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# Predicted and Observed Performance of an Anchored Retaining Wall in Granite

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## Abstract

**The results of suitability & acceptance tests carried out on tie – back ground anchors for a deep excavation in Dublin are presented. Excavation was made through firm to stiff gravelly clay and weathered granite. The excavation has been supported by 750mm diameter hard – soft secant pile wall rock – socketed into intact granite and tied – back with a single level of ground anchors at capping beam level.**

**Anchor test results are back – analysed to estimate the apparent free lengths of the anchors, these are compared with original design estimates. Monitored wall deflections at various stages in the excavation process are also compared with design predictions.**

**Measured results are discovered to be lower than design estimates. Existing design approaches are shown to be quite conservative. It is recommended that wherever possible, designers should place greater reliance on data from relevant local case histories in similar ground conditions.**

## Introduction

The use of tied-back embedded retaining walls has been a widely adopted practice in civil & geotechnical engineering. While design recommendations and guidelines largely abound in the literature, they are somewhat considered to be only general indicative guides. Experience and knowledge of local ground conditions is thought to be the most valuable tool to a safe, effective and economical design.

This paper is centred on the design, construction, testing and monitoring of the deep basement for a commercial development in Dublin, Ireland. Due to the requirement to provide groundwater cut-off in addition to retaining an excavation of 4.5m-7m depth, temporary works for the basement comprised of a secant pile wall rock-socketed into granite bedrock for effective seal against groundwater ingress. Areas of the basement with retained height $H \leq 5m$ were designed as cantilevered wall sections, while areas with $H > 5m$ were designed to be tied–back at capping beam level with ground anchors inclined at $45^0$.

In recent years, many case histories of tied-back excavation supports have been published in the literature e.g. Finno & Roboski (2004), Symons et al. (1987), Clough & Tsui (1984) and Mulligan (1983). However, most of these have concentrated on overall stability & serviceability performance of the walls, as well as soil deformation around the excavations,

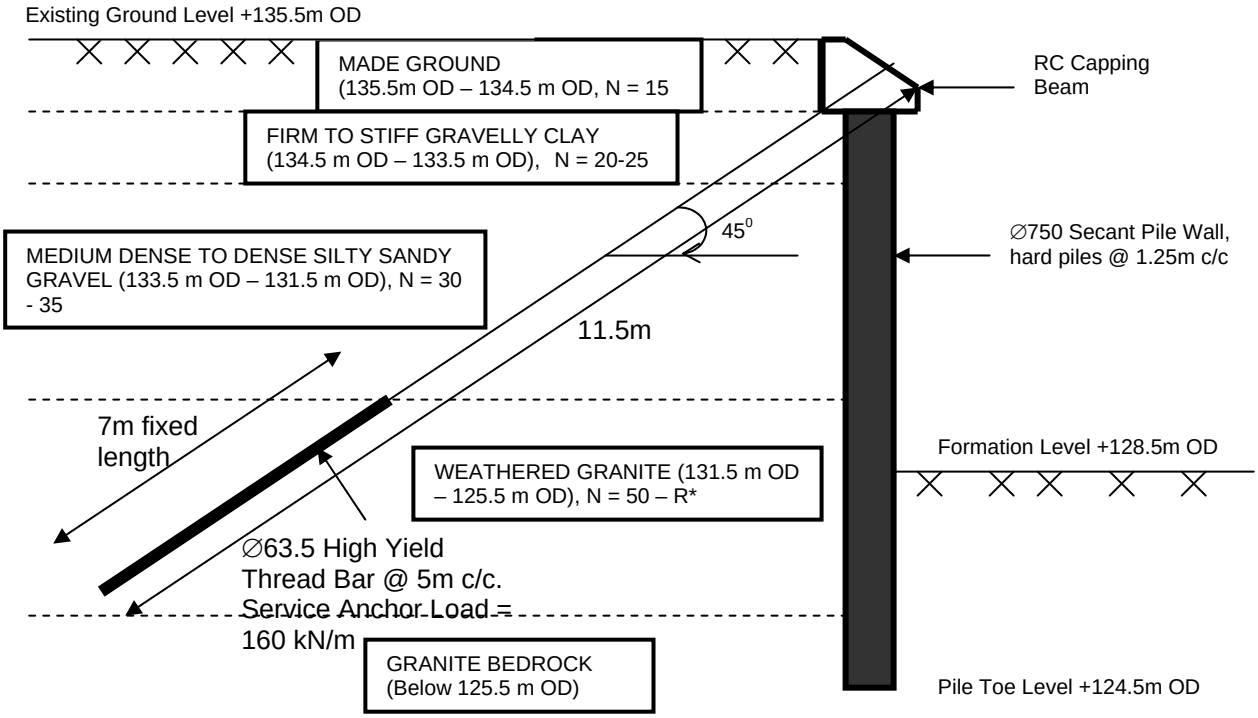
with little reference to the performance of the ground anchors. Detailed studies on the monitored behaviour of tie-back ground anchors in service have been scarce in the literature in recent years. While suitability and acceptance tests are often carried out on installed anchors on projects, results are seldom shared through publications.

In the current study, greater emphasis has been placed on the monitored behaviour of the tie-back ground anchors. The results of the suitability tests on some of the installed anchors are presented, these are back-analysed and related to design predictions. Construction stages are related with monitored inclinometer readings of wall deflection, while comparisons are made with estimates of the pseudo – finite element model used for the design.

## Local Geology

The development is located on the southern end of Dublin, in the foothills of the Dublin mountains. Demolition of the previously existing public house on the site left a legacy of abandoned shallow foundation systems and buried services on the site. While the site terrain is relatively flat, topographical survey shows rapidly rising ground levels to the rear of the site. Site geology chiefly comprises of superficial glacial drift deposits underlain by granite bedrock.

Site stratigraphy includes 1m of made ground overlying a layer of firm to stiff gravelly clay, which extends to 2m depth. Below this stratum is a 2m thick layer of medium dense to dense silty sandy gravel, which is underlain by highly to moderately weathered granite. The highly to moderately weathered granite overlies the granite bedrock, with rock head varying between 7m – 10m across the site. Figure 1 shows the generalised site stratigraphy in relation to the support system. Static groundwater level at the site is 0.5m bgl.




N – Standard Penetration Test N value, R* - Refusal.

Figure 1 – Site stratigraphy in relation to excavation support system.

The firm to stiff gravelly clay is of low to intermediate plasticity with liquid limit ranging between 23% - 43%, while plastic limit varies between 16% - 24%. Clay content in the gravelly clay varies between 5% - 7%. The bedrock is typically strong to moderately strong crystalline fine to medium grained granite. Rotary cores show this to be fresh to locally slightly to moderately weathered, with RQD values of between 48% to 75%. Unconfined compressive strength in the bedrock varies between 40 MPa – 100MPa. Parameters adopted in design are summarised in table 1. It must be noted that coefficients of earth pressure $K_o$ used in design (and shown in table 1) are lower than actual measured values, so as to account for piling – induced stress relief in the ground.

Table 1 – Soil parameters adopted in design.

| STRATUM | $\gamma$ (kN/m$^3$) | E (kPa) | $\phi$' (degrees) | $C_u$ (kPa) | $K_o$ | $\tau_{ult}$ (N/mm$^2$) |
|---|---|---|---|---|---|---|
| Made Ground | 18.0 | 30,000 | 30.0 | - | 0.5 | - |
| Firm to stiff gravelly clay | 19.0 | 40,000 | 30.0* | - | 0.5 | - |
| Medium dense to dense silty sandy gravel | 20.0 | 60,000 | 37.0 | - | 1.0 | - |
| Weathered Granite | 22.0 | 110,000 | 41.0 | - | 1.0 | 0.7 |
| Intact Granite | 25.0 | 300,000 | - | 5,000 | 1.0 | - |

* - analysis based on drained parameters.

## Proposed Development

A large – scale hotel development is being undertaken in South Dublin. The proposed multi – storey development will incorporate the construction of a double–level basement over the entire building foot print for car-parking and storage purposes. The site is bounded on the north & east sides by existing roads and residential buildings. Open grasslands and old stone walls make up the south and west boundaries. Maximum excavation depth for the basement is 7m (6.5m below the static groundwater level on the site), while excavations for the foundations were locally made in the weathered granite to a maximum depth of 1m below basement formation level.

Since excavation was to be made below the existing groundwater level on the site, a secant pile wall was proposed as the most economical and effective soil retention/groundwater cut-off solution for the site. It is anticipated that the lower basement slab will be subject to hydrostatic uplift after construction and hence, it was proposed to install permanent anti-uplift anchors grouted into the underlying intact granite bedrock, to guide against flotation.

The piling and ground anchorage sub-contract for the project was awarded to Quinn Piling Limited on a design & build basis. Parsons Brinckerhoff (Ireland) Limited were appointed by Quinn Piling to provide consultancy services for the design aspect of the piling and ground anchorage sub-contract.

## Construction Procedure

The construction of the basement involved 7 major stages as defined in table 2. Similar steps have also been followed in the analysis and design of the soil support system.

Table 2 – Stages in basement design & construction

| Stage No. | Description of works |
|---|---|
| 1 | Initial conditions; in-situ soil properties and relevant surcharge loads |
| 2 | Install guide frames and hard & soft piles |
| 3 | Excavate to 1.5m depth and construct RC capping beam |
| 4 | In sections where proposed retained height H ≥ 5m, install and prestress tie-back anchors |
| 5 | Complete excavation to basement formation level |
| 6 | Construct lower basement slab and permanent RC retaining wall |
| 7 | Construct upper basement slab and permanent RC retaining wall |

## Secant Pile Wall Construction

Piles were constructed using a bored cast-in-situ method. Pile length varies from 9m in the cantilevered section to 11m in the anchored section. Prior to piling, guide frames were put in place to ensure the accurate positioning of piles in accordance with proposed basement layout. Drilling in the overburden layers was done with a ∅750 Casagrande B250 rotary rig. A smaller diameter (∅600) DTH hammer was used for rock open-holing below the overburden layers. With this approach, the required maximum rock-socket of 3m was achieved. Hard piles were spaced at 1.25m c/c. These were reinforced and concreted with grade C35 concrete, while soft piles were concreted with grade C5 concrete without steel reinforcement.

Pile installation followed the following sequence:

- Install every second soft pile (this is to limit disturbance to fresh concrete).
- After 24 hours, install intermediate soft piles and allow a minimum of 24 hours for concrete to set.
- Install every second hard piles to interlock the soft piles
- After 24 hours, install intermediate hard piles.

## Excavation & Tie-Back Anchor Installation

Excavation was carried out in two stages. An initial excavation to 1.5m depth was done to allow for the construction of the RC capping beam and the tie-back anchors. Tie-back system comprised of 11.5m length ∅63.5 high yield thread bars installed at $45^0$ angles through the capping beam at 5m intervals along the wall. Anchor installation was done with a Casagrande M9 anchoring rig (see figure 2). Figure 3 shows a typical anchorage system after completion, comprising of the grouted tendon, bearing plate and the load bearing threaded nut.

Stages involved in the construction of the anchorage system are summarised below:

- Prebore and install ∅168 steel casing.
- Advance ∅168 borehole to 11.5m length
- Install ∅63.5 high yield thread bar over full length of borehole
- Place OPC grout over full length of borehole (water/cement ratio = 0.4)
- Remove casing and leave grout in place for minimum of 24 hours
- Install 250mm square bearing plate and load bearing nut
- Carry out suitability & acceptance tests and lock off at pre-determined prestress level (50% of anchor safe working load i.e. 400 kN).

Installation and testing of the tie-back anchors was followed by excavation to the required formation level and subsequent construction of the basement slabs & permanent RC walls.

Figure 4 shows the secant pile wall after completion of excavation, while figure 5 shows the construction of the lower basement slab.



Figure 2 – Installation of tie-back anchors



Figure 3 – Completed anchorage



Figure 4 – Wall after complete excavation.



Figure 5– Lower basement construction

## Design of Tie-Back Anchors

Prior to the detailed design of the tie-back anchors, overall stability and serviceability analysis of the secant pile wall was carried out. In the overall stability analysis, an ultimate limit state approach was adopted, using factored soil parameters and loads, to estimate the required depth of embedment of the wall for overall stability. Total and effective stress parameters presented in table 1 were reduced with factors of safety of 1.5 and 1.2 respectively, in accordance with CIRIA Report 104 (1984). Analysis has been done with 'Oasys STAWAL' finite difference modelling software. In the serviceability analysis, unfactored soil parameters as presented in table 1 were used to estimate the service anchor loads, bending moments, shear forces and lateral deflection of the wall. This has been done with 'Oasys FREW' pseudo-finite element modelling software.

### Anchor Design Methodology

The 'Oasys FREW' pseudo-finite element modelling software estimates the maximum service load on the tie-back anchors to be 160 kN/m run of wall. The piling contractor preferred an

anchor spacing of 5.0m c/c, therefore each anchor had to be designed to support a minimum safe working load T of 800 kN. The empirical method of stability analysis shown in figure 6 has been used to determine the location of the uppermost fixed anchor zone, assuming a planar surface.

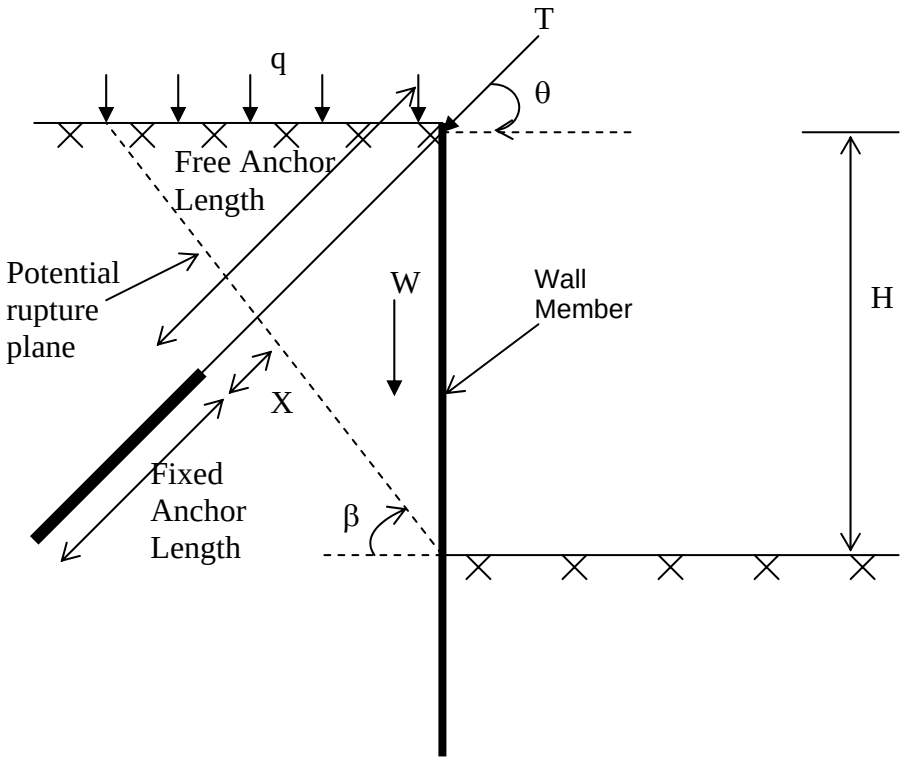



Figure 6 – Wedge method of stability analysis.

In figure 6, β is the angle of inclination of the potential rupture plane, θ is the angle of inclination of the anchorage force, H is the depth of excavation, q is the surcharge and X is the distance between the top end of the fixed anchor and the point where the free anchor intersects the assumed rupture plane. β is estimated from the expression;

$\beta = 45 - \phi'/2$ ---------- (1)

Typically, many designers tend to conservatively adopt a value of $\beta = 45^0$ for the rupture plane (e.g. Mansur & Alizadeth (1970), Clough & Tsui (1984) and Booth (1966)). Adopting a similar approach, $\beta = 45^0$ was assumed in design, anchorage angle $\theta = 45^0$, maximum excavation depth H = 7m. Assuming a design fixed length $L_2$ of 7m in the weathered granite (after Ostermayer (1974), BS 8081 (1989)), analysis based on figure 6 gives a corresponding design free length $L_1$ = 4.5m, X = 0.7m and a total anchorage length L = 11.5m. A typical cross – section of the anchored retaining wall has been presented in figure 1.

The following design checks were carried out on the anchorage system:

- Tendon-Grout Bond Capacity; for an assumed bar/grout bond stress of 2.0 N/mm$^2$ and factor of safety F = 2.5 (after BS8081, 1989) on a ∅63.5 tendon, design was found to be satisfactory.
- Grout-Ground Bond Capacity; BS8081 (1989) recommends that ultimate skin friction $\tau_{ult}$ could be estimated from the expression;

$\tau_{ult} = 10\% - 30\% \times f_{cu}$ ---------- (2)

where $f_{cu}$ is the characteristic compressive strength of rock. Taking $f_{cu}$ in the highly to moderately weathered granite stratum to be 10 MPa, corresponding 10% $f_{cu}$ = 1.00 N/mm$^2$. Suzuki et al. (1972) recommend $\tau_{ult}$ = 1.5 – 2.5 N/mm$^2$ for weathered granite, while Koch (1972) suggests $\tau_{ult}$ = 0.35 – 0.7 N/mm$^2$ for weak rocks. Accordingly, $\tau_{ult}$ = 0.7 N/mm$^2$ has been adopted in the design of the anchors on the current project. For an assumed $\tau_{ult}$ of 0.7 N/mm$^2$, ∅168 borehole, 7m fixed anchor length in the weathered granite and a factor of safety F = 2.5 (after BS8081, 1989), design was found to be satisfactory.

- Tendon Capacity; based on a factor of safety F = 1.6 for temporary anchors (after BS8081, 1989), the most economical and readily available grade of solid thread bar (∅63.5, 800kN SWL) was recommended for construction.

## Anchor Testing

All installed anchors were tested in accordance with clause 11.3 of the BS8081 (1989). The programme of tests comprised of:

- On-site suitability tests; carried out on the first 6 No. anchors installed on site, to verify the suitability of the ground anchors for local conditions on site. Anchors were loaded to 1.5 times their safe working load (SWL) in 3 cycles.
- On-site acceptance tests; conducted in accordance with clause 11.4 of the BS8081 (1989) on all installed anchors. This involved loading the anchors to 1.5 times their safe working load (SWL) in 2 cycles.

Loads were applied with a hydraulic jack at a loading rate of 40 kN/min. For each cycle, load was increased in stages to 10%, 50%, 100% and 150% of the SWL. Subsequent unloading is done in similar stages. Measurements of tendon displacements were made with LVDTs attached to the surface of the tendon. A multi-channel data logger was used to acquire output data from the LVDTs at 1 second intervals. Each stage loading was held for at least 1 minute, while measurements of displacements were recorded at the start and end of each period. However, each peak cycle load was held for 15 minutes, with an intermediate displacement reading taken at 5 minutes.

## Experimental Results & Discussion

### Anchor Test Results

Measurements of load and displacement from the suitability tests can be illustrated with load-displacement curves. Load displacement curves for the test on anchor No. 13 is presented in figure 7. The figure shows the load-displacement response of the anchor during 3 cycles of loading. After each cycle, the anchor is seen to exhibit stiffer responses on reloading. This may be attributed to cyclic preloading, which causes the consolidation of the surrounding material and mobilizes a longer fixed length. Plastic deformation $d_p$ increases with increasing number of cycles; after the first cycle, $d_p$ = 0.2mm and this increased to 0.32mm and 0.38mm after the 2$^{nd}$ and 3$^{rd}$ cycles respectively. Also, change in plastic deformation $\Delta d_p$ (which is a measure of the difference between $d_p$ at the end of a given load cycle and $d_p$ recorded at the end of the preceding cycle) is observed to reduce with increasing number of cycles. $\Delta d_p$ = 0.12mm at the end of the 2$^{nd}$ cycle, this reduced to 0.06mm at the end of the 3$^{rd}$ cycle.

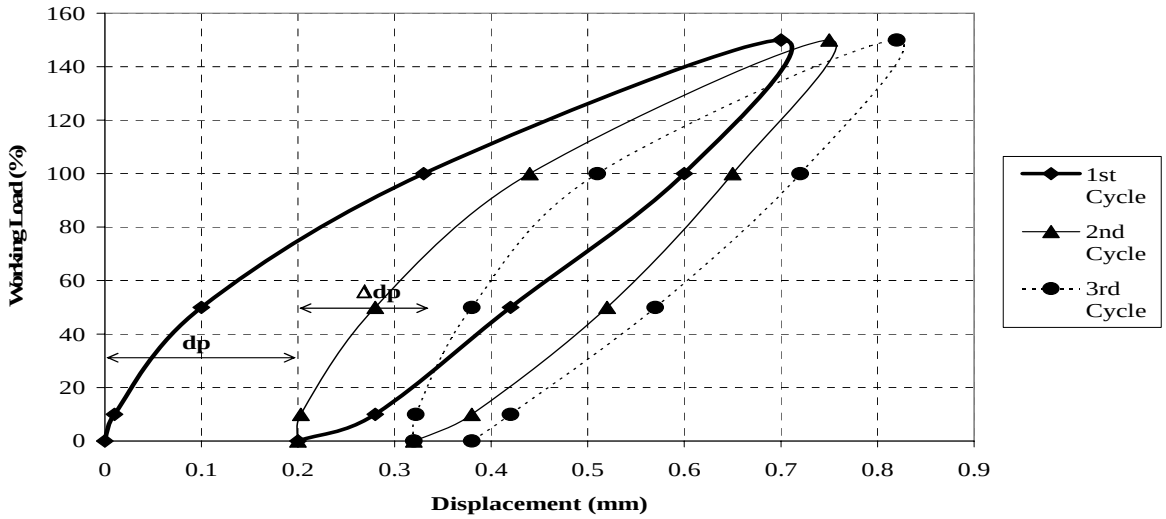



Figure 7 – Load-displacement response under cyclic loading.

In accordance with BS8081 (1989), the apparent free length $L_d$ of the anchor can be deduced from the load-displacement curve over the range 5% $f_{pu}$ – 80% $f_{pu}$, where $f_{pu}$ is the characteristic tensile strength of the tendon, using the expression;

$$L_d = A_t * E_s * \Delta e / T \quad \text{---------- (3)}$$

Where $A_t$ = tendon cross-sectional area (= 3167mm$^2$), $E_s$ = tendon elastic modulus as provided by manufacturer (= 200 x 10$^6$ kN/m$^2$), $\Delta_e$ is tendon elastic displacement given by the difference between the displacement at peak cycle load and displacement at datum load, after allowing for temperature and bedding of the anchor head and T is the difference between the peak cycle load and datum load. Analysis has been based on the unloading curve for the 2$^{nd}$ cycle of each test, results of these for 6 No. anchors are presented in table 3. In table 3, the design free lengths $L_1$ had been estimated with a 45 degree failure wedge ($\beta = 45^0$) as illustrated in figures 1 & 6, while fixed length $L_2$ = 7m in weathered granite had been assumed in design as described earlier.

Table 3 – Summary of analysis results

| Anchor No. | Installed Anchor Length(m) | Design Free Length $L_1$ (m) | Design Fixed Length $L_2$ (m) | Maximum Applied Load (kN) | X* (m) | Apparent Free Length $L_d$ (m) | Estimated Redundant Anchorage Length $L_r$ (m) |
|---|---|---|---|---|---|---|---|
| 12 | 11.5 | 4.5 | 7.0 | 1200.0 | 0.75 | 0.3 | 4.2 |
| 13 | 11.5 | 4.5 | 7.0 | 1200.0 | 0.75 | 0.23 | 4.27 |
| 14 | 11.5 | 4.5 | 7.0 | 1200.0 | 0.75 | 0.13 | 4.37 |
| 15 | 11.5 | 4.5 | 7.0 | 1200.0 | 0.75 | 0.19 | 4.31 |
| 19 | 11.5 | 4.5 | 7.0 | 1200.0 | 0.75 | 0.12 | 4.38 |
| 21 | 11.5 | 4.5 | 7.0 | 1200.0 | 0.75 | 0.2 | 4.3 |

* - X is the clearance between the top end of the fixed anchor and the point of intersection of the free anchor with the failure plane.

Apparent free lengths $L_d$ have been back-figured from the tensile test results using equation 3. Redundant anchorage length $L_r$ is given by;

$L_r = L_1 - L_d$ ---------- (4)

Estimates of $L_d$ vary between 0.12m – 0.3m. The value of $L_d$ gives an indication of the extent of debonding that occurs at the proximal end of the anchor when loaded. The BS8081 (1989) recommends that; $0.9L_1 \leq L_d \leq 1.1L_1$.However, test results presented in table 3 show that while estimated $L_d$ values are less than $1.1L_1$ in all cases, $L_d$ generally lies below $0.9L_1$ and consequently points to a significant level of redundancy in design i.e. apparent free lengths $L_d$ are considerably lower than design free lengths $L_1$. As shown in table 3, corresponding redundant anchorage lengths range between 4.2m – 4.4m.

A number of factors may have been responsible for the wide differences between design values and back-analysed results, these are summarised below:

- Adoption of a 45 degree failure wedge as typically done by many designers (e.g. Mansur & Alizadeth (1970), Clough & Tsui (1984) and Booth (1966)) may have been an overly conservative approach. Considering a representative frictional angle $\phi$' of the materials at the back of the wall, such that failure plane angle $\beta = (45 - \phi'/2)$ may have yielded less conservative, yet effective design solutions.
- 7m fixed length ($L_2$) in weathered granite (as adapted from relevant case histories in the BS8081 (1989)) used in design may have been quite conservative, leading to high factor of safety in design. A safe, effective and more economical solution could have been achieved with a shorter fixed anchor length.
- Ultimate bond stress $\tau_{ult}$ of 0.7 N/mm$^2$ adopted in design may have been quite low, also contributing to a high factor of safety in design. While $\tau_{ult}$ & $L_2$ adopted in design have been deduced from related case histories presented in the BS8081 (1989), rock strength and characteristics vary significantly across different geographical regions and hence, typical parameters and historical data presented in the BS8081 (1989) and other relevant publications in the literature may be considered to be only indicative. Such data should only be used in conjunction with the designer's knowledge of the local geology to achieve a reasonably safe and economical solution.
- Since the entire tendon length has been fully grouted during installation, skin friction mobilised in the boulder clay and the silty sandy gravel may have contributed to the tensile capacity of the anchor, such that actual free length turns out to be considerably lower than the design value.
- Due to slight variations in the depth of intact bedrock across the site, design had been based on the lowest rock head recorded during site investigation. Therefore, at some anchor locations, part of the anchor fixed length may have been located in intact granite and not in the weathered layer as assumed in design, such that actual grout-ground bond stress over a fraction of the anchor fixed length may have been higher than design values.

## Overall Wall Performance

The lateral deflection of the anchored secant pile wall has been monitored with inclinometers during and after excavation. Maximum values recorded at each construction stage are compared with original design estimates in table 4. Figure 8 also compares monitored deflections along the pile length after completion of excavation, with design predictions. From table 4 and figure 8, it is clear that measured wall deflections are considerably lower than design estimates (typically 9.2% – 11.5% of predicted values). After the completion of excavation to formation level, a maximum wall deflection of 2.3mm was recorded. The serviceability analysis of the wall had predicted a maximum deflection of 21.5mm. Hence

design predictions may be considered to be quite conservative. The service loads on the anchors may have been over-estimated in the wall serviceability analysis, leading to the adoption of higher anchor lengths than necessary. $K_o$ values for the materials had been significantly reduced to allow for piling – induced stress relief in the ground, while the soft piles haven't been considered in the estimation of the overall wall stiffness, although they are believed to influence the wall stiffness to a certain degree.

Table 4 – Comparison of monitored wall deflections with design estimates

| Stage No. | Description | Predicted wall deflection (mm) | Measured wall deflection (mm) |
|---|---|---|---|
| 2 | Install hard & soft piles | 0.0 | 0.0 |
| 3 | Excavate to 1.5m depth and construct RC capping beam | 3.5 | 0.4 |
| 4 | Install and prestress tie-back anchors | 6.0 | 0.55 |
| 5 | Complete excavation to basement formation level | 21.5 | 2.3 |

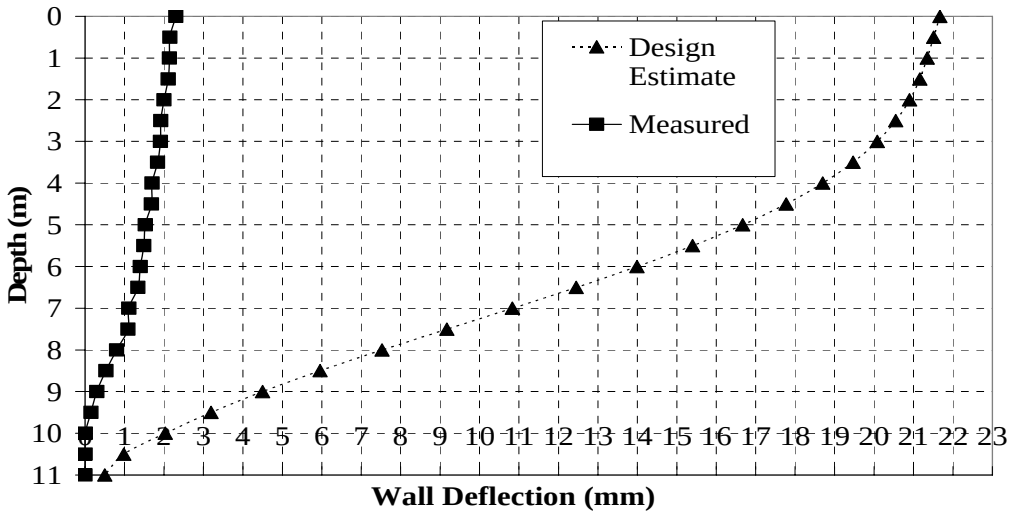



Figure 8 – Comparison of measured wall movement with design predictions.

Also, actual earth pressures acting behind the wall may have been much lower than assumed by the pseudo-finite element modelling software used in the analysis. As suggested by Ivor et al. (1996), Dublin glacial deposits (including the gravels) have the capability to behave as undrained materials and stand unsupported, even when subjected to loading. This has been attributed to negative excess pore pressures, cohesion and cementation of the soil grains. On the contrary, in the current retaining wall analysis, overburden materials were assumed to be drained; an approach taken after the CIRIA C580 (2003), which considers total stress analysis of embedded retaining walls in cohesive materials to be an approach associated with a certain degree of risk, especially in the permanent condition.

## Conclusions

The design, construction, testing and monitoring of an anchored retaining wall has been described. The results of the suitability and acceptance tests on some of the installed tie-back anchors have been presented; these have been back-analysed and compared with design

predictions. Monitored readings of wall deflection at different stages in construction have also been compared with estimates of the pseudo-finite element model used for the design.

Measured results have been found to be significantly lower than design estimates; likely factors responsible for this have been highlighted. While the majority of anchor design parameters (e.g. bond stresses, fixed anchor lengths) recommended in the literature seem to be sufficient for the safe design of anchors on this project, a more economical design could have been achieved if a sizeable database of published test results in similar ground conditions around the same locality had been available to the designers. Wherever possible, designers should place greater reliance on data from relevant local case histories in related ground conditions.

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