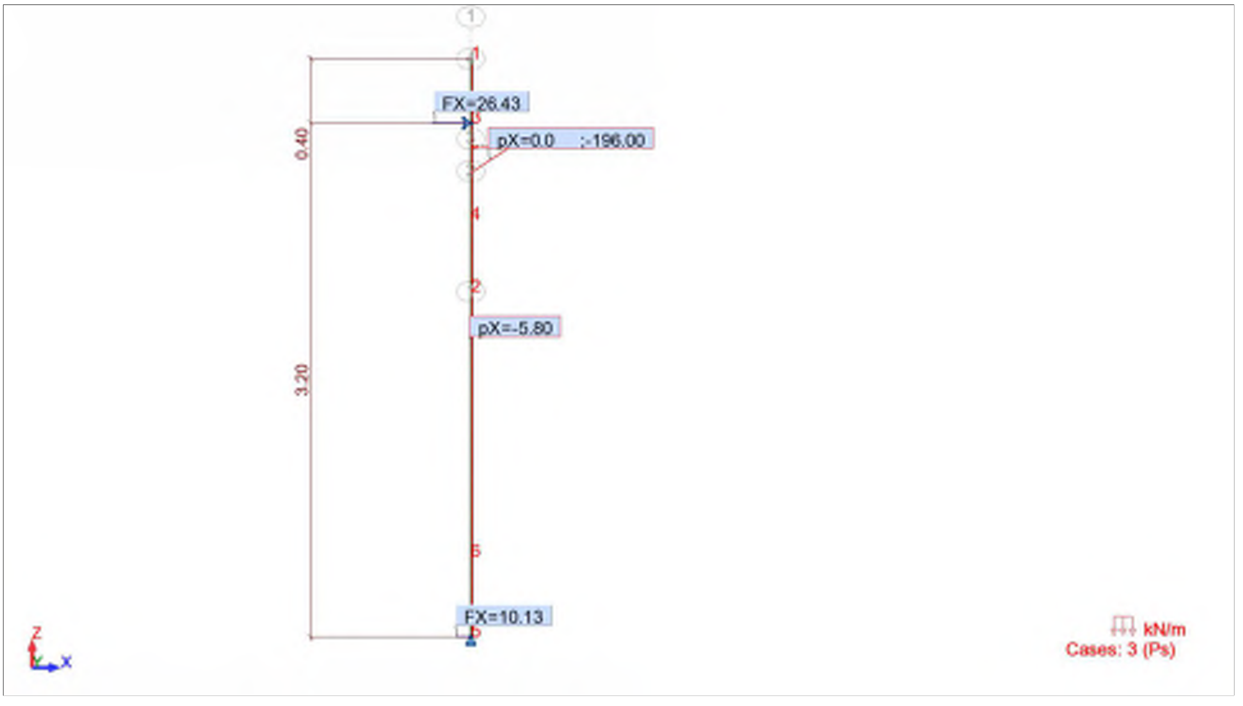
203x203x46UC is sufficient

Bearing on concrete face = F_v / 0.3 x 0.3 = 3.6 N/mm2 < 40N/mm2 / 1.5 = 26N/mm2

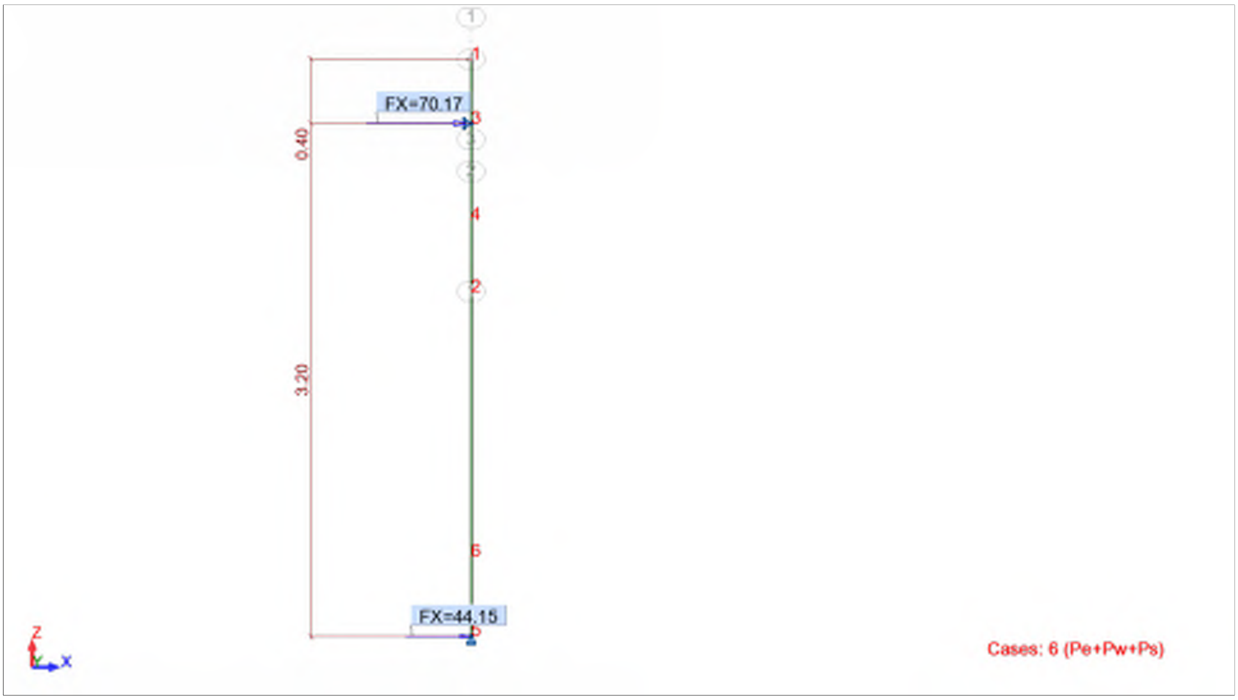
View - Reaction forces(kN,kN/m), Cases: 2 (Pe)



View - Reaction forces(kN,kN/m), Cases: 3 (Ps)



View - Reaction forces(kN,kN/m), Cases: 6 (Pe+Pw+Ps)



AXIOM STRUCTURES

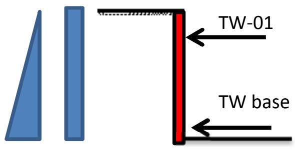
Introduction:
Permanent retaining wall is not sufficient to resist sliding and overturning without connection with the basement slab, hence propping is to be adopted until base slab is constructed.

Refer to TW-400 for sequence of works.

Temporary Struts to Retaining Wall - Façade

DIG 2

g =	20 kN/m3	(unit weight)
fi=	25 degree	(Angle of internal friction)
Ko =	0.58	(Stiff Clay)
a =	0.4 m	
b =	3.05 m	
c=	0.15 m	
H =	3.6 m	
0.25H =	0.90 m	



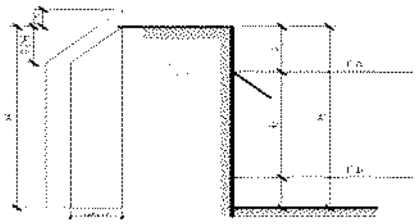
Active Pressure on Wall

Earth:

Pe at H =	k0 x g x H =	41.8 kN/m/m	
Pe T =	PeH x H x 0.5 =	75.2 kN/m	(conventional triangular shape)

Surcharge

S =	(Road)	10 kN/m2
Ps =	S x k0 =	5.8 kN/m/m
Ps T =	Ps x H =	20.9 kN/m
he =		0.29 m



Water:
not considered in temporary condition

at 0	5.8 kN/m/m	
Pe+Ps at H=	47.6 kN/m/m	(short term temporary works)

Loads in line of struts:

Fa =	40 kN /m	Not critical, see 83/85 Camden Mews Retaining Wall
Fb=	56 kN /m	

Design of Basement Toes to Work as Ground Beam between Walls:

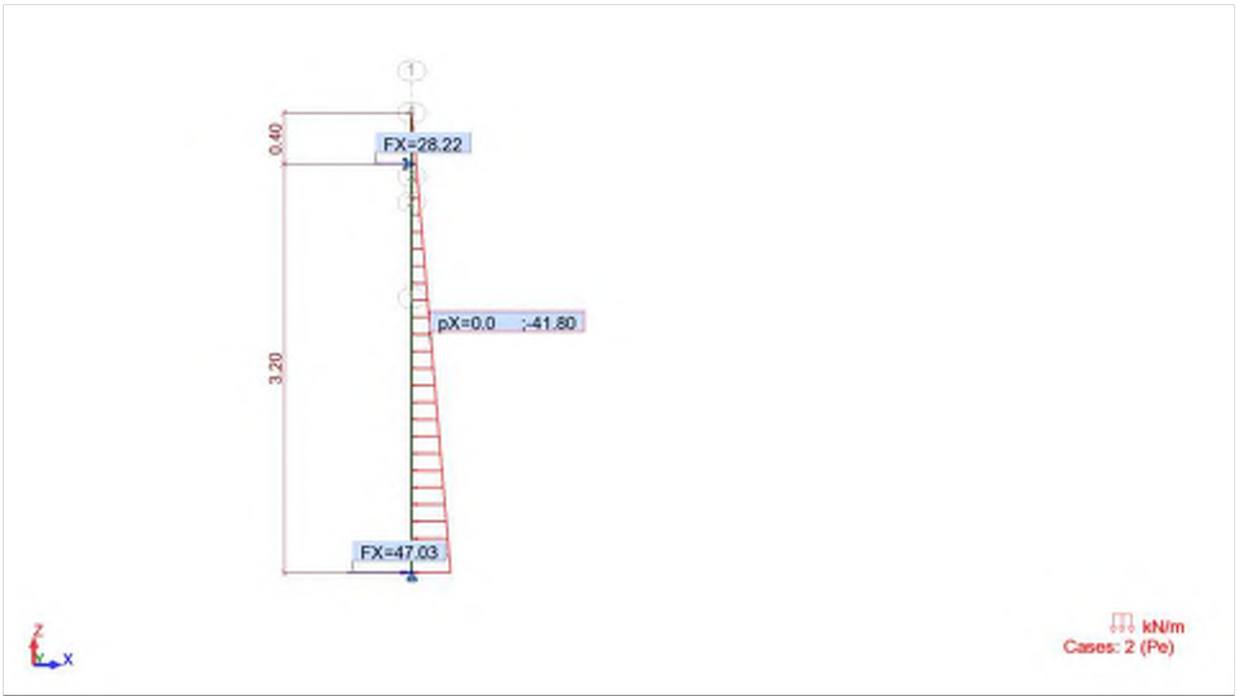
From analysis = w_sls = 56.2 kN/m

L = Span bet'n footings to act as struts = **6.6 m**
Muls = FOS x Ruls x L^2 / 8 = **459** kNm (FOS=1.5)

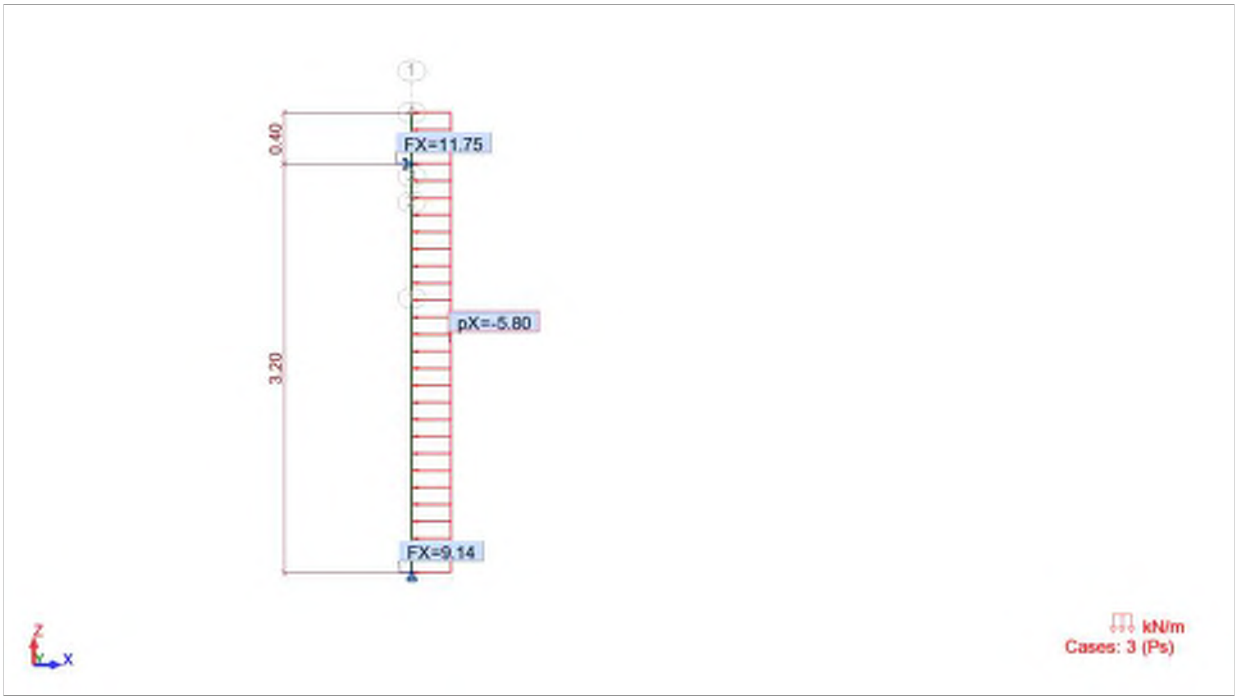
INPUT	Location	RC TW base					
	Design moment, M	459.0	kNm	fcu	35	N/mm²	gc = 1.50
	ßb	1.00		fy	500	N/mm²	gs = 1.15
	Span	4500	mm	steel class	A		
	Height, h	**1800**	**mm**	Comp cover		50	mm to main reinforcement
	Breadth, b	300	mm	Tens cover		50	mm to main reinforcement
	Tens Ø	12	mm	Side cover		50	mm to main reinforcement
	Comp Ø	12	mm	Section location			

OUTPUT
RC Ground Beam
d = 1800 - 50 - 12/2 = 1744.0 mm
(3.4.4.4) K' = 0.156 > K = 0.013 ok
(3.4.4.4) z = 1,744.0(0.5 + (0.25 - 0.013/0.893)^½) = 1,719.3 > 1,656.8 mm
(Fig 3.3) fst = 434.8 N/mm²
As = 1E6 x 459 /1,656.8 /434.8 = 637 mm²
PROVIDE 6H12 tension steel = 678 mm²
fs = 2/3 x 500 x 637 /792 = 268.3 N/mm²
Comp mod factor = 1 + 0.043 /(3 + 0.043) = 1.014 < 1.5
Tens mod factor = 0.55 + (477 - 268.3) /120 /(0.9 + 0.503) = 1.790 < 2
Permissible L/d = 26.0 x 1.014 x 1.790 = 47.192
Actual L/d = 6800 /1744.0 = 3.899 ok

View - Reaction forces(kN,kN/m), Cases: 2 (Pe)



View - Reaction forces(kN,kN/m), Cases: 3 (Ps)



View - Reaction forces(kN,kN/m), Cases: 6 (Pe+Pw+Ps)