DEEP EXCAVATIONS IN DUBLIN
RECENT DEVELOPMENTS

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Photograph shows Westgate 14 m deep excavation in June 2006

SYNOPSIS

A number of Deep Excavations up to 23m in depth have recently been completed in Dublin. Different approaches including propped and unpropped, Secant and Contiguous Pile Wall Solutions have been employed on various projects. The paper updates a database for propped and cantilevered wall supported excavations in Glacial Tills. A comment and interpretation of recorded wall movement versus retained heights and wall stiffness is provided. Modelled predications are also discussed. A number of case histories of deep basement excavations including Spencer Dock in the Docklands, 14m excavation at Westgate (Heuston Square) and other projects are presented and discussed.
1. INTRODUCTION

The recent period of sustained economic growth in Ireland has led to an increase in the use of underground space, with some development now including 4 underground levels. This period has also seen the development of marginal sites, for example areas of Dublin docklands, which previously would have been considered unsuitable for deep basement construction.

The purpose of this paper is to provide an update on recent developments in deep excavations in the Dublin area. Specifically the paper will:

• Briefly review the background geology
• Summarise the situation as existed up to about 2002
• Present recent developments in the use of cantilever retaining walls.
• Review the lessons learned from deep open cuts.
• Present current approaches for the analysis, design and construction of deep excavations by reference to some case histories namely:
  → 7 m cantilever wall at Ballycullen Rd.
  → 14 m excavation at Westgate supported by a single row of anchors.
  → 7 m deep excavation in complex ground conditions at Spencer Dock in Dublin docklands.

Finally some issues related specifically to construction will be discussed and some recommendations given for future works.

2. BACKGROUND GEOLOGY

Bedrock in the Dublin area is a thin to medium interbedded homogenous grey argillaceous limestone and calcareous shale. Over much of the city, it is overlain by glacial deposits, known colloquially as Dublin boulder clay (DBC). This is hard lodgement till which was deposited beneath the ice sheet that covered much of Ireland during the Pleistocene period. It was known that the ice thickness in Dublin was approximately 1 km and that several advances and retreats of the glaciers occurred in the area. The grinding action of this sheet as it eroded the underlying rocks coupled with its loading effect resulted in the formation of a very dense / hard low permeability deposit, which contains pockets of lenses of coarse gravel, particularly at depth. Oxidation of the clay particles in the top 2 m to 3 m has resulted in a change in colour from black to brown and a lower strength material.

With the construction of the Dublin Port Tunnel, a clearer understanding of the detailed geology of these deposits has emerged, see Skipper et al. (2005). The details of the engineering properties and engineering behaviour of DBC has been reported by Farrell and Wall (1990), Long and Menkiti (2007a and 2007b).

Geological conditions in the Dublin docklands are complex and comprise a series of estuarine clays, slits, sands and gravels. The situation in the docklands area is complicated by the presence of a pre-glacial channel just north of the River Liffey which was identified by Farrington (1929). It diverges from the present channel of the River Liffey near Connolly Station and returns near the mouth of the river. It is not clear whether widely varying sea levels, tectonic movements or some weakness in the underlying rock gave rise to the channel. However from an engineering point of view, it has significant importance in that it generally filled with deposits of glacial and fluvio-glacial gravels. A study of the deposits in this area is currently being carried out at UCD (Research student Brian Kearton). Useful information can also be found on the website of the Geological Survey of Ireland (www.gsi.ie).

3. SITUATION UP TO 2002

Up to about five years ago, basements in Dublin generally comprised two underground levels and were often constructed within lightly supported contiguous or secant piled retaining walls. Some typical examples of these projects are the 6 m deep excavation at Dáil Eireann, see Figure 1A and the 7.4m excavation at Trinity College (see Figure 1B) (Brangan, 2007)

A possible exception to these early developments was that of the Jervis St. shopping centre, where the secant piled retaining wall had to be designed for an 8 m deep dig and to support large loads from the old Jervis St. hospital façade and the adjacent Marks and Spencer store on Mary St., see (Dougan et al., 1996).

In general these retaining systems behaved very well. Lateral wall movements were very small and prop forces were less than traditional design approaches would predict in boulder clay. At Jervis St and the Dáil some attempts were made to monitor the prop forces and they were found to be dominated by temperature effects. Loading resulting from the excavation was close to zero. Brangan and Long (2001) (see also Brangan, 2007 and Long 2002b provided a summary of the situation up to that time and included data from 9 propped or anchored walls and 3 cantilever walls. All of these sites were underlain by competent glacial deposits.

![Figure 1. Dáil Eireann 6 m excavation adjacent to (a) Senate chamber and (b) Trinity College](Image)
The conclusions of this work were:

- The support systems behaved in a very stiff manner with displacements and prop forces being much lower than for world wide stiff soil cases,
- In general traditional wall designs are conservative,
- In order to properly model the behaviour soil parameters such as the coefficient of in situ horizontal stress ($K_0$), the variation in stiffness with strain and the undrained shear strength ($s_u$) are very important,
- The pore pressure behaviour during excavation is poorly understood and it is likely that high negative pore pressures (suctions) develop.

4.0 UPDATED DATABASE FOR PROPPED WALLS

The database of Brangan and Long (2001) / Brangan (2007) and Long (2002b) for propped walls in competent glacial deposits is reproduced in Table 1 below and has been augmented by data from eight other sites including the 14 m deep Westgate excavation and data from the Dublin Port Tunnel project where excavation depths were up to 25 m (see Figure 4).

A plot of maximum measured lateral movement ($\delta_h$) versus retained height (H) is shown on Figure 3a. Except for the project in Tallaght (see Figure 2B) all $\delta_h$ values are less than 10 mm. There does appear to be some weak tendency for an increase in $\delta_h$ with H.

The stiff behaviour of the very deep Westgate and Dublin Port Tunnel excavations are particularly worthy of note.

Also shown on Figure 3a are lines representing normalised movement ($\delta_h/H$) of 0.18% and 0.4%. The former relationship was obtained by Long (2001) for an average of 169 case histories worldwide where there was stiff soil at dredge level. The behaviour of the Dublin projects is significantly stiffer than the worldwide average. The 0.4% line represents a typical design value as recommended by CIRIA report C580 (Gaba et al., 2003) and clearly this relationship is very conservative for the Dublin cases.
5. RECENT DEVELOPMENTS IN CANTILEVER WALLS

5.1 General

Cantilever walls form ideal temporary or permanent works. The site remains free of internal props or struts allowing construction to continue without obstacles. In particular perimeter works such as drainage can proceed without hindrance. Difficulties associated with ground anchorages, such as obtaining wayleaves or proof testing are also avoided. Soil mechanics textbooks suggest cantilever walls are only suitable for modest excavations up to about 4.5 m retained height. However throughout the world and more recently in Ireland cantilever walls have been used to retain excavations significantly greater than 4.5 m. In the following sections the performance of cantilever walls worldwide and in Ireland will be reviewed and attempts will be made to explain why the behaviour of these structures surpasses that suggested by conventional soil mechanics.

5.2 Cantilever walls worldwide

A summary of the case histories used is given on Table 2. Relatively high cantilever walls have been used for some time. Use of these walls was based on the practical experience of open cuts in the glacial tills being able to stand unsupported at very steep angles. Some examples of contiguous piles retaining walls from Intel in Leixlip and Charlemont Place, in Ranelagh, Dublin are shown on Figure 8 (Long, 1997).

More recently cantilever walls of 7.5 m or so are being used regularly. An example of the 7.5 m high cantilever 600 mm diameter contiguous pile wall at Hunters Wood, Ballycullen Rd. is shown on Figure 9. The design, construction and performance of this structure will be discussed in more detail in a later section of the paper.
Figure 5. Cantilever retaining walls $\delta_h$ versus $H$ (a) world-wide case histories, (b) Dublin

Figure 6. Cantilever retaining walls $\delta_h/H$ versus $H$ (a) world-wide case histories, (b) Dublin

Figure 7. Cantilever retaining walls $\delta_h/H$ versus Clough et al. (1989) system stiffness (a) world-wide case histories, (b) Dublin
Figure 8. Cantilever retaining walls at Intel, Leixlip (6.8 m max.) and (b) North Wall Quay (7.5 m max.) Sept 2007

Figure 9. 7.5 m high cantilever wall at Ballycullen Rd. (a) during construction in June 2005 and (b) in service September 2007

Plots of $\delta_h$ versus H, $\delta_h/H$ versus H and $\delta_h/H$ versus Clough et al. (1989) system stiffness for the Dublin case histories are shown on Figures 5b, 6b and 7b respectively. In each case the best-fit trendline from the worldwide database of cantilever walls are also shown.

The Dublin walls have performed very well, with values falling in general well below the trendlines. Average $\delta_h$ and $\delta_h/H$ values are about 5 mm and 0.08% respectively. An exception is the Thorncastle St. case history (Long et al., 2002) where there was soft alluvial soil at dredge level.

Again it seems that lateral movement is independent of system stiffness and once a sufficiently stiff system is provided the walls will behave well. Again it seems Irish practice is conservative and perhaps even more conservative than that worldwide. Based on these data it seems there is scope for the use of higher cantilever walls, at least for temporary work purposes.

5.4 Behaviour with time

The data presented above omit one very important factor, i.e. how does the lateral movement vary with time. This has important implications as to whether these walls can be used for permanent work as well as temporary works and also in the temporary case how long is the useful life span.

Data from five sites in Dublin, Ballycullen Rd., Cork St., New St. and the Dublin Port Tunnel all in glacial till and Thorncastle St. in soft alluvial soils are shown on Figure 10a. The data for Ballycullen Rd is of particular interest as it spans a period of some 140 weeks (2.7 years). In most of the projects, after a relatively short time, the retaining wall was incorporated into the permanent works meaning it was no longer acting as a cantilever. The Ballycullen Rd site is unusual as the wall forms the permanent works, see Figure 9b. On Figure 10b the data for the first 40 weeks is shown in more detail.

It can be seen that in all cases there is a gradual development of movement increasing with time. For the glacial till cases the rate of increase of lateral displacement is relatively slow. However for the soft alluvial soils case at Thorncastle St the development of movement is rapid.
Figure 10. Cantilever retaining wall movement with time

The reason for this behaviour is the gradual dissipation of negative pore pressures (suctions) and the build up of positive pore pressure. This will be discussed in the following sections.

6. LESSONS LEARNED FROM STEEP UNSUPPORTED CUTS IN DBC

6.1 Natural slopes

There are several examples of natural steep slopes in glacial tills in the Dublin area. Hanrahan (1977) and Long et al. (2003) describe near vertical slopes of up to 33 m high in Howth (north of Dublin), Killiney (south), Greystones (south of Dublin in Co. Wicklow) and Chapelizod (west). There are similar high natural cuts along the River Dodder, in the south of the city.

A view of the cliffs at Greystones is shown on Figure 11a. Local instability can be seen, for example when the slope was undercut by sea erosion.

6.2 Man made temporary cuts

Temporary cuts of significant thickness, made for engineering works, can stand unsupported for relatively long periods, see Figure 11b for example. Groundwater level (as recorded by piezometers) was at about 2 m depth. The cut was made during the summer time and is mostly in the weathered upper brown boulder clay (UBrBC). No ground water control system was used within the excavation and no cracks were observed at the slope crests, for a period of several months. The authors have seen other near vertical 8 m deep cuts in central Dublin stand unsupported, adjacent to heavily trafficked roads, for periods of several months.

6.3 Instability in steep cuts

Some instances of instability of cuts in Dublin boulder clay have been reported. For example Hanrahan (1977) discusses failures caused by water bearing granular layers in slopes in Glencullen, south Dublin. During December 2000, the railway line adjacent to the sea cliffs at Killiney had to be closed on three occasions due to landslides. These were due to surface slips in the upper (weathered) portion of the till which occurred after periods of high rainfall. Some local instability can be seen in the slope beneath the Martello tower at Howth, Co. Dublin. This failed in December 2002 following a period of heavy rain and caused the road underneath to be blocked.

Long et al. (2003) describe some minor failures in an 8 m high steep open cut in north Dublin.

Figure 11(a) Glacial till sea cliffs at Greystones, Co. Wicklow, (b) man made 6 m cut in Rathfarnham, south Dublin.
Menkiti et al. (2004) describe the failure at the trial trench excavated for the Dublin Port Tunnel project.

The failures typically occur after periods of high rainfall, at the location of granular lenses and are characterized by the slip of thin flakey wedges.

The behaviour of these natural and man made cuts should give some clues as to the surprisingly stiff behaviour of the Dublin retaining walls.

6.4 Possible reasons for stable steep cuts

The possible reasons why steep cuts in Dublin glacial till can stand unsupported for significant time periods are discussed in detail by Long et al. (2003). Possible explanations for this observed behaviour are:

- The till possesses a high drained cohesion (c’) component.
- The particles are cemented together.
- Pore water suctions.

Long and Menkiti (2007a and 2007b) have explained why the depositional nature of the till and its particle size distribution have resulted in it having a curved rather than the normally assumed linear failure surface with c’ = 0. Evidence from scanning electron microscope photographs has confirmed no appreciable cementation exits between the soil particles. Therefore pore water suction must perform a significant role in the observed behaviour of the material. This is consistent with the failure of the material in the presence of granular lenses and high rainfall.

6.5 Pore water suctions

The pore water pressure conditions in the slope before and after excavation are shown diagrammatically on Figure 12. Before excavation the pore water pressure (u) is simply given by the hydrostatic head below the groundwater table, which is at about 2 m depth, i.e.

\[ u = \gamma w z \]  

The change in u caused by the excavation can be determined (approximately at least) from Skempton’s (1954) classical equation for pore water pressure change:

\[ \Delta u = B[\Delta \sigma_t + A(\Delta \sigma_t - \Delta \sigma_3)] \]  

In this case both the all round pressure (\(\Delta \sigma_t\)) and the deviator stress (\(\Delta \sigma_t - \Delta \sigma_3\)) reduce due to the excavation-induced stress relief. This means that u reduces and, depending on its initial value, could become negative. If u reduces then the effective stress increases, thus improving, in the short term, the stability of the slope. The length of time over which this reduced pore water pressure can be sustained is a complex issue and depends on the soil type, its fabric, permeability, the sequence of construction, slope protection, weather, etc.

As well as increasing the strength of the soil, from the point of view of retaining wall design the net effect is that there will be none or at least very low pore water pressure acting on the wall. Normally designers assume hydrostatic water pressures, which can be higher than soil pressures.

As can be seen from Skempton’s formula above it is necessary to determine \(\Delta \sigma_t\) and \(\Delta \sigma_t - \Delta \sigma_3\) in order to calculate \(\Delta \sigma_t\) and \(\Delta \sigma_t - \Delta \sigma_3\) and the finite element analysis. This is not a trivial matter. Some elastic based solutions exist. In the Dublin Port Tunnel project use was made of the finite element approach. Further details of these analyses can be found in Menkiti et al. (2004) and Kovacevic et al. (2007). This work was carried out by the Geotechnical Consulting Group (GCG), who made use of the Imperial College, London, geotechnical finite element code called ICFEP. Some typical output for the Dublin Port Tunnel trial trench excavation is shown on Figure 13.

6.6 Some examples of measured pore water suctions

The techniques used for measuring pore water suctions on the Dublin Port Tunnel project are described in detail by Long et al. (2004). This includes a description of the special piezometers used and the method used to seal them into the ground.

A typical example for a piezometer at about 5.5 m depth in the trial trench is shown on Figure 13b. It can be seen that the measured suction of about –30 kPa is very similar to that predicted by the finite element analysis.

To the authors knowledge there has been no measurements made of pore water suctions behind retaining walls in Dublin boulder clay. Such data would be of significant practical use in the future design of these systems.

**Figure 12. Pore water pressure reduction in cut slopes**

![Diagram showing pore water pressure changes before and after excavation](image-url)
Figure 13a. Predicted pore water pressures for Dublin Port Tunnel trial trench and (b) typical measured value for piezometer at 5.5 m (Long et al., 2004)

7. OTHER SOIL FACTORS INFLUENCING RETAINING WALL BEHAVIOUR

In addition to the role of pore water suctions there are other significant factors influencing the very stiff behaviour of retaining walls in Dublin boulder clay. These are best illustrated by examining a series of high quality isotropically consolidated (CIUC) triaxial tests (carried out by NMTL Ltd. for ARUP Consulting Engineers) for a site in the south city centre. The tests were carried out on GeoBore-S rotary cores.

From the test results it can be seen:

- The material is very strong with undrained shear strength ($s_u$) typically between 400 kPa and 500 kPa.
- It is ductile with peak strength being developed at relatively high strains. (This is advantageous from the point of view of steep slopes or retaining wall moments as the signs of incipient failure will be seen some time before failure occurs).
- It is highly dilatant with negative pore pressures developing during shear.
- Stiffness is highly non linear.
- Stiffness values are very high. Shear modulus ($G$, as measured by local sample mounted transducers) varies between 400 to 600 MPa at strains of $5 \times 10^{-4}$ % to less than 100 MPa at 0.01%.
- Stiffness measured by local strain transducers in the laboratory is consistent with very small strain stiffness measured in situ using MASW surface wave techniques (Donohue et al., 2003).

Figure 14. CIUC triaxial test results for Dublin boulder clay (Courtesy NMTL Ltd. And ARUP Consulting Engineers)
8.0 DESIGN APPROACH

The previous sections have shown that low level displacement has occurred for a number of various types of retaining walls, primarily in boulder clay soils. From a practitioners point of view in order to understand why low level deflections are occurring it is important we understand the background to retaining wall design. The following section briefly outlines the general pile design approach and the issues critical to wall movement.

As with 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 the following issues which are separated into those critical to movement and other issues:

Movement Related Issues:

- Allowable movements
- Ground Model - Cohesive or Cohesionless
- Quality of Site Investigation
- Depth of required excavation
- Prevailing groundwater conditions
- Retained structures and services – surcharges
- Propped/cantilevered
- Temporary/Permanent retention

Other Issues:

- Space limitations/available plant
- Budget and programme
- 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.

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 are applied and earth pressure diagrams are derived, 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.

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 and hence cannot be used to predict deflection.

8.3 Serviceability Limit States

Serviceability limit states correspond to conditions beyond which specific service performance requirements are no 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 it’s 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.

Before construction activities occur soil exists in-situ at “At-rest” or “Ko” (co-efficient of earth pressure at rest) conditions, see Figure 15. 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 15 Earth Pressure at Rest Ko](image)

As an excavation proceeds, see Figures 16, 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 16 – Ko Condition](image)
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 17 below.

**Figure 17 - Example of PLAXIS Output**

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

### 8.4 Modelling Method/Approach

After horizontal or vertical unloading theoretical soil and groundwater pressures in cohesive soils can be significantly less than zero and can sometimes provide no active load on a retaining wall. This will be further discussed below. For a number of reasons this is not an acceptable design approach as set out below:

- Tension cracks can develop to the rear of the wall. The relevant code (CIRIA 580) requires the use of a Minimum Equivalent Fluid Pressure (MEFP) for tension cracks
- Sand and gravel lenses which are a common occurrence in the Dublin boulder clay can supply sufficient groundwater to allow hydrostatic groundwater pressures develop to the rear of the wall
- Potential unidentified pockets of sand and gravel could provide loading conditions and failure modes that tend towards effective stress conditions. (This condition applies to the passive side also).

In order to account for the above criteria in retaining wall design BLP currently use one of the following approaches – (a key factor in which approach is employed is the level of the groundwater for the particular project).

- Effective stress conditions with a low Ka value
- Undrained soil parameters with a minimum equivalent fluid pressure employed in the analysis as per CIRIA C580.
- Undrained soil parameters with full hydrostatic groundwater pressures
- A combination of the above criteria

The above paragraphs outline conditions that can occur and the relevant design criteria. In reality, full hydrostatic water pressures, MEFP conditions or effective stress conditions may not exist and actual loading conditions on the wall could be closer to the undrained or partially undrained conditions where complex (incl. negative) pore pressures exist. This results in low or zero lateral pressures on the wall. Obviously the deflection predictions from the analysis with undrained parameters will be significantly less than any of the design approaches discussed above.

The use of undrained parameters in conjunction with the observational approach may be considered for reducing predicted deflections to simplify the construction sequence and reduce costs. (In addition to this, how we model groundwater with time post excavation should also be considered but is not addressed as part of this paper). This approach should also only be considered where the predicted deflections using traditional approaches are within defect limits to prevent the possibility of damage, economic loss or unsafe situations. The risk associated with the decision should be clearly assessed in terms of understanding of the site geology, type and condition of structures to the rear of the pile wall and the quality of monitoring procedures put in place. This decision to use undrained parameters will have a large impact on predicted deflections as discussed above.

The benefits of using undrained parameters are not so much that pile sizes will be reduced but that more cost effective overall solutions can be employed in more areas. It seems there is scope for the greater use of cantilever walls and also greater retained heights at least for temporary works purposes.

Similar design issues arise in relation to undrained soil parameters on the passive side of the wall and undrained analysis can potentially allow for very shallow embedment depths. In order to allow for the possibility of unknown gravel layers, effective stress parameters are often used to determine the overall stability requirements. However the serviceability analysis may require the benefit of using undrained material on the passive side in order to more accurately predict deflection.

### 8.5 Quality of Site Investigation Data

As with all geotechnical applications, quality of site investigation is critical to accurately predict movements. The results of recent consolidated triaxial tests presented in Section 7.0 are consistent with monitored pile deflections in boulder clay, in that they suggest that we are currently significantly under estimating the stiffness parameters of the glacial tills. Higher quality laboratory testing on larger projects should be considered.

Investigations should accurately establish the static groundwater profile at the site. As noted previously, to the author’s knowledge there have been no measurements made of pore water suctions behind retaining walls in Dublin boulder clay. Such data would be of significant practical use in the future design of these systems.

### 8.6 Critical Issues for Movement

A summary of the critical issues in relation to retaining wall deflections and predictions are as follows:

- Cohesive or Cohesionless model – in conjunction with how we model groundwater with time
- Continuous Wall (Secant/sheet pile) or contiguous
- Surcharge
- Propped/Cantilevered
- Groundwater conditions
- Soil parameters – quality of investigation
- Attitude to risk

### 9.0 CASE HISTORIES

The design and monitoring of 3 No. deep basements are described in the following sections. All projects were completed in the past three years. The projects include Spencer Dock Development at North Wall Quay Wall, the Westgate (now called Heuston Square) project and a residential development in Dublin 24.
Figures 18/19 - Spencer Dock and Westgate

The projects were selected to compare the performance of permanent and temporary, propped and cantilevered walls in various soils types and the accuracy of predicted deflections.

9.1 Westgate

The Westgate Development (see Figure 19) is a combined commercial and residential development, adjacent to Heuston Station, south of the Liffey. Topographical levels on the site vary between 14m OD at the south end of the site and 6m OD at the north. Basement formation was 0m OD resulting in a 14m retained height at the southern end.

Ground stratigraphy consisted of stiff to hard glacial till interlayered with dense gravel. The gravel layers were intermittent and did not exist consistently across the site. A geological x-section of the site is provided in Figure 20. Soil parameters employed for the different strata are as indicated in Table 3.

A monitoring programme indicated that groundwater levels varied between +8m OD of the southern boundary and 0.9m OD at the northern boundary. The variable levels were due to groundwater flow towards the River Liffey.

Construction Sequence

The construction sequence was as follows:

- Install 900mm Ø “Secant” Soft-Hard Pile wall to 20m bgl at 0.7m cc to achieve overlap. Piles were constructed with a 30 Tonne metre Torque CFA Rig. The male piles were reinforced with 18 No. T32’s over the central third of the pile and 10 No. T32’s over the top and bottom third of the pile and were constructed with C35 Concrete.
- Construct a reinforced capping beam.
- Install, test and pre-stress 700 kN SWL anchors @ 2.25m cc through the capping
- Complete excavation in staged manner with a detailed monitoring programme.

Table 3 - Westgate Soil Parameters

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Depth (m bgl)</th>
<th>E (MPa)</th>
<th>K₀</th>
<th>φ (°)</th>
<th>Cᵤ</th>
<th>Cₜ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>0-1</td>
<td>50</td>
<td>5</td>
<td>30</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Dense Gravel</td>
<td>1-25</td>
<td>150</td>
<td>1</td>
<td>38</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Stiff Brown Clay</td>
<td>1-25</td>
<td>150</td>
<td>1</td>
<td>35</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The following protocol was agreed if the monitored deflection exceeded the trigger value.

1. Excavation works would cease.
2. A second set of readings would be taken the following day.
3. If further movements were observed the wall would be partially backfilled.
4. If the wall had stabilised, a second layer of the tie-back anchors would be installed.

Recorded Movement

Figure 21 illustrates a typical x-section through the pile wall.

A key aspect of the retaining wall design was the use of the “Observational Approach” to monitor the movement of the wall as excavation proceeded. 6 No. inclinometers were installed at key locations across the site to measure pile wall deflection. Inclinometer monitoring was carried out by BLP in-house with monitoring being carried out at critical stages of the excavation.

“Trigger level” lateral movement criteria was established prior to the works commencing for the different stages of excavation. The original design analysis predicted pile deflections of 50mm, the pile was structurally designed to resist the bending moment that reflected this profile. The maximum trigger level value of 35mm for complete excavation was selected to ensure that excessive deflections did not have an impact on the existing listed boundary wall to the rear of the pile wall.

The original Site Investigation suggested that significant gravel layers may be present, inspection of the bulk excavation at the southern end of the site indicated the soil was predominantly a clay material.

Figure 21 illustrates a typical x-section showing the original predicted deflection and the recorded pile inclinometer data. The original analysis included modelling the soil stratigraphy with drained parameters using both FREW and PLAXIS analyses. In the PLAXIS analyses the made ground and glacial soils were assumed to behave as elastic perfectly plastic materials with failure defined by Mohr Coulomb using drained parameters. The analyses gave reasonably consistent predictions within 20% of each other.

Maximum recorded pile deflections were 12mm. The modelled movements are significantly different. It is noted also that while the original Site Investigation suggested that significant gravel layers may be present, inspection of the bulk excavation at the southern end of the site indicated the soil was predominantly a clay material.
For the purposes of this paper an analysis of the wall with undrained soil parameters and a static water level 7m OD was carried out using FREW. Predicted pile deflections were 20mm suggesting that the undrained parameters would model actual pile deflections more accurately. Figure 21 also indicates the predicted pile deflection profile using the drained parameters. The introduction of a consolidation stage in the analysis to model groundwater pressures with time may further improve predictions.

9.2 Spencer Dock

Spencer Dock (see Figure 18) is located in the Dublin Docklands and is 2km east of the city centre adjacent to the River Liffey. Topographical level at the site is +2.5m OD. The site is bounded to the west by the Spencer Dock canal and the south by the River Liffey.

Figure 22 illustrates the typical geology of the site.

The water table on the site is 0m OD and is not significantly influenced by the tidal cycle. The upper made ground consists of brick and masonry in a clay matrix. Due to the variable depositional environment in this section of the site close to the river, the alluvium stratum is a complex mixture of soil types typical of the docklands. The upper 3.5m comprises a loose to medium dense sand and gravel. This is underlain by a soft clay and silt with organic material.

N-values in the soft clay layer vary from 2 to 7. The underlying glacial deposits consist of 2 to 3m of dense gravel overlying a hard till.

Construction Sequence

A 900mm \( \phi \) soft/hard secant pile wall was employed. Tie back anchors were employed at approximately 2.5m bgl rather than at capping beam level. The construction sequence therefore involved an initial cantilevered stage of 2.5m.

Piles were extended to 16m bgl to include a 2m embedment into boulder clay to achieve groundwater cut-off. As with the Westgate project a detailed inclinometer monitoring programme was implemented to monitor pile movement.

Movement

FREW was employed to carry out the original pile wall analysis and indicated that predicted pile deflections would be in the range of 35mm to 40mm. A sensitivity analysis was carried out at the time of the design varying the stiffness and strength characteristics of the soils as well as modelling the layer with drained and undrained parameters for the soft alluvium layer.

Inclinometer readings over the course of the construction stage recorded the deflection profile as being reasonably consistent with modelled deflections, approximately 15% less as per Figure 22, illustrating that the model may be better at predicting movements in the cohesionless soil layers and low strength strata’s as encountered.

9.3 Ballycullen Road

The Ballycullen Road project was a residential development located in the Knocklyon area of Dublin 24 (see Figure 23). The development consists of a number of apartment blocks constructed over a double level basement. Datum level at the site was 107 mAD and basement formation was 99.5mAD providing a 7.5m effective retained height. The wall was installed in January 2005 and excavation was carried out in February 2005.

Groundwater monitoring at the site indicated that static water levels were below excavation level.

The basement directly bounds a public access road to the residential estate necessitating the requirement for the retaining wall.

The geology of the site consisted predominantly of glacial till. A brown Firm to Stiff boulder clay was noted to overlay Hard Black Boulder Clay as per Figure 23. The brown and black boulder clay layers were modelled with effective stress parameters of 35 and 37 degrees respectively. The deeper boulder clay layers were modelled with stiffness parameters of 120,000kPa and 80,000kPa in the temporary and permanent conditions respectively.

Construction Sequence

The pile wall was required to provide both permanent and temporary retention (see Figure 9a & 9b). A 600mm diameter contiguous pile wall solution was chosen based on the geology of the site and the retained height. The construction sequence was as follows:

- Install 600mm diameter hard piles at 750mm cc to a total depth of 14m below ground level. Piles were constructed with a 24 Tonne metre Torque CFA Rig. The piles were reinforced with 8 No. T32 over the full height of the pile wall.
Prior to construction commencing a number of design criteria were established including the following:

- If local gravel layers were encountered they would be shotcreted to prevent loss of ground between piles. If more significant gravel layers were encountered a Secant Pile Wall solution would be installed.
- If movement levels in the construction sequence exceeded a trigger level of 25mm, tie-back anchors would be installed. The original design analysis employed drained conditions to the rear of the wall in the temporary stage. Predicted pile deflections were 40mm and 45mm respectively in the temporary and permanent conditions. The trigger values were selected to ensure that long term deflections would not be excessive in order to avoid damage to the adjacent road.

**Recorded Movement**

2 No. inclinometers were installed at key locations on the pile wall for measuring pile wall deflections. These have been left in place to allow monitoring on an ongoing basis to be carried out and this has been done over the past 2 years.

Recorded pile deflections during the construction stage and post construction to November 2007 (22 Months) were 10mm and 17mm respectively. Recorded deflections to date are significantly less that predicted the deflections of 45mm. An analysis with undrained parameters was carried out for this paper which indicated deflections in the temporary condition of 17mm which is in line with those recorded to date.

**10. CONCLUSIONS**

1. Case history data confirms retaining wall behaviour in Dublin glacial till is extremely stiff. This applies to excavations up to 25 m deep.
2. It appears that current approaches over predict walls deflections and the use of overall current design practice is clearly conservative.
3. The use of undrained parameters in conjunction with the observational approach may be considered for reducing predicted deflections to simplify the construction sequence and reduce costs.
4. This approach should also only be considered where the predicted deflections using traditional approaches are within defect limits to prevent the possibility of damage, economic loss or unsafe situations.
5. Cantilever walls have been successfully constructed up to 7.5 m high. These walls show smaller movements than expected, though the development of movement with time is very important.
6. It seems there is scope for the greater use of cantilever walls and possibly also higher retained heights, at least for temporary works purposes.
7. Important insights into the above can be gained from observations of steep slopes in Dublin boulder clay, where pore water suction plays an important role.

**ACKNOWLEDGMENTS**

Dr. Chris Menkiti and Dr. George Milligan for input from DPT project, Dr. Carl Brangan, former PhD student at UCD. Mr. Tony O’Dowd, PJ Edwards & Co., Mr. Pat Fox, Murphy International Ltd and Mr. Douglas Cook, FK Lowry Piling Contractors.

**REFERENCES**


**Figure 22 Spencer Dock Pile Section**

- Construct the reinforced capping beam.
- Complete excavation in a staged manner with detailed monitoring.
- Install a basement concrete slab to provide a low level prop/restraint.

**Figure 23 Ballycullen Rd Pile Section**

8. Laboratory testing on high quality samples of the material confirm that it is stronger and stiffer than normally assumed in design.
9. Measurement of Pore Water Pressure to rear of walls should be carried out.
10. The analysis employed accurately predicted deflections in the boulder clay.


Table 1. Summary of Case Histories - Propped Walls in Dublin Glacial Deposits

<table>
<thead>
<tr>
<th>Case</th>
<th>Location</th>
<th>Soil at</th>
<th>Soil strength</th>
<th>H (m)</th>
<th>h (m)</th>
<th>Support</th>
<th>s* (m)</th>
<th>Wall type</th>
<th>EI (kN/m²)</th>
<th>Del. h (mm)</th>
<th>Del. v (mm)</th>
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s = Soft
s* = Stiff
Table 2. Summary of Case Histories - Cantilever Walls

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Soft soil at dredge

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<tr>
<th>Case history</th>
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<th>Soil strength $s_u$ (kPa)</th>
<th>H (m)</th>
<th>h (m)</th>
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<th>Del. h (mm)</th>
<th>Del. v (mm)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>UOB Singapore</td>
<td>Soft clay</td>
<td>30 (vane)</td>
<td>3</td>
<td>32</td>
<td>4.2</td>
<td>Diaphragm</td>
<td>4320000</td>
<td>10*</td>
<td>?</td>
<td>Wallace et al (1992)</td>
</tr>
<tr>
<td>26</td>
<td>San Francisco</td>
<td>Soft clay</td>
<td>soft</td>
<td>10</td>
<td>?</td>
<td>23.5</td>
<td>Sheet</td>
<td>61000</td>
<td>220*</td>
<td>?</td>
<td>Clough ??</td>
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<td>28</td>
<td>Thonoma</td>
<td>Soft clay</td>
<td>SPT 10</td>
<td>4</td>
<td>13</td>
<td>5.6</td>
<td>Secant</td>
<td>381700</td>
<td>23.5</td>
<td>?</td>
<td>Long et al. (2002)</td>
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</tbody>
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*s = H or H + fixity