GEOTECHNICAL DESIGN ASPECTS OF BASEMENT RETAINING WALLS

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ABSTRACT

The aim of this paper is to present general geotechnical information on basement retaining wall design to an audience of predominantly structural engineers. Practical information is presented on various topics from site investigation through numerical design and site monitoring. A limit state design approach is presented with advice on ultimate and serviceability limit states. Appropriate factors of safety are presented along with some guidance on practical issues.

1.0 INTRODUCTION

The principal aim of this paper is to broadly set out the main geotechnical design aspects in relation to basement retaining wall design.

The paper presents information and recommendations on the following:

- Sources of Geotechnical Information
- Codes of Practice
- Site Investigations
- Forms of Retaining Systems
- Groundwater
- Buildings Adjacent to Retaining Walls
- Design Approach
- Support Systems
- Monitoring

2.0 SOURCES OF GEOTECHNICAL INFORMATION

This section has been included to identify the various sources that can provide geotechnical information and geotechnical services to the structural engineering community. Depending on the simplicity or complexity of a particular scheme, some or all of these should be consulted.

2.1 Historical Maps

While not strictly geotechnical, a review of historical maps can be a good starting point when undertaking a geotechnical appraisal of a site. Historical maps can provide information on previous activities on a site such as filling (e.g. close to river walls) or buried structures (e.g. foundations of previous structures).

The National Library carries a collection of Manuscripts, Ordnance Survey and older printed maps. A summary of their map catalogue can be viewed on their website www.nli.ie under Collections/Maps. Historic Ordnance Survey maps are either in bound volumes available in the National Library's main reading room or some maps which have been treated in the Library conservation project (a list of the *"Conserved OSI Maps"* is available on the website) can be consulted by appointment only.

The Ordnance Survey of Ireland provides a digital service for viewing, downloading and ordering OSI historical maps. Their website is www.irishhistoricmaps.ie. Their archive contains:

- 6 inch mapping series (1:10,560) colour 1837-1842
- 6 inch mapping series (1:10,560) greyscale 1837-1842
- 25 inch mapping series (1:2,500) greyscale 1888-1913

There are fees charged for this service.

2.2 Geological Survey of Ireland

There is a wealth of information available from the Geological Survey of Ireland (GSI) which can be either viewed or purchased at their premises at Beggars Bush, Haddington Road Dublin 4. Their information can be summarised as follows:

- Quaternary Geological Maps – these maps describe the surface geology
- Bedrock Maps & accompanying geological books
- Groundwater & Bedrock Aquifer Maps
- Aerial Photographs
- Karst Features Maps and Information

Further information is available at www.gsi.ie

2.3 Geotechnical Field Services

This is probably the area that the structural engineering community will be most familiar with. Geotechnical field services are generally provided by site investigation contractors, geophysical companies and well drilling specialists.These are generally private individuals or companies and they will prepare either purely factual reports or an interpretation of the data.

2.4 Geotechnical Consultants

Geotechnical consultants, which can be practising individuals, consulting firms or groups within multi-disciplinary consulting practices can provide specialist geotechnical advice and design. In the context of basements retaining walls a geotechnical consultant might:

- Provide advice on appropriate site investigation (scope, supervision and interpretation)
- Prepare detailed specifications for inclusion with tender documents
- Provide schematic and detailed designs of embedded retaining walls
- Design suitable tie-back anchors or other support systems
- Specify allowable and predicted wall movements
- Calculate expected settlements of retained structures
- Design appropriate groundwater cut-off or groundwater lowering schemes
- Provide advice on appropriate construction methods
- Provide advice at pre-award technical interviews
- Classify the material to be excavated in terms of any potential contamination
- Provide a design for the foundations of the main structure
- Provide advice on suitable movement monitoring systems

3.0 CODES OF PRACTICE

The following is a list of the main codes of practice in relation to basement retaining wall design:

- *BS5930: 1999, Site Investigations (will be superseded by EC7).*

- *Eurocode 7: Geotechnical Design.*

- *BS8002: 1994, Earth Retaining Structures (will be superseded by EC7).*

- *BS8102: 1990, Code of Practice for Protection of Structures against Water from the Ground.*

- *CIRIA C580, 2003, Embedded Retaining Walls Guidance for Economic Design*

- *Institution of Civil Engineers, 1996, Specification for Piling and Embedded Retaining Walls.*

- *CIRIA C515: 2000, Groundwater Control Design & Practice*

4.0 SITE INVESTIGATION

It is assumed that the reader is familiar with the generic methods of site investigation routinely carried out in Ireland. These are generally trial pits, boreholes, rock-cores, rotary-percussive drilling, dynamic probing, cone penetration testing, geophysical surveying and geotechnical laboratory testing. However in the context of deep basement there are a few specific points which are worth discussing.

4.1 Permeability

An accurate determination of the soils co-efficient of permeability value "k" can have a major influence on the design of a deep basement system and on the success or failure of that system once implemented. The Irish boulder clays are low permeability materials and can generally be considered to provide a groundwater cut off layer. However when gravels, sands or silts are present the k value of each material will determine the quantity and rate that groundwater will enter the excavation. This will in turn determine the extent to which the retaining wall will be required to reduce or cut-off groundwater.

There are a number of methods to determine the k value such as:

- Rising, falling and constant head tests in boreholes and in the laboratory
- Groundwater pumping tests on site
- Particle Size Distribution (PSD) analyses and triaxial cell tests in the laboratory

It is advisable to get specialist advice on the particular test suitable for a particular site and geology. It is also advisable to carry out a number of different types of tests and to compare the results obtained.

4.2 Stratigraphy

While it may seem an obvious comment, the accurate determination of the stratigraphy to an appropriate depth below the basement level is essential. Embedded retaining walls may penetrate to double the basement depth or greater. Low-strength or high permeability layers may exist at depths below the basement level. The presence of bedrock may restrict the use of some common basement retention systems such as sheet piling or CFA piling. Therefore sufficient stratigraphical information to sufficient depth is a vital component of a successful basement retaining wall design.

4.3 Stiffness

The stiffness of the soil plays a major role in the deflections realised in basement retaining walls. The soils in Dublin and across the country present particular challenges when attempting to accurately determine their stiffness. The boulder content of the glacial soils can preclude either Cone Penetration Testing (CPT) in gravels or standard "U100" sampling techniques in boulder clays.

For boulder clay deposits it may be worth investing in high quality "Geobor-S" borehole drilling to obtain high quality relatively undisturbed samples. These can then be tested in specialised triaxial cells to assess the stiffness of the material.

The stiffness of gravel layers in Ireland is normally assessed from the Standard Penetration Test (SPT). Groundwater in boreholes can sometimes cause the gravel to "boil" and thus loosen and yield a low SPT value. Measures should therefore be taken to avoid or reduce boiling. This can be achieved by keeping the borehole casing topped up with water to equalise the groundwater pressures.

4.4 Boulders

The inclusion of the term "boulders" on boreholes logs is common in Ireland. This can be for example:

" ……sandy Gravel with many cobbles and boulders"
"…….chiselling boulders for 2hrs"
"…….refusal, presumed rock or boulder"

BS5930 *Site Investigations* defines a boulder as being greater than 200mm with no upper limit.

The quantities of boulders in a soil stratum are described as:

- "…..with occasional boulders" (up to 5% boulders)
- "…..with some boulders" (5% to 20% boulders)
- "…..with many boulders" (20% to 50% boulders)

Boulders up to 1m in diameter are not unusual in some soils types. It is important that the size and quantity of the boulder content is adequately reflected in the site investigation report as this can have a significant bearing on the ability of various piling plant to penetrate to the required depth.

Figure 1 below presents an example of a boulder.

Figure 1 Boulder from site in Limerick

4.5 Bedrock

The presence or possible presence of bedrock needs to be investigated as part of a site investigation. If the presence of bedrock is confirmed then detailed consideration will need to be given to:

- Type of rock
- Strength
- Degree of fracturing/weathering
- Permeability
- Angle of bedding planes
- Faults, folds etc

Many “standard” site investigations can misrepresent bedrock because incorrect methods were specified or the methods employed were not altered as the site investigation proceeded.

Many site investigations can be based mainly on Shell and Auger borehole drilling. This technique has only limited ability to penetrate boulders and cannot penetrate much into bedrock. Boreholes sometimes terminate on “Presumed Rock”. In many instances these boreholes will actually have terminated on a boulder and rock head can be many metres below. Therefore this term should not be used to identify rock level.

Bedrock can often comprise of an upper weathered/fractured zone and a lower more intact deposit. Depending on the strength of the rock and the degree of weathering/fracturing, various piling methods will have different capabilities to penetrate rock and the following points can be used as a guide:

- Sheet piling can generally not penetrate into bedrock.
- CFA piling can penetrate weathered rock to a limited depth and generally cannot penetrate intact rock.
- Bored piling using appropriate rock augers or core barrels can penetrate both weathered and intact rock. However as the rock strength increases the rate of penetration will decrease.
- Down-The-Hole-Hammer (DTHH) techniques can readily drill intact rock to provide rock-socket piles.

The requirement to penetrate both weathered and intact rock can arise from the following:

- The basement level may be below rock head level
- A rock socket may be required to provide sufficient pile toe restraint.
- A groundwater cut-off may be required through the weathered and into the intact rock.
- The basement wall may need to accommodate significant vertical loads. This can sometimes require extending piles to/into rock.

The most common ways to investigate bedrock, each with advantages and disadvantages, are

- Rotary coring
- Open hole/DTHH techniques
- Geophysical techniques

5.0 FORMS OF RETAINING SYSTEMS

There are a number of commonly used retaining systems in Ireland for basement retaining walls and these are:

- Sheet Piling
- Contiguous Piling
- Secant Piling
- King Post Walls
- Diaphragm Walls

and each is discussed below. These systems can either operate in cantilever or propped mode.

5.1 Sheet Piling

Steel sheet piling is a well-known and reasonably well-understood method of retaining wall construction. Steel sheets of various profiles and lengths are either vibrated or impacted in order to gain the required embedded depth.

The joints between sheet piles can be relatively, but generally not totally, waterproof. The inclusion of a water proofing sealant can reduce the quantity of groundwater passing through the joints in the temporary condition while the joints can be welded above formation level in the permanent condition.

Sheet piles can penetrate most soil types but generally cannot penetrate bedrock unless pre-drilling is undertaken. While impact hammers can generate significant noise and vibration, modern high frequency vibrating hammers have been used successfully in close proximity to existing structures. Sheet piling is used routinely as a temporary retaining system where the piles are removed when the permanent basement structure is complete. Permanent unlined sheet pile walls have also been used on a number of projects in Ireland. Sheet piles can also accommodate permanent axial loads.

Figure 2 below presents an example of a sheet pile wall.

Figure 2 Sheet Pile Wall, Spencer Dock, Dublin (Murphy International Ltd)

5.2 Contiguous Piling

Contiguous piling generally involves the installation of in-situ concrete piles at distances of 1.5 to 3.0 times pile diameter apart. This type of retaining system is only suitable in particular soil types where soil loss and groundwater ingress can be prevented or controlled. For example the method may be suitable for a site with clay from ground level to formation level but would not be suitable where a water-bearing gravel is present. It is important to consider the possible presence of sand and gravel lenses and layers in clay deposits and to have contingency plans should these become apparent at construction stage.

Contiguous piling can be installed in most soil and rock types, subject to suitable installation plant and equipment. The piling can be either temporary or can be incorporated into the final structure to provide retention and vertical load carrying capabilities.

Figure 3 below presents an example of a contiguous pile wall.

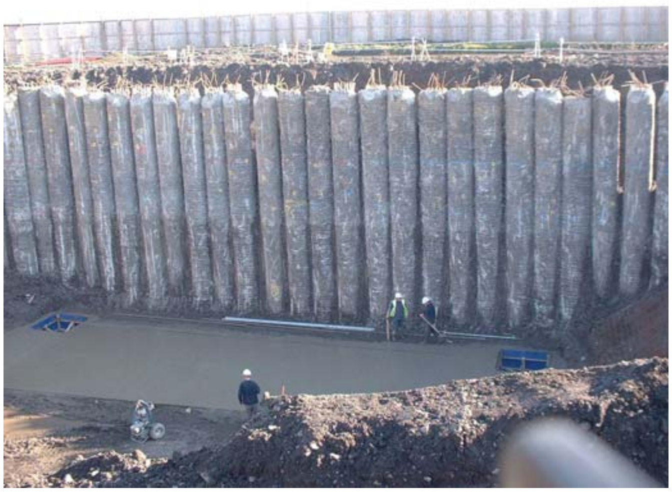

Figure 3 Contiguous Pile Wall, Dublin Port Tunnel (PJ Edwards & Co.)

5.3 Secant Piling

Secant piling involves the installation of concrete piles which interlock and therefore provide a continuous concrete wall. The piles are installed in a hard-soft (more commonly) or hard-hard arrangement. The soft piles are installed initially using a soft concrete mix (e.g. C7N) and these piles are unreinforced. As the hard piles are installed they secant into the soft piles on either side. The hard piles are constructed using structural concrete (e.g. C35N) and are reinforced. The hard piles therefore provide all the structural strength and stiffness. Subject to issues of tolerance, quality and stratigraphy a secant pile wall can provide a near total ground water cut-off in the temporary condition.

In the permanent condition a secant pile wall should be considered to be “Type A” in accordance with BS8102. That is to say the structure itself does not prevent water ingress. Protection is dependent on a total water or water and vapour barrier system applied internally or externally.

As with contiguous piles, a secant pile wall can either be temporary or incorporated into the permanent structure.

An example of a secant pile wall is presented in Figure 4.

Figure 4 Secant Pile Wall, Dunnes Stores, Georges St., Dublin (Murphy International Ltd)

5.4 King Post Walls

King Post Walls can provide an economic alternative when constructed in suitable ground conditions. Essentially they involve the installation of a structural post (usually a UC or UB section) at various distances apart (generally 2m to 4m) and infilling the space between the posts with horizontally spanning material. This material can be timber, steel or more usually in-situ or precast concrete. Infilling between the King Posts is normally carried out in a top-down manner. Therefore the retained soil is required to stand partially unsupported while the infill panel is being constructed.

King Posts can either be installed by drilling a hole with a piling rig (generally recommended) or by excavating "slots" through a battered excavation. King post walls provide only minimal waterproofing above formation level and no groundwater cut-off below formation level. Ground movements with king post walls can be greater than with some of the systems mentioned above and therefore may not be suitable close to settlement sensitive structures.

5.5 Diaphragm Walls

Diaphragm walling involves the excavation of a trench with crane suspended grabs or milling cutters. The excavation takes place while the trench is filled with a support fluid, normally bentoite slurry. The diaphragm wall is formed by filling the excavated trench with concrete which displaces the support fluid.

5.6 Piling Tolerances

It is advisable to take account of piling tolerances when designing retaining walls. The ICE *Specification for Piling and Embedded Retaining Walls* states a normal piling tolerance of 25mm in plan position when a guide wall is constructed and a vertical tolerance of 1:100.

Therefore if we use an example of a 7m excavation then a single pile can deviate from its design position by 95mm in any direction at formation level.

Adjacent piles can therefore separate by twice this distance (190mm in this example) from their design position. Normal secant piling overlap is typically 100mm to 150mm. Therefore a void can occur between piles which can have implications for groundwater ingress. These voids can be filled above formation level but are obviously very difficult to locate and fill below formation level.

The issue of piling tolerance can also cause a reduction in basement dimensions at formation level.

6.0 GROUNDWATER

Groundwater considerations will usually be uppermost in the designers mind when undertaking basement projects. Groundwater can potentially flow through retaining walls, under retaining walls and up through the basement floor. In some cases it can cause instability of the retaining wall or basement floor.

It is imperative at the site investigation stage to identify the level or levels of groundwater, any tidal influences, the permeabilities of the various soil and rock horizons and any local influences.

Groundwater can have more than one level. There can be a perched water table in an upper stratum, an artesian head in a lower stratum or a groundwater gradient across the site. Nearby activities can also have an effect on groundwater levels.

Basement retaining walls can be designed to provide a total or near-total groundwater cut-off or the basement can be temporarily dewatered by pumping. If pumping is chosen then there will be a requirement to discharge abstracted groundwater. There needs to be sufficient drainage infrastructure in place and a discharge license will be required from the local authority.

Local authorities can sometimes limit discharge volumes to relatively low quantities making the pumping option unworkable. It is worth remembering that moderate variations in the co-efficient of permeability value “k” can have a significant effect on the volumes of groundwater to be abstracted.

Abstraction of groundwater and the lowering of groundwater levels can have an effect on surrounding buildings. This can be due to ground loss associated with the removal of fines, increases in soil stresses due to groundwater lowering or the shrinkage of clay or silt layers due to a reduction in moisture content.

Where contiguous piled walls are being considered, even minor seepages of groundwater can sometimes cause washout between the piles and behind the wall. Provision should always be made to undertake remedial measures such as providing a concrete infill between the piles.

7.0 RETAINED STRUCTURES

It is common practice in Ireland to excavate single, double or triple level basements immediately adjacent to large settlement-sensitive structures.

It is therefore important to determine the following information in relation to retained buildings:

- The distance of the retained structure from the basement
- The foundation details of the retained structure including size, type, condition and level
- The imposed dead and live load
- The details of the superstructure including material, form, grid spacing, condition
- The geometry of the structure including any overhangs which may effect piling plant
- The sensitivity of the occupants to noise and vibration.

The above information will facilitate an assessment of the allowable retaining wall deflections, ground movements and building settlements as these issues will have a significant influence of the form of basement retaining structure chosen and the temporary support to be provided.

8.0 DESIGN APPROACH

As with the design of any structure the design approach for basement retaining structures commences with the overall design philosophy and proceeds through to detailed numerical design. Assuming that the site investigation has been completed the initial considerations will include:

- Ground conditions
- Depth of required excavation
- Groundwater lowering/cut-off options
- Retained structures and services
- Allowable movements
- Space limitations/available plant
- Temporary support options
- Budget and programme
- Temporary/permanent retention
- Vertical load carrying requirement
- Recovery of material (e.g. sheet piles)
- Working space (single vs. double sided shutters for permanent RC walls)

Once the broad requirements, based on the above criteria, have been established then the design can proceed to the numerical design stage.

Some of the text below has been abstracted from CIRIA C580.

8.1 Limit States

Simpson and Driscoll (1998) define limit state design as a procedure in which attention is concentrated on avoidance of limit states, i.e. states beyond which the retaining wall no longer satisfies the design performance requirements. This relates to the possibility of damage, economic loss or unsafe situations.

Retaining walls should be designed for Ultimate Limit States (ULS) and Serviceability Limit States (SLS).

8.2 Ultimate Limit States

Ultimate limit states are those associated with collapse or with other similar forms of structural failure. They are concerned with the safety of people and the safety of the structure.

The following should be considered:

- Loss of equilibrium of the structure
- Failure by rotation or translation of the wall
- Failure by lack of vertical equilibrium of the wall
- Failure of a structural element such as wall, anchor, prop or wailing beam
- Movements of the retaining structure that may cause collapse of the structure, nearby structures or services which rely upon it
- Failure caused by fatigue or other time-dependent effects.

The ULS design involves carrying out a Limit Equilibrium Analysis. Appropriate Factors of Safety (discussed further in Section 8.4) are applied and earth pressure diagrams are constructed, see Figure 5, using classical earth pressure theory in which full "Active" and "Passive" conditions are assumed. Computer programmes allow rapid analyses to be carried out and readily allow for changes to geometry, stratigraphy, soil parameters etc.

A limit equilibrium analysis involves applying factors of safety which reduce the soil strength and increase the imposed surcharge load on the wall. The required depth of embedment is then calculated to provide a factor of unity on rotational or translational failures.

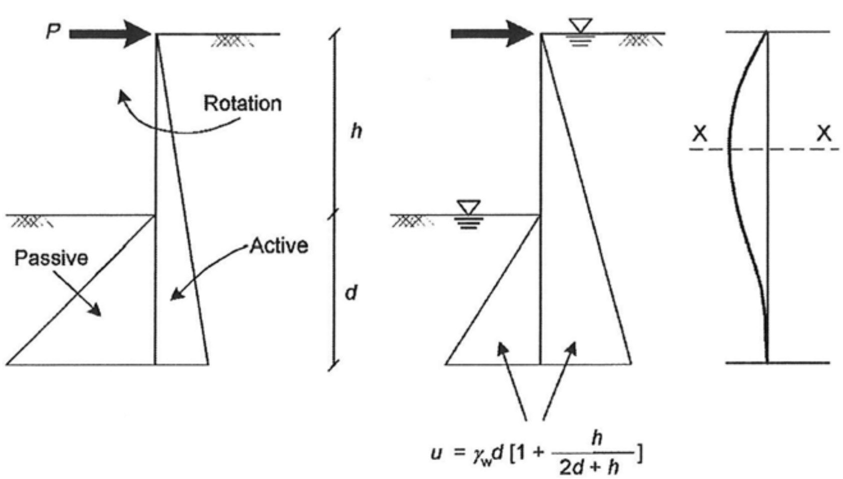

Figure 5 ULS Earth Pressure Diagram, Propped Wall including Bending Moment Profile

This type of analysis will produce a set of bending moments and shear forces in the wall along with calculated propping forces. These forces may or may not be the actual design forces as discussed further in Section 8.5.

8.3 Serviceability Limit States

Serviceability limit states correspond to conditions beyond which specific service performance requirements are not longer met, for example pre-defined limits on the amounts of wall deflections.

The permissible movements specified in the design should take into account the tolerance of nearby structures and services to displacement.

It is worth remembering that the overall stability analysis was carried out using full “Active” and “Passive” soil conditions with factored soil and surcharge parameters.

The next stage of the analysis procedure is to calculate the actual wall, ground and building movements along with the forces in the wall in the “serviceability condition”. Soil generally does not exist in the ground, pre-development, at either its active or passive limit and generally will not fully reach its passive limit during the works as appropriate factors of safety will have been employed. Importantly, the soil may not reach its active limit either and therefore the soil load on the wall in the SLS condition may be greater than in the ULS condition.

As stated above bending moments, shear forces and prop loads were calculated based on the soil model presented in the ULS analysis. However if the soil does not fully reach its passive (or active) limit then this model will not be accurate and an alternative model or models are required.

By way of background explanation, before construction activities occur soil exists in-situ at what is known as “At-rest” or “Ko” (co-efficient of earth pressure at rest) conditions, see Figure 6. As soil cannot strain laterally as it is being compressed during its formation, lateral stresses are generated. This subject has generated a vast amount of research but for the purpose of this paper a simple example will be used.

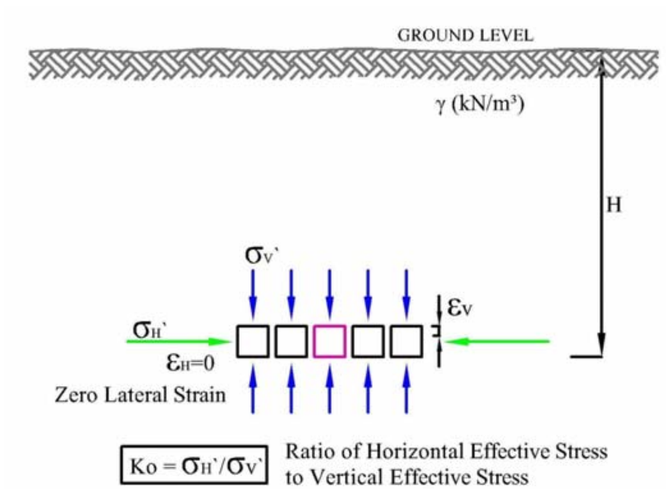

Figure 6 Earth Pressure at Rest Ko

For normally consolidated clays Ko can be determined by the formula:

- Ko = 1-Sinϕ'

For many glacial soils the value of Ko is much greater than 1-Sinϕ' but for the purposes of this paper this example will be used.

As an excavation proceeds, see Figures 7, the earth pressures move from the Ko condition towards either the Ka or Kp condition. The magnitude and distribution of the shift and the "serviceability" pressures on the wall are based on numerous factors, however the stiffness of the various soil strata have a significant influence.

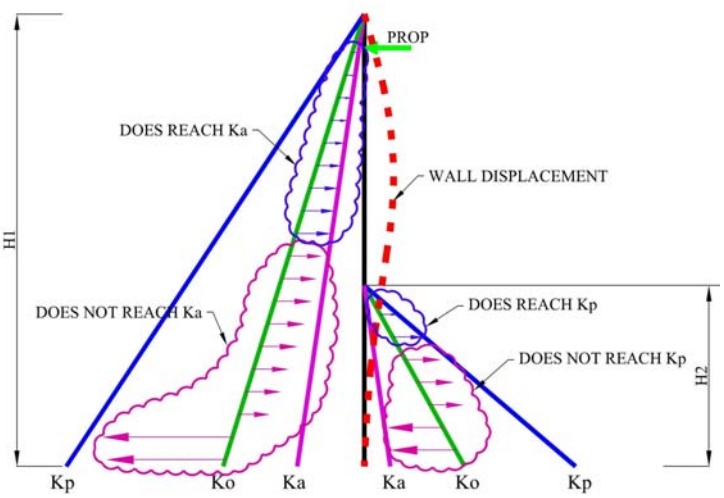

Figure 7 Notional SLS Earth Pressures

It is therefore apparent that the SLS pressures are different in magnitude and shape to those in the ULS model. Therefore the ULS model cannot be used to accurately predict the displacement of the wall.

Modern SLS analyses are normally carried out using finite element (FE) or finite difference (FD) techniques and there are many computer packages available to carry out these. An example from the PLAXIS package is presented in Figure 8 below.

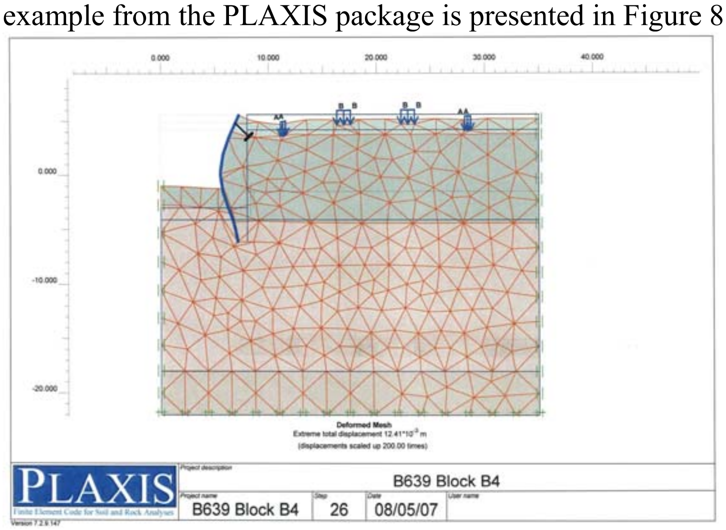

Figure 8 Example of PLAXIS Output

The serviceability analyses are carried out using unfactored soil parameters with no over-dig allowance.

The bending moments, shear forces and prop forces are compared to those calculated in the ULS calculation as discussed in Section 8.5.

8.4 Soil Parameters & Factors of Safety

The selection of appropriate soil parameters and factors of safety for both the ULS and SLS calculations is obviously critical to overall design. In fact the selection of soil parameters and factors of safety are intrinsically linked.

CIRIA C580 outlines three "Design Approaches" and details appropriate factors of safety for each. The design approaches involve selecting soil parameters, groundwater pressures, loads and geometry as follows:

- **A – Moderately Conservative:** a cautious estimate of the relevant values

- **B – Worst Credible:** the worst values that the designer reasonable believes might occur, a value that is very unlikely. This approach is not appropriate for SLS calculations.

- **C – Most Probable:** values that have a 50% probability of being exceeded. This approach should only be used in conjunction with an "Observational Method" where the design can be reassessed based on observed deflections etc of the wall.

Soil parameters are factored as follows:

$$\tan \phi'_d = \frac{\tan \phi'}{F_{\phi'}}$$

$$c'_d = \frac{c'}{F_{c'}}$$

$$C_{ud} = \frac{C_u}{F_{cu}}$$

where

ϕ', c', C_u = soil parameters based on Design Approach A, B or C

$F_{\phi'}$, $F_{c'}$, F_{cu} = factors of safety on friction angle, cohesion intercept and undrained shear strength

ϕ'_d, c'_d, C_{ud} = design soil parameters

Appropriate Factors of Safety are detailed in Table 1 below.

Design Approach	Ultimate Limit States			Serviceability Limit States		
	Effective Stress ϕ', c'		Total Stress C_u	Effective Stress ϕ', c'		Total Stress C_u
	$F_{c'}$	$F_{\phi'}$	F_{cu}	$F_{c'}$	$F_{\phi'}$	F_{cu}
A: Moderately Conservative	1.25	1.25	1.5	1.0	1.0	1.0
B: Worst Credible	1.0	1.0	1.0	See note (1)	See Note (1)	See note (1)
C: Most Probable	1.25	1.25	1.5	1.0	1.0	1.0

Table 1: F_s factors appropriate for use in design calculations

Notes:
(1): Not appropriate for SLS calculations

The author generally applies Design Approach A.

In relation to imposed surcharges the factors in Table 2 should be applied for ULS calculations.

Permanent/Variable	Condition	Value
Permanent	Unfavourable	1.1
Permanent	Favourable	0.9
Variable	Unfavourable	1.5
Variable	Favourable	0

Table 2: Factors on Surcharges

8.5 Design Forces

As described above, sets of bending moments, shear forces and prop forces will be determined from both the ULS and SLS calculations. The ultimate forces for which the wall should be structurally designed are:

- BM & SF: the greater of the ULS or 1.35*SLS values

- Prop Forces: the greater of 1.35*SLS or 1.85*ULS. This is because ULS calculations can underestimate the actual prop force generated to keep deflections to acceptable limits.

An additional prop load due to potential temperature effects should also be included.

8.6 Overdig Allowance

An allowance for overdig or unplanned excavations needs to be taken into account for ULS calculations. This allowance should not be included in SLS calculations.

Foreseeable excavations such as service or drainage trenches in front of a retaining wall are planned excavations and the overdig allowance is for additional unforeseen events.

The allowance should be the lesser of:

- 0.5m
- 10% of cantilever height
- 10% of retained height below lowest support

8.7 Wall Friction

Advantage can be taken of the friction, which must be generated between the soil and retaining wall if wall failure were to occur. The value of wall friction is based on the soil and wall type and typical values are:

- Active Zone: $\delta = 0.67\ \phi'$
- Passive Zone: $\delta = 0.5\ \phi'$

Wall friction therefore can be employed to effectively reduce the "Ka" value (and thus the load on the wall) and to increase the "Kp" value (and thus the implied strength of the soil in front of the wall). The method generally employed by the author is that of Caquot and Kerisel and presented in Figure 9 below. In the example below a granular material with a $\phi' = 35^{o}$ has its "Ka" value reduced from the value of 0.27 with no wall friction to 0.24 and its "Kp" value increased from 3.7 to 6.0.

This example assumes no axial load on the wall.

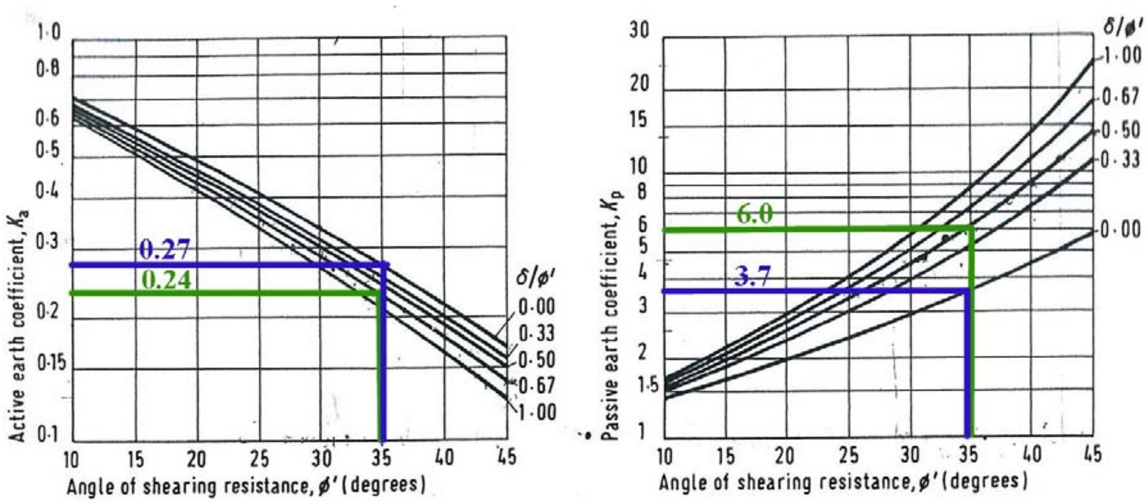

Figure 9 Effect of Wall Friction on Ka and Kp

8.8 Effective Stress

It is important to understand that earth pressure co-efficients (Ka, Kp, Ko) are Effective Stress parameters and must take account of the prevailing pore water pressures. Generally (but not always) this involves using the buoyant weight of the soil below the groundwater level and this actually reduces the "soil" load on a retaining wall. However the total load must also include the groundwater pressure and therefore the total load on the retaining wall will generally be greater where groundwater is present.

8.9 Undrained Analysis

It is valuable to be able to take account of undrained soil conditions for short term (temporary works, construction phase) conditions. However the application of this, especially when using computer programs, needs to be carefully applied.

The approximate active earth pressure at any depth H in the undrained condition is given by the following formula, (assuming no surcharge load):

- $Pa = \gamma H - 2Cu$

Where:
γ is the unit weight
H is the height and
Cu is the undrained shear strength

Therefore, say, for a

- 7m retained height
- Unit weight of $20kN/m^3$
- Undrained shear strength Cu of $100kN/m^2$

- $Pa = (20x7) – (2x100) = 140 – 200 = -60kN/m^2$

This is a negative load or essentially a zero load on the wall and implies that the soil will stand unsupported. However this does not take account of the fact that water can become trapped behind the wall (even if the groundwater level is lower) or that sand or gravel layers may be present in the soil mass.

Therefore in this condition it is advisable to use one of the following approaches, depending on the project specific conditions:

- Apply a nominal Ka value, say 0.2
- Apply a Minimum Equivalent Fluid Pressure (MEFP) as detailed in CIRIA C580, retaining a fluid of $5KN/m^3$ where groundwater is not expected.

9.0 SUPPORT SYSTEMS

Embedded retaining walls are designed to be either:

- Cantilevered
- Singularly propped
- Multi-level propped

In urban environments generally the main criteria when deciding on the required support system is the allowable displacement of the wall rather than issues of ultimate strength.

It is generally always desirable for a retaining wall to operate in cantilever mode or to be supported by tieback anchors. This is due to operational and water proofing difficulties when working around internal propping systems. However it is not always possible to use either cantilevers or tieback anchors and in these instances internal propping will be required.

In relation to cantilever walls Figure 10 and Table 3 below detail the approximate displacements which could be expected for the ground conditions and geometry shown. The analyses were carried out on the computer package FREW. The maximum pile head displacement varies from 20mm to 130mm, depending on the configuration used. In many cases deflections above, say, 50mm would not be acceptable and an alternative configuration would be required.

Excavation Depth (H_1,m)	Pile Diameter (D,mm)	Pile Spacing (m)	Embedded Depth (H_2,m)	Maximum Displacement (S,mm)
4	600	0.50	4.25	20
5	600	0.50	5.50	55
5	900	0.75	5.50	25
6	600	0.50	7.00	130
6	900	0.75	7.00	55
7	900	0.75	8.25	105
7	1050	0.87	8.25	75
7	1200	1.00	8.25	60

Table 3: Predicted Displacement Values using FREW

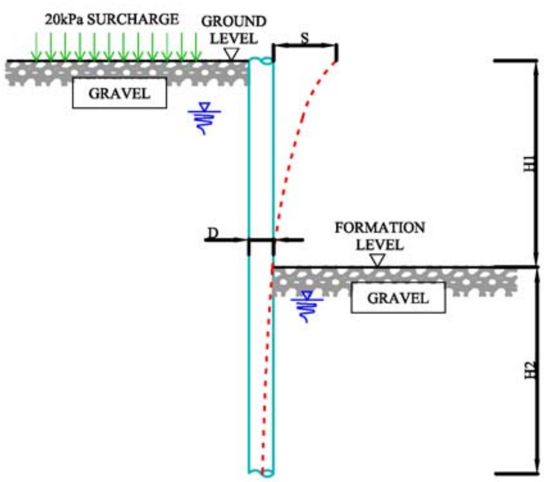

Figure 10 Cantilever Wall

This simple analysis assumes 20kN/m² surcharge loading in the serviceability condition.

The above analysis is based on cohesionless (e.g. gravel) soil conditions, lower pile displacements would be expected in a short-term analysis if, for example, a stiff clay existed in place of the gravel. Greater pile head displacements would be expected if the wall were surcharged by even a modest building foundation (e.g. 150kN/m run).

An example of a cantilever wall is presented in Figure 3.

If typical values of maximum allowable pile head displacement of:

- building close to wall: 10mm – 15mm
- roads close to wall: 15mm – 50mm

are assumed then it becomes apparent that in many cases some form of retaining wall support system will be required. The most commonly used support systems are:

- Passive props
- Active props
- Tie-back anchors

All of these systems are generally used in conjunction with capping beams or wailing beams. As discussed earlier in this paper the soil–structure interaction models used to predict retaining wall behaviour are essentially stiffness models. The stiffness along with any prestress load in the support system is an integral component of these stiffness models.

Passive raking props or cross props usually comprise Universal Column (UC) or Circular Hollow (CHS) steel sections. They generally cannot have an initial prestress load and they will elastically shorten when load is applied from the retaining wall. Where raking props are used then reaction can be provided by either a thrust block, a piled foundation or alternatively part of the permanent works (e.g. basement slab, column foundation etc can be used). The movement of thrust block must also be calculated and included as part of the wall movement as must the displacement of any wailing beam or capping beam which is used to span between these props.

An example of passive propping is presented in Figure 4.

Active props (e.g. Groundforce) can include an initial prestress load which can reduce overall retaining wall movements. However these systems are usually more expensive than the passive systems described above.

An example of active props is presented in Figure 11 below.

Figure 11 Active Props, James Joyce St, Dublin (G&T Crampton)

Tie-back anchors usually comprise of high strength single bars or multiple strand steel elements. These can be classified as high-strength-low-stiffness support systems. Due to their low stiffness it is normal to apply an initial prestress loading of approximately 50%-75% of their design service load. This initial prestress load will substantially reduce the retaining wall movements.

Tie back anchors are normally, but not always, installed at 45^{o}. These elements can be quite long and in many cases will extend into neighbouring properties. Therefore appropriate permissions/wayleaves must be arranged in advance.

An example of a tied back retaining wall is presented in Figure 2.

10.0 MONITORING

10.1 Movement

Movement monitoring of embedded retaining walls forms an important part of the overall design sphere and can be used for the following distinct purposes:

- To validate the retaining wall design displacement predictions
- To record actual movements so that data is available in relation to neighbouring structures and services.
- To provide an early warning system if greater than expected movements occur as the excavation proceeds.
- In cases where the design may predict that the retaining wall movement is marginally above the allowable limits but conservative design parameters have been employed then an "observational" approach can be used. This is where the wall is monitored as the excavation proceeds and additional support is only installed if the measured movement exceeds a pre-set criteria. This is a very useful approach as it can sometimes allow cumbersome support systems to be omitted.

Movement monitoring is generally carried out in three main ways although many systems are available. These are:

- Inclinometers
- Tiltmeters
- Conventional Surveying (levels, total stations etc)

An inclinometer system comprises a 75mm plastic tube (with running tracks in orthogonal directions) and an inclinometer instrument. The inclinometer tube is grouted into a borehole or installed into a concrete pile, with the tracks as close as possible to perpendicular and parallel to the excavation line.

The inclinometer instrument is installed into the tubing and records the inclination of the tubing at 0.5m intervals. By integrating these inclinations the displaced shape of the tubing, and thus the retaining wall, can be determined. Readings are carried out at various stages during the basement works and so the performance of the retaining wall can be monitored. This is then compared with the design predictions and can allow the design to be refined if required.

An example of an inclinometer instrument is presented in Figure 12 while an example of a displacement profile recorded with an inclinometer is presented in Figure 13.

Figure 12 Inclinometer instrument being lowered into position

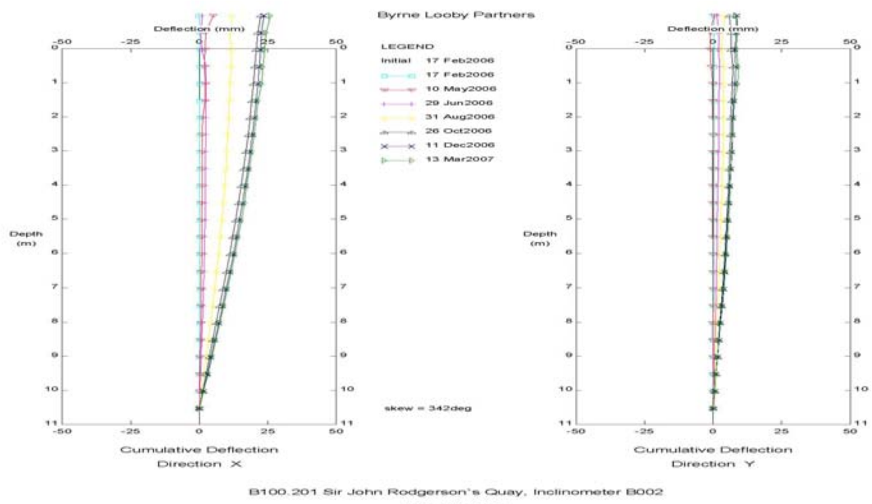

Figure 13 Inclinometer Profile

Tiltmeters, as their name suggests, are devices that accurately measure tilt or rotation and they can be used in a variety of ways. Most commonly they are attached to lightweight beams (typically 2m long), which are in turn attached to adjacent buildings or other retained structures. These instruments can provide data in real-time which can be uploaded to a site computer or website (via a modem) and thus a very rapid appraisal of movements can be made. The instruments can be used to measure the rotation in any pre-defined plane or the results can be converted into differential or total settlement.

An example of a tiltmeter mounted on a beam is presented in Figure 14.

Figure 14 Radio-Tiltmeter

Conventional surveying techniques can be used to monitor settlements and lateral movements of retaining walls and retained structures. However their accuracy (c. ±1-2mm) is somewhat lower than the systems described above.

Survey targets are visible to the left of the tiltmeter in Figure 14 above.

10.2 Forces

The displaced profile of the retaining wall, as discussed above, can be used to determine the bending moment actually experienced by the retaining wall and thus ensure the wall does not become overloaded. Other systems, such as strain gauges on pile reinforcement or support systems and check lifting of tie back anchors can be used to check the axial forces in these elements.

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