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AN INNOVATIVE USE OF BORED TENSION PILES IN EMBEDDED RETAINING WALL DESIGN & CONSTRUCTION

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The design, construction and monitoring of the deep excavation support for the basement of a large-scale commercial development in Cavan, Ireland is presented. Excavation area was adjacent to existing structures and retained height H varied between 4.0m – 11.0m. In areas where excavation depth H < 5.0m, wall was designed as cantilevered section. However in sections where 5.0m ≤ H ≤ 11m, additional wall supports were required for serviceability reasons. Traditionally, designers often provide additional wall support through the use of tie-back ground anchors, raking props and braces. On the current project, the main contractor was not in support of the use of temporary props so as to maximise working space, while owners of adjacent properties disapproved the use of tie-backs. The limited set-back between the wall line and site boundary also contributed to the complexity of the project.
A rare alternative solution has been adopted. This novel approach involved buttressing the wall with steeply raking bored tension piles at regular intervals along the wall. Comparison of the monitored deflections in the cantilevered sections with those recorded at sections where tension piles were used shows the novel solution to be safe and quite effective. This approach is also considered to be more economical when compared with ground anchorage and propping.

INTRODUCTION

The use of embedded retaining walls as supports for deep excavations has been a widely adopted practice in civil and geotechnical engineering. In suitable ground conditions, where excavation depths are relatively shallow (H ≤ 5.0m) and surcharge loads behind walls are negligible, retaining walls are typically designed as cantilevered structures. However, for deeper excavations (H > 5.0m) with considerable surcharge loads, additional supports are usually required for serviceability reasons.

Traditionally, such additional supports are provided through the use of tie-backs or steel props. Several publications on the design, construction, testing and monitoring of anchored and propped embedded retaining walls abound in the literature (Adekunte et al., 2007; Loveridge, 2001; Batten & Powrie, 2000a & 2000b; Richards et al., 1999; Twine & Roscoe, 1999; Batten, 1998; Carder et al., 1997; Potts & Bond, 1994; Symons et al., 1987 and Milligan, 1983). Except for a few recorded cases of failure (see Broms & Stille, 1976), workers have generally found the performance of anchored and braced embedded retaining walls to be quite satisfactory. However, very little attention has been paid to the development and application of alternative approaches, especially in situations where site constraints limit the application of tie-backs and props.

The current study is centred on the design, construction and monitoring of a contiguous pile retaining wall in overconsolidated clay. At sections of the wall where additional structural supports were required, an unconventional approach has been adopted. This involved the buttressing of the wall with raking CFA bored piles at regular intervals. Details of the design, installation and monitored performance of the wall are presented in the following sections. Observed performance is also compared with design predictions.

LOCAL GEOLOGY

The development is located in north central Ireland as illustrated in figure 1. Site geology chiefly comprises of glacial till, which is a mixed material containing gravels, cobbles, sand and clay-sized particles randomly mixed, resulting from the movement and subsequent deposition by glacier ice with little or no sorting by water. In Ireland, glacial tills were formed in the Pleistocene period, when the whole of Ireland was enveloped in ice. The ice gradually eroded the carboniferous limestones directly supporting it and subsequently resulted in the formation of glacial till. Hence, intact limestones are typically found beneath Irish glacial tills. Typically, Irish

glacial till is a very dense overconsolidated material of low permeability with occasional pockets of gravels, cobbles and boulders.



Figure 1 – Site location.

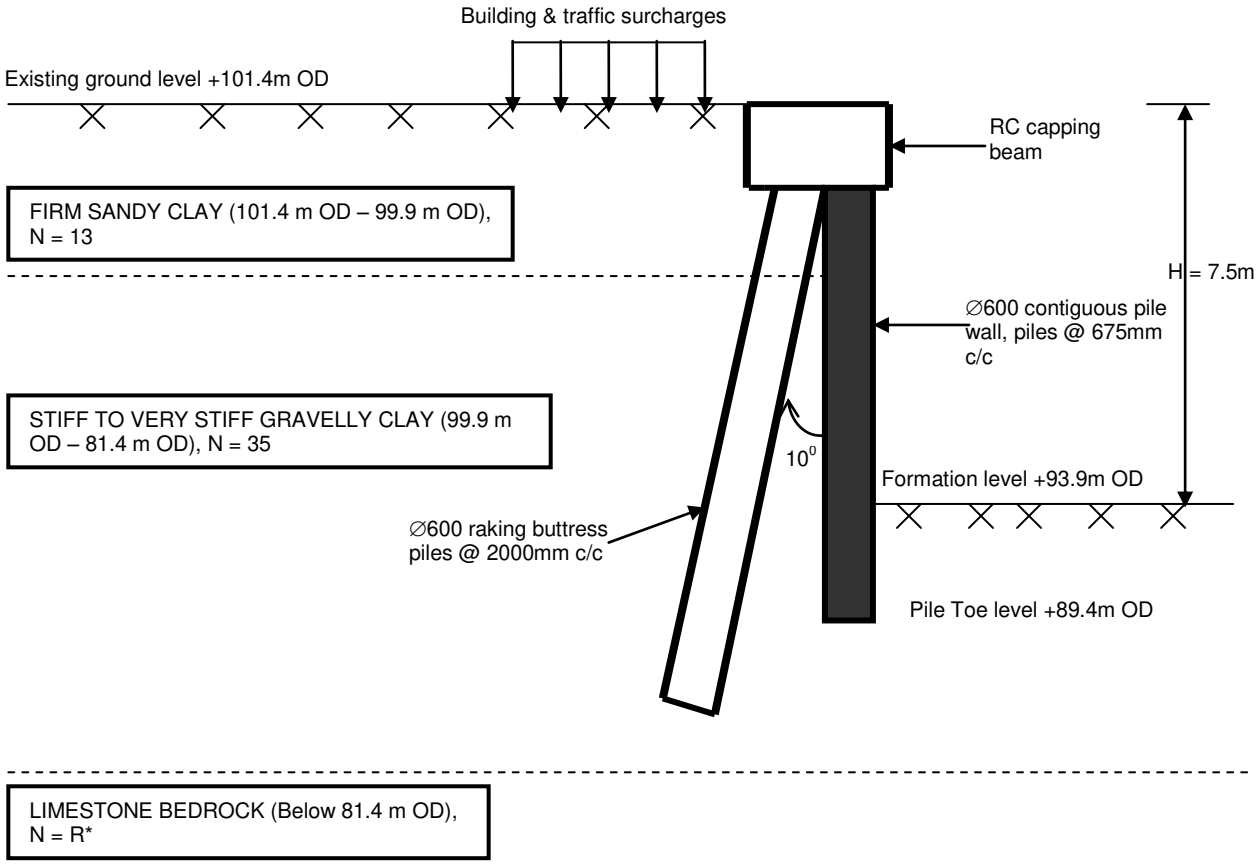
Site stratigraphy includes 1.5m of firm sandy clay. Below this stratum is 18.5m thick layer of stiff to very stiff gravelly clay, which is underlain by limestone bedrock. Rock head varies between 19m – 20m across the site. Figure 2 shows the generalised site stratigraphy in relation to the excavation support system. Static groundwater level at the site generally lies below the basement formation level at 14m depth. The glacial till at the site is of low to intermediate plasticity with liquid limit ranging between 17% - 42% while plastic limit varies between 14% - 25%. Particle size analysis shows the glacial till to comprise of 10% cobbles, 32% gravel, 30% sand, 16% silt and 12% clay size particles with an effective size D_{10} of 0.0015mm and a uniformity coefficient of 600. Thus the glacial till can be classified as well graded silty sandy gravelly clay with occasional cobbles. The material contains a significant percentage of fines, with more than 30% of a representative sample passing through the 63μm sieve. The high fraction of fines (especially silt) makes the glacial till subject to dilatancy. A comparison of the grading curve for the glacial till at the site with those of other samples obtained from other regions of Ireland (after Hanrahan, 1977) is shown in figure 3. The figure shows a remarkable level of consistency in the composition of Irish glacial till.

Being an overconsolidated material, the Irish glacial till tends to be expansive. The retaining wall construction process results in the unloading of the material, negative pore pressures are developed and consequently reductions in mean effective stress occur. As temporary negative pore pressures return to equilibrium, the glacial till softens. Soil parameters adopted in the design of the current project are presented in table 1. However, 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, this is to allow for the construction–induced remoulding and stress relief that occur in the glacial till, as described above.

For each angle of shearing resistance ϕ' presented in table 1, corresponding active and passive pressure coefficients K_a & K_p were estimated using Caquot & Kensel's (1984) chart. In line with CIRIA Report No. 104 (1984), maximum effective wall friction angle equivalent to $0.67\phi'$ and $0.5\phi'$ have been adopted for the active and passive zones respectively. In the overall stability analysis of the wall, design pressures have been based on yielding wall conditions, while in the wall serviceability analysis, design pressures were based on non-yielding wall conditions.

PROPOSED DEVELOPMENT

A large-scale retail development was proposed. The multi-storey development involved the construction of a single to triple level basement for underground parking and storage purposes. The site is adjacent to an existing hotel on the northwest boundary, while it is bounded on the southwest and southeast sides by existing office & industrial buildings. The Cavan – Dublin national road bounds the site at the northeast end.



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

Figure 2 – Site stratigraphy in relation to excavation support system in double-level basement area.

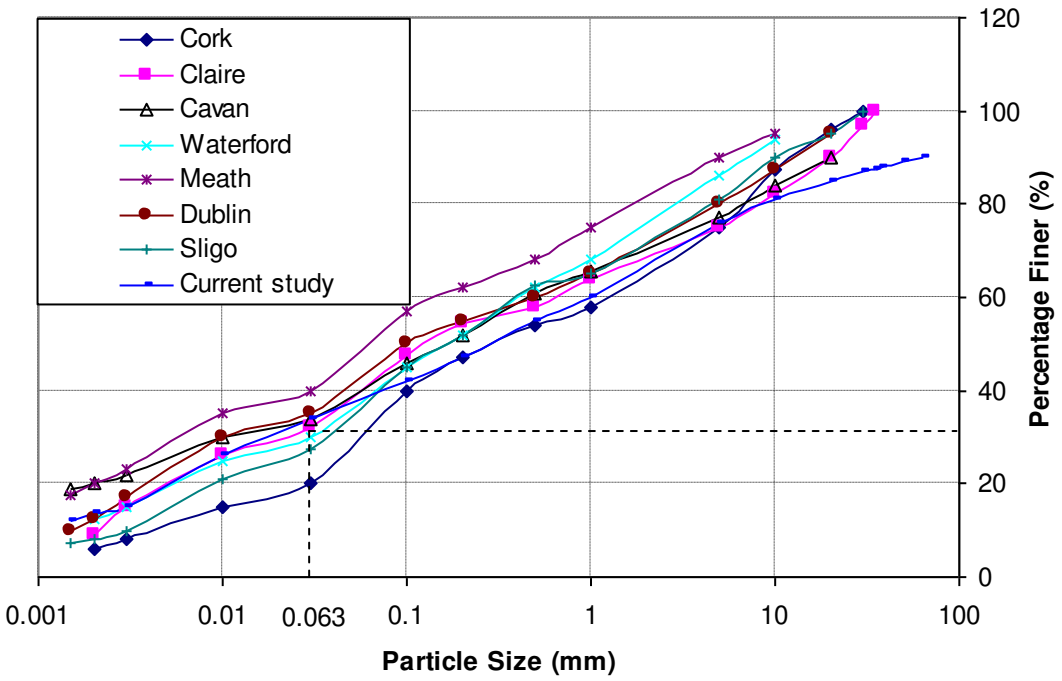


Figure 3 – Comparison of grading curves for Irish glacial tills (after Hanrahan, 1977)

Table 1 – Soil parameters adopted in design.

Stratum	Bulk density γ (kN/m^3)	Young's modulus E (kPa)	Angle of shearing resistance ϕ' (°)	Assumed coefficient of earth pressure K_o	Skin friction resistance f_s (kPa)
Firm sandy clay	18.0	32000	30.0	1.0	50.0
Stiff to very stiff gravelly clay	20.0	87000	37.0	1.0	110.0

Maximum excavation depth for the basement was 10.5m (3.5m above the static groundwater level at the site), while excavations for the pad footings to the main structure were locally made in the very stiff gravelly clay to a maximum depth of 1.5m below basement formation level. Since there were no requirements to control groundwater flow into the excavation, a contiguous pile retaining wall was considered to be the most economical and effective excavation support solution for the site. The contiguous pile wall comprised of Ø600 piles spaced at 675mm c/c. While the piles in the single level basement area could easily be designed as cantilevered members, wall sections in the double – triple level basement area required additional supports for serviceability reasons.

Due to the layout of the site, working space had to be maximised and hence, the option of restraining the wall with steel props could not be adopted. Also, the developers had difficulties in obtaining approval from owners of adjacent properties for the use of tie-back ground anchors. As an alternative solution, wall sections in the double – triple level basement area were buttressed with 10 degree – raking ∅600 piles at 2m intervals along the wall. These were rigidly connected to the contiguous pile wall with a reinforced concrete capping beam. While the raking buttress piles solely provided the required additional support in the temporary condition, the contiguous wall forms part of the permanent structure. It derives propping effects from the basement and ground floor slabs in the permanent state.

While it would have been more appropriate to model the glacial till as an undrained in the temporary condition and switching to drained parameters in the permanent condition, in the current design, the material has been conservatively modelled with effective stress parameters in both the temporary and permanent conditions. 50% wall relaxation was also considered in the modelling of the permanent conditions, this models the additional displacements and stresses that result from the long term reduction of the wall stiffness.

CONSTRUCTION PROCEDURE

The basement construction involved 6 major stages as shown in table 2. Similar sequence has been followed in the analysis and design of the excavation support system.

Contiguous Pile Wall Construction

Piles were constructed using continuous flight auger (CFA) drilling technique. Pile length varies from 9m in the cantilevered single level basement area to 17m in the buttressed double – triple level basement area. Guide frames were installed prior to drilling to allow for the accurate positioning of piles in accordance with proposed basement layout. Drilling was done with a Cassagrande Hutte 205 MP drilling rig, the auger stem of which could be tilted at angles of up to 45 degrees to the vertical plane. Pile installation involved no rock-socketing as piles were designed to be wholly embedded in the overburden layers. The ∅600 piles making up the contiguous pile wall were spaced at 675mm c/c, while the buttressing piles were spaced at 2m intervals behind the pile wall. All piles were cast with grade C35 concrete and reinforced with grade 500 steel. In order to limit disturbance to fresh concrete, the following sequence was followed during pile installation;

- install every second pile and allow a minimum of 24 hours for concrete to set;
- after 24 hours, install intermediate piles.

Excavation was carried out in two stages. The first stage involved excavation to 1m depth to allow for the construction of the reinforced

concrete capping beam. This was followed by excavation to the required formation level and subsequent construction of the basement/ground floor slabs and permanent walls. Figure 4 shows the capping beam during construction, while figure 5 shows the contiguous pile wall after completion of excavation.

WALL DESIGN

The design of the wall involved the following;

- overall stability analysis of the contiguous pile wall;
- serviceability analysis of the contiguous pile wall;
- structural design of the contiguous pile wall;
- geotechnical & structural design of raking tension piles;
    - shaft – ground bond capacity check;
    - tendon capacity check;
    - tendon – concrete bond capacity check.

Overall Stability Analysis

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. Effective stress parameters presented in table 1 were reduced with a factor of safety F = 1.25, in accordance with CIRIA Report No. 104 (1984). Building and traffic surcharges were also factored up with F = 1.5.

This analysis has been done with 'Oasys STAWAL' geotechnical limit equilibrium modelling software. Analysis shows that the required pile length in the buttressed double level basement area (retained height H $\cong$ 7.5m) is approximately 12m.

Serviceability Analysis

In the serviceability analysis, unfactored surcharge loads and soil parameters as presented in table 1 were used to estimate the lateral deflection of the contiguous pile wall, the bending moments and shear forces on the wall, as well as the service loads on the raking buttress piles behind the wall. This has been done with 'Oasys FREW' pseudo-finite element modelling software. Maximum lateral wall deflection in the buttressed double level basement area was predicted to be 13mm, while the service load on the raking buttress piles was estimated to be 505 kN/m run of wall.

Structural Design of Contiguous Pile Wall

Steel reinforcement for the contiguous pile wall was designed to resist the bending moments and shear forces estimated in the wall serviceability analysis as described earlier. Structural design was done to BS 8110 (1985) using 'Oasys ADSEC' structural modelling software. Piles in both the single level (cantilevered) and double level (buttressed) basement areas were designed to be reinforced with 6 No. ∅25 grade 500 steel and ∅10 links @ 200mm c/c.

Geotechnical & Structural Design of Raking Tension Piles

Service load on the raking buttress piles were estimated to be 505 kN/m run of wall. For 2m c/c spacing of buttress piles, this corresponds to a maximum service load of 1010 kN on a single raking pile. For each single tension pile, the following design checks were carried out;

- *Shaft – ground bond capacity;* typical section of the raking tension piles is shown in figure 6. The shaft free length was estimated by assuming a failure wedge with an angle equivalent to 45 - ϕ'/2. Skin friction within the free zone was ignored in the estimation of shaft-ground bond capacity. Ultimate skin friction resistances were derived from the corrected N_{spt} values measured in the relevant strata using the empirical correlation suggested by Poulos (1989). This is expressed as;

  $$f_s = \alpha + \beta * N_{spt} \text{ (kPa)} \qquad (1)$$

  where f_s = ultimate unit shaft resistance, N_{spt} = SPT penetration resistance, α and β are constants depending upon soil and pile type. For bored piles in cohesive materials, Poulos (1989) recommends values of $\alpha = 0$ and $\beta = 5.0$.

Table 2 – Stages in basement design and construction

Stage No.	Description of works
1	Initial conditions; in-situ soil properties and relevant surcharge loads
2	Install ∅600 piles @ 675mm c/c to form contiguous pile wall
3	In sections where proposed retained height H ≥ 5m, install ∅600 buttress piles inclined at 10° to the vertical plane, at 2m intervals behind contiguous pile wall
4	Excavate to 1m depth and construct RC capping beam to connect buttress piles to contiguous pile wall
5	Complete excavation to basement formation level
6	Construct basement/ground floor slabs and permanent RC retaining wall

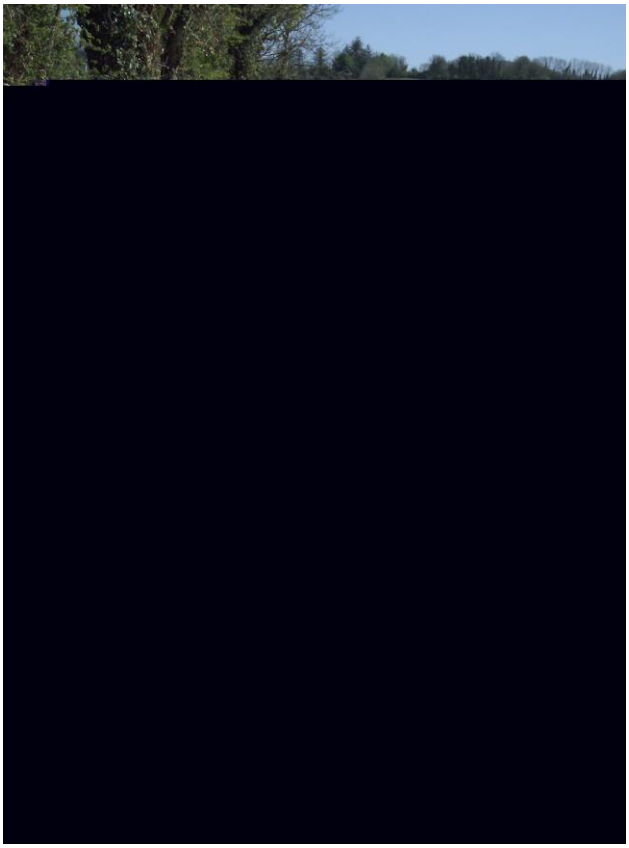


Figure 4 – Capping beam during construction.



Figure 5 – Wall after completion of excavation.

Equation 1 is more relevant to piles subject to compression. Previous experimental works (e.g. Chow, 2008) had shown that pile skin friction capacity in tension is approximately 80% of skin friction capacity in compression. This has been attributed to Poisson effects and rotation of the direction of the principal stresses. Hence, in the current design, capacity estimates made with equation 1 were reduced by 20% to arrive at the corresponding skin friction capacities in tension. Skin friction resistances f_s adopted in design are shown in table 1. For a Ø600 pile and a factor of safety F = 3.0, the required length L of buttress pile was estimated to be 15m.

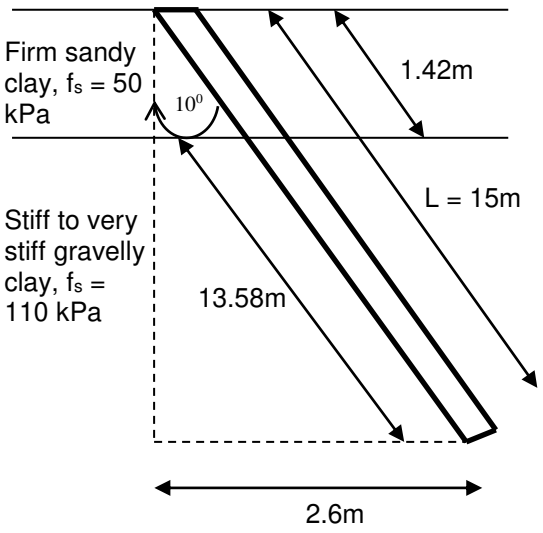


Figure 6 – Typical section of raking buttress piles

- *Tendon capacity;* using a factor of safety F = 2.0 for permanent tendons (after BS8081, 1989), 9 No. 25mm (2") dia. grade 500 deformed steel bars were recommended as tension reinforcement. These were installed over the pile full length.

- *Tendon – concrete bond capacity;* for an assumed bar/grout bond stress of 2.0 N/mm^2 and factor of safety F = 3.0 (after BS 8081, 1989) on 25mm (2") dia. tendons, design was found to be satisfactory.

WALL MONITORING

To monitor the wall movement during and after excavation, 6 No. inclinometers were installed at selected points along the wall. These were installed after pile installation and baseline readings were recorded just before the start of excavation. Measurements were taken at intervals of 7 working days during excavation and fortnightly after completion of excavation until basement and ground floor slabs were constructed. One additional set of readings was taken after basement slab construction. Monitoring was done over a period of 5 months after pile installation.

EXPERIMENTAL RESULTS & DISCUSSION

Overall Wall Performance

As described above, the lateral deflection of both the cantilevered and buttressed sections of the contiguous pile wall has been monitored with inclinometers during and after excavation. Maximum wall deflections recorded after completion of excavation in the single level basement area (cantilevered, H = 4m) and the double level basement area (buttressed, H = 7.5m) are compared with original design estimates in figures 7a and 7b respectively. A comparison of the wall movement in the cantilevered (H = 4m) and buttressed (H = 7.5m) sections is also shown in figure 8.

From figures 7(a) and 7(b), it is clear that measured maximum wall deflection is considerably lower than design estimate in both the cantilevered and buttressed sections of the wall. Measured maximum deflection is 55% and 71% of predicted value in the cantilevered and buttressed sections respectively. Therefore, design predictions may be considered to be quite conservative.

The actual earth pressures acting behind the wall may have been much lower than assumed by the pseudo-finite element software used in the serviceability analysis. Also, as shown in table 1, the glacial till has been modelled as a drained material; this approach has been taken after CIRIA Report No. C580 (2003), which considers total stress analysis of embedded retaining walls in clays as an approach associated with a certain level of risk, especially in the permanent condition. However, Ivor et al. (1996) have suggested that Irish glacial tills have the potential to behave as undrained materials and stand unsupported, even when subjected to loading. They have attributed this to negative excess pore pressures, cohesion and cementation of the soil grains.

Figure 8 shows that despite the retained height H in the buttressed section (H = 7.5m) being higher than that of the cantilevered section (H = 4m), the recorded wall movement is much lower than that measured in the cantilevered section. A maximum lateral deflection of 7mm was recorded in the buttressed section, while the maximum deflection in the cantilevered section was 12mm.

Most importantly, this observation points to the effectiveness of the raking tension piles at limiting the wall deflection, as well as maintaining overall stability in the double level basement area. Hence, this alternative approach of buttressing the contiguous pile wall with tension piles may be considered satisfactory from a design and construction perspective.

In addition, contractors and developers may find this novel approach to be more attractive than conventional propping and ground anchorage. In comparison with steel propping, it offers a larger working space within the excavation area. Also, it is a more economical solution when compared with tie-back anchorage; most ground anchorage projects require the placement of special orders for tendons, which are generally considered to be relatively expensive. In contrast, the novel solution of using bored tension piles only requires concrete and ordinary high yield reinforcement bars, which are cheaper and more readily available to the contractor. Unlike tie-backs, with the new approach, intrusion into third party properties may be avoided during construction.

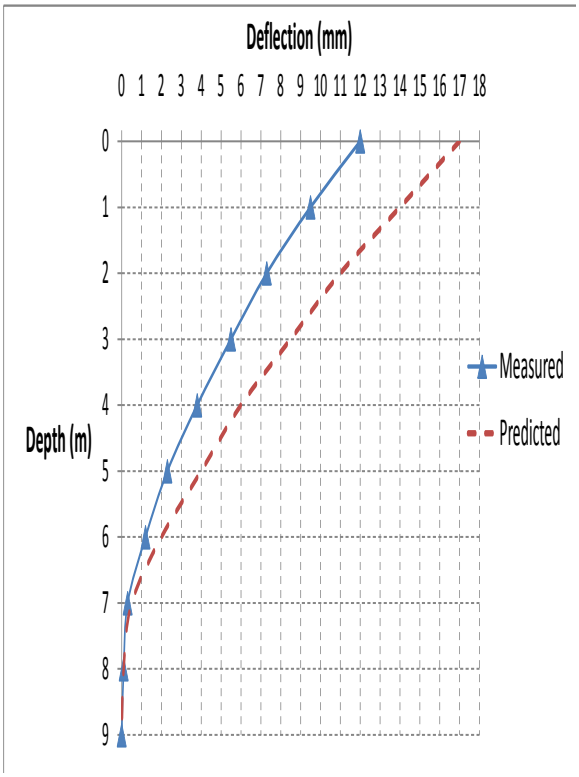


Figure 7(a) – Comparison of monitored wall deflection with design prediction in single level basement area (cantilevered).

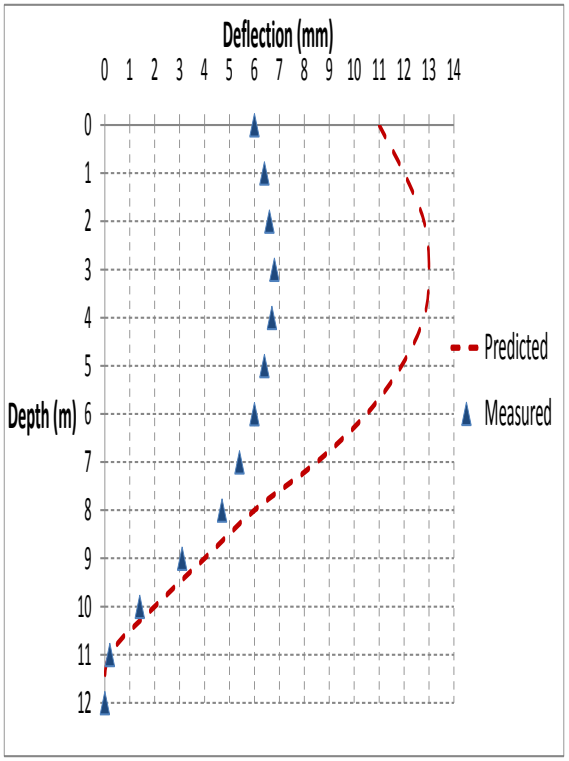


Figure 7(b) - Comparison of monitored wall deflection with design prediction in double level basement area (buttressed).

CONCLUSIONS

The design, construction and monitoring of an embedded retaining wall has been presented. A novel approach has been adopted in providing additional supports to the wall in sections where it could not be cantilevered. The unconventional solution involved buttressing the wall with steeply raking bored tension piles at regular intervals.

Wall movement in the cantilevered and buttressed sections during and after construction was monitored over a 5 – month period. Measurements have been compared with design predictions, while comparisons have also been made between wall deflections in the buttressed and cantilevered sections.

Monitored wall movements have been found to be considerably lower than design estimates; possible reasons for this have been highlighted. Results also show lateral deflections in the buttressed sections to be much lower than those recorded in the cantilevered sections regardless of the fact that retained heights in the

cantilevered areas were much lower, suggesting the satisfactory performance of the novel solution.

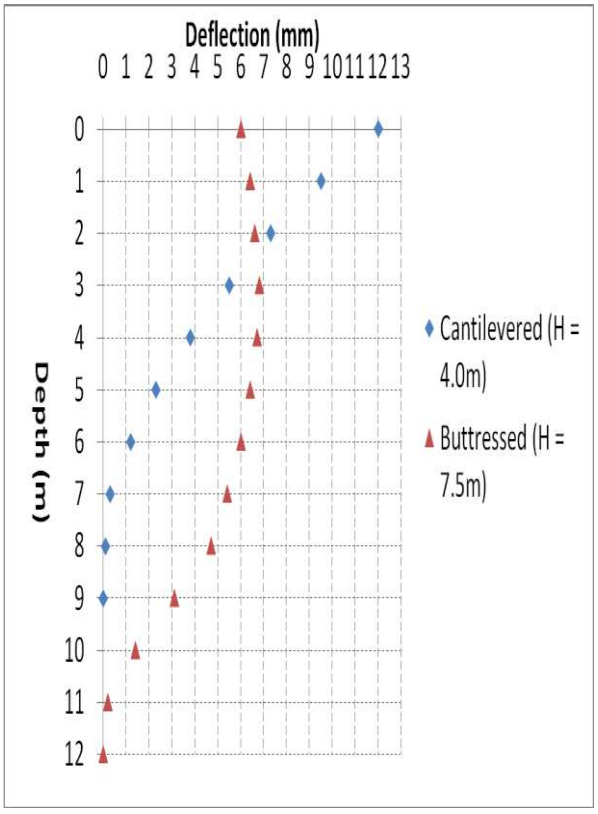


Figure 8 – Comparison of wall movement in the cantilevered and buttressed sections.

Most contractors and developers would find this unconventional approach to be more attractive than the traditional propping and tie-back anchorage because it is more economical in terms of material costs and availability, while it also allows for the maximisation of working space on construction sites.

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