Visible means of restraint

Abid Adekunte, Pat O' Hara and Austin Dennany report on novel ways of restraining embedded retaining walls

Finished wall on site 1

In the current global economic climate, it is imperative that construction specialists offer competitive solutions to clients that are innovative, economical and safe. In line with this notion, many geotechnical specialists now consider innovation and economy to be paramount in design and construction.

Traditionally, designers and contractors provide additional external supports to retaining walls through the use of tie-back anchors or braces. Both approaches have shortcomings: braces limit working space in excavation areas, while tie-backs are seldom used in built-up areas where intrusion into adjacent properties must be avoided or where sensitive structures exist behind the wall.

In recent years, these restrictions have prompted a number of companies to focus on the development and application of alternative restraint methods. This article looks at the design, construction and monitoring of contiguous pile retaining walls on two sites with different ground conditions. On both projects, site constraints have led to an innovative approach.

Site geology

Both developments are located in the Republic of Ireland: site 1 lies on the east coast while site 2 is in the northern-central region.

The stratigraphy at site 1 comprises made ground, overlying medium-density gravel, which is underlain by hard, glacial till. The bedrock lies at a depth of 27m and mainly comprises competent limestone, interbedded with thin layers of mudstone.

The geology at site 2 chiefly comprises very stiff glacial till, overlying limestone bedrock at a depth of 20m. The glacial till contains a random mixture of gravel, cobbles, sand and clay sized particles, resulting from the movement and subsequent deposition of material by glacial ice, with little or no sorting by water. It is, essentially, a very dense, consolidated material of low permeability with occasional pockets of gravels, cobbles and boulders.

The glacial tills on both sites are of low to intermediate plasticity, with liquid limits of 14-42%, while plasticity limits vary from 10-25%. The static groundwater level on site 1 lies at a depth of 4m whereas it is 14m below the surface on site 2.

In order to allow for the influence of remoulding and stress relief that occurs in soil during construction, the co-efficients of earth pressure (Ko) for the over-consolidated materials were limited to 1.0, in accordance with CIRIA Report C580.

In the geotechnical design of the site 1 wall, the glacial till was modelled with undrained parameters in the temporary condition, while the permanent condition was modelled with effective stress parameters.

The site 2 wall was designed with effective stress parameters for glacial till in both temporary and permanent conditions.

Proposal: site 1

The project at site 1 was centred on the upgrade of a roll-on, roll-off (Ro-Ro) ramp at Dublin Port, as part of the ongoing regeneration programme undertaken by the Dublin Docklands Authority. It proposed the construction of a new Ro-Ro floating ramp to cater for cruise liners of variable widths entering the city. ►

Site 1 wall during construction

Van Elle was appointed as the specialist geotechnical contractor for the project. The scope of the geotechnical works included the design and construction of a contiguous pile wall with a retained height of up to 7m to accommodate the proposed link span bridge, as well as heavily loaded foundation piles to support compression, tension, moment and lateral loading from the new bank seat.

All of the wall sections were originally required to be designed as free-standing cantilevers in both temporary and permanent conditions. However, in the 6-7m excavation area, the presence of pre-existing caisson wall bases at a depth of 11m rendered a cantilevered solution inapplicable. Hence, this section of wall required permanent restraint.

"The project at site 1 was centred on the upgrade of a roll-on, roll-off (Ro-Ro) ramp at Dublin Port"

Tie-back anchors could not be used as there was an existing underground service culvert 3m behind the wall line, while access for an anchoring rig was also a problem. Temporary propping/bracing was not an option as the working space in front of the wall had to be maximised, and the proposed scheme did not allow for the construction of any form of permanent restraining structure in front of the wall.

Therefore an alternative solution was developed, involving the simultaneous excavation of both sides of the contiguous wall to a depth of 6-7m and constructing a 2.5m-wide, concrete counterweight structure behind the wall.

Proposal: site 2

The site 2 project was a multi-storey, commercial develop-ment with a single to triple level basement for underground parking and storage purposes. The site is bounded by existing properties and a busy main road. The scheme required the construction of contiguous pile walls to enable the excavation and construction of the basement. The retained height varied from 4-10.5m.

While the wall sections in the single-level basement area could be designed as free-standing, cantilevered structures, the wall sections in the double to triple level basement area required additional supports for serviceability.

The working space in the excavation area had to be maximised, so conventional propping was unapplicable. In addition, the developers had difficulties in gaining approval from the owners of adjacent properties for the use of tie-back anchors, so a wall-restraining method was adopted. This involved buttressing the wall with 10° raking piles, spaced at 2m intervals behind the pile wall. The buttress piles were connected to the contiguous pile wall with a reinforced-concrete capping beam.

While the required wall restraint was provided temporarily by the raking buttress piles, the basement and ground-floor slabs offered additional propping to the wall in its permanent state.

Construction

At site 1, a Soilmec CM48 hydraulic piling rig was used to install piles using CFA drilling. The 600mm-diameter piles were installed at 750mm c/c (centre to centre) spacing and cast with grade 35 concrete and reinforced-grade 500 steel to form the contiguous pile wall. After the wall had been installed, initial excavation was carried out in front and behind it to a depth of 750mm, to allow for the construction of the reinforced-concrete capping beam. This was followed by the simultaneous excavation of both sides of the wall to formation level.

Behind the wall, excavation was only carried out over a 2.5m-wide area, while temporary shoring and formworks were installed to allow for the construction of the permanent counterweight structure behind the wall. This was built to perform two main functions:

- provide additional gravitational support/ restoring moment to the contiguous pile wall;
- provide extra stiffness/flexural rigidity to the pile wall above formation level.

Concrete for the counterweight structure was poured in three stages over a three-day period. Rapid-hardening cement was used in order to limit active pressure from wet concrete placed behind the wall. Dowel bars were installed at capping-beam level, mid-height and just above formation level. The dowel bars were installed at every pile position to tie

Contiguous pile wall at site 2

the counterweight structure to the piles as it was poured in stages.

For safety reasons, the reinforcement for the counterweight was designed to allow the cages to be tied and lowered into the excavation behind the pile wall, rather than requiring the steel fixers to enter the deep excavation.

Following excavation and construction of the counterweight structure, the permanent reinforced-concrete wall was built in front of the contiguous pile wall, with the associated drainage system.

As the watertable was located at 4m depth in the sandy clay stratum, which is of low permeability, minimal groundwater control was required during excavation. The static groundwater level remained relatively constant.

CFA drilling was also used for the piles on site 2. The length of the piles used for the walls varies from 9m in the free-standing, cantilevered single-level basement area to 17m in the buttressed, double-triple level basement area. Drilling was executed with a Casagrande Hutte 205 MP drilling rig, with an auger stem that could be tilted to an angle of up to 45º. The 600mm-diameter piles were spaced at 675mm c/c to form the contiguous pile wall. The raking buttress piles, also 600mm in diameter, were spaced at 2m intervals behind the contiguous wall. The weight of the buttress piles gave extra gravitational support to the wall, and initial deflection of the wall at the start of excavation mobilised skin friction around the buttress piles. This resistance helped to limit deflection.

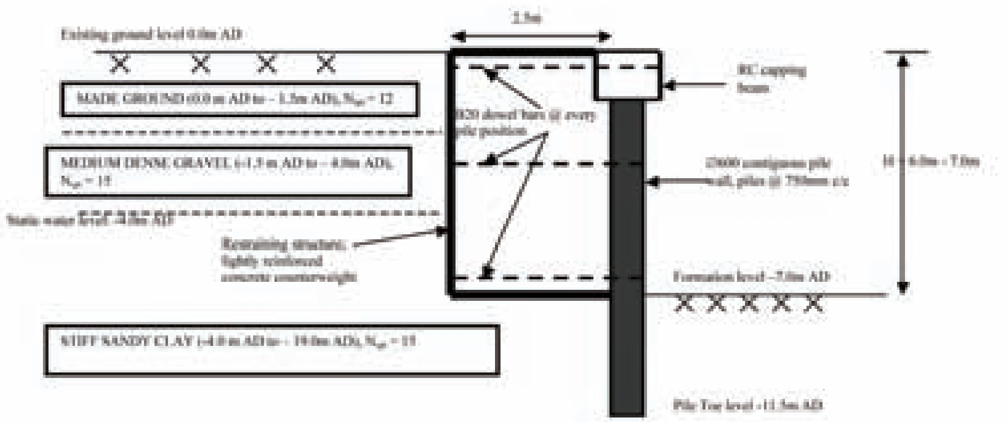
Excavation was carried out in two stages. Initial excavation was undertaken on both sides of the wall to a depth of 1m to allow for the construction of the reinforced-concrete capping beam, which ties the buttress piles to the contiguous wall. This was followed by excavation in front of the wall to formation level, and subsequent construction of the basement and ground-floor slabs, as well as the permanent, reinforced-concrete wall. As the ground-water level was well below excavation level, groundwater control was not required during construction.

Wall design

The wall on site 1 was designed to be restrained by a permanent, counterweight structure that was dowelled to the wall at three levels. Wall analysis and design involved the following:

- Overall stability analysis of contiguous pile wall/counterweight structure;
- Serviceability analysis of contiguous pile wall/counterweight structure; and
- Structural design of contiguous pile wall, capping beam and counterweight structure.

The glacial till was modelled as an undrained material in the temporary condition, while it was assumed to exhibit drained behaviour in the permanent state. Wall relaxation of up to 50% was also considered in the modelling of the permanent conditions to allow for the additional displacement and stress that results from the long-term reduction in wall stiffness. ►



Site 1 stratigraphy in relation to excavation support system on the site

Site 1: geotechnical design parameters

Stratum	Bulk density γ (kN/m^3)	Young's modulus E (kPa)	Undrained strength C_u (kPa)	Angle of shearing resistance Φ' (°)	Assumed co-efficient of earth pressure K_o
Made ground	18.0	30,000	-	30.0	0.5
Medium dense gravel	19.0	40,000	-	32.0	1.0
Stiff, sandy clay	19.0	40,000	75.0	32.0	1.0

Site 2: geotechnical design parameters

Stratum	Bulk density γ (kN/m^3)	Young's modulus E (kPa)	Angle of shearing resistance Φ' (°)	Assumed co-efficient of earth pressure K_o	Skin friction resistance f_s (kPa)
Firm, sandy clay	18.0	32,000	30.0	1.0	50.0
Stiff to very stiff, gravelly clay	20.0	87,000	37.0	1.0	110.0

In the overall stability analysis, an ultimate limit state approach was adopted using factored soil parameters and loads to estimate the required embedment depth of the wall for overall stability.

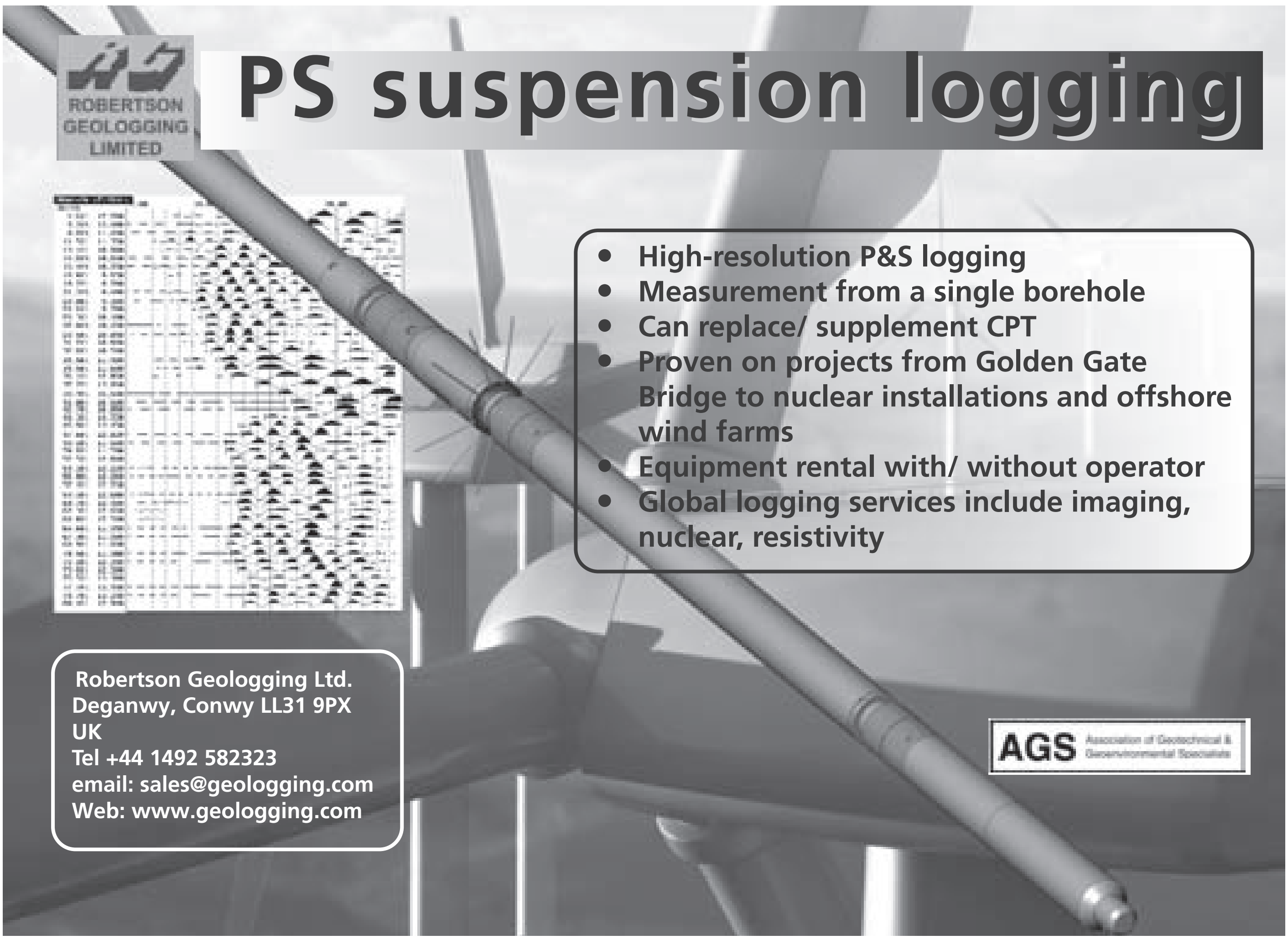
Effective stress parameters were reduced with a factor of safety of F = 1.25, while the undrained parameters were reduced with a factor of safety of F = 1.5 in accordance with CIRIA Report C580: recommendations for moderately conservative design approach. Surcharge loads were also factored up with F = 1.5.

The analysis was undertaken using geotechnical limit equilibrium-modelling software. It showed the required pile length to be 11.5m, which corresponds to a minimum embedment of 4.5m below formation level. In the serviceability analysis, unfactored surcharge loads and soil parameters were used to estimate the lateral deflection of the pile wall, as well as the bending moments and shear forces exerted on it. This was performed using pseudo-finite element modelling software. Maximum lateral wall deflection in the temporary condition was predicted to be 22mm.

Steel reinforcements were designed to support the bending moments and shear forces estimated through the wall serviceability analysis. The piles were designed to be reinforced with high-yield, steel-reinforced bars (characteristic yield strength = 500 N/mm^2) and 10mm-diameter links spaced at 320mm c/c over the full depth of the pile.

The contiguous pile wall on site 2 was designed to be buttressed by raking bored-tension piles connected to the wall at capping-beam level. Analysis and design 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 and structural design of the raking tension piles;



- Shaft – ground bond capacity check;
- Tendon capacity check; and
- Tendon – concrete bond capacity check.

Analysis showed that the wall for the buttressed, double-level basement area (retained height 7.5m) needed to be 12m long, which corresponds to a minimum embedment of 4.5m below formation level. The buttress piles were designed to be 15m long and were reinforced with nine steel bars over the full length.

Monitoring

The wall-monitoring schemes used at the sites differed. While wall movement on site 2 was monitored with six inclinometers installed in selected piles, wall deflection on site 1 was judged through optical surveying.

The procedures involved in wall monitoring are outlined below.

Site 1

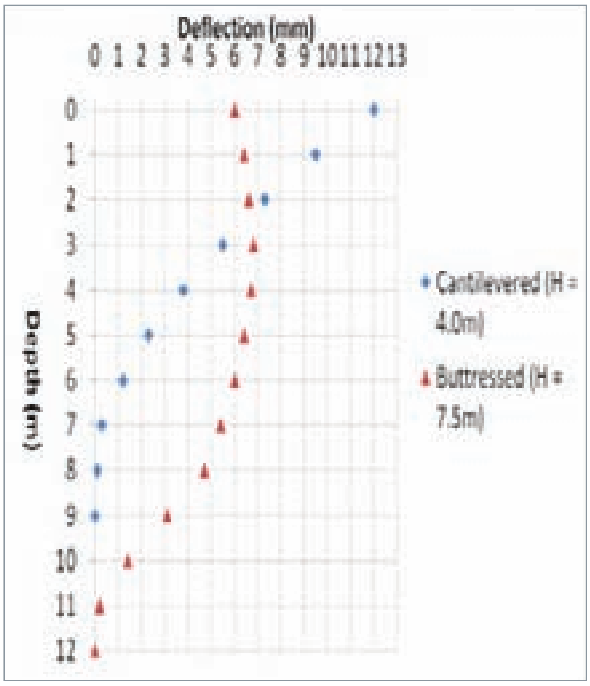
- After pile installation and capping-beam construction, shot fix nails were installed as targets at selected positions on the capping beam;
- Initial co-ordinates of targets were recorded using the survey instrument;
- After recording the initial co-ordinates, readings were taken at one-day intervals during, and every two weeks after, excavation; and
- Monitoring extended for eight months after pile installation.

Site 1: construction sequence

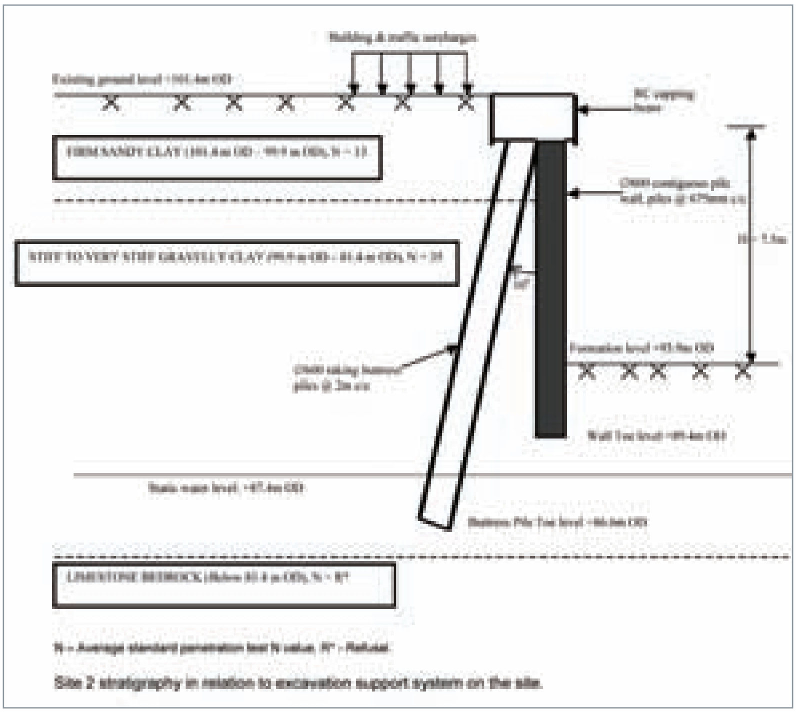
Stage	Description of works
1	Install 600mm-diameter, CFA bored piles at 750mm c/c to form contiguous pile wall
2	Excavate both sides of wall to 750mm depth and construct RC capping beam
3	Continue excavation on both sides of wall to formation level, while installing temporary shoring/formwork for counterweight structure behind wall
4	Backfill back of wall with first layer of concrete (2m thick) and dowel to contiguous pile wall
5	After 24h, backfill back of wall with second layer of concrete (2m thick) and dowel to contiguous pile wall
6	After another 24 hours, backfill back of wall with third layer of concrete (2m-3m thickness) and dowel to capping beam
7	Permanent conditions: in wall analysis, design parameters for glacial till are switched from undrained to drained, while 50% wall relaxation is also accounted for

Site 2: construction sequence

Stage	Description of works
1	Install 600mm-diameter piles at 675mm c/c to form contiguous pile wall
2	In sections where proposed, retained height exceeds 5m, install 600mm-diameter buttress piles 10° to the vertical plane at 2m intervals behind contiguous pile wall
3	Excavate to 1m deep and construct RC capping beam to connect buttress piles to contiguous pile wall
4	Complete excavation to basement formation level
5	Construct basement/ground floor slabs and permanent RC retaining wall



Site 2 comparison of deflections in the free standing cantilevered section with wall movement in the buttressed area



Site 2 stratigraphy in relation to excavation support system on the site

Site 2

- During pile installation, inclinometer casings were installed in selected piles;
- After installation, traversing inclinometer probes were lowered down into the casings to record baseline readings;
- Readings were then taken at one-day intervals during excavation and every two weeks after its completion. Only one extra reading was taken after basement construction as no significant movement was recorded once the permanent slabs were put in place; and
- Monitoring was undertaken for six months after pile installation.

> "Monitoring results have shown that the novel wall restraint methods used on both sites have been effective"

Results

At site 1, wall-monitoring data showed that approximately 60% of movement occurred during the construction of the counterweight structure. This is attributable to the active thrusts mobilised behind the wall as wet concrete was poured to form the counterweight structure.

The predicted wall deflections for this stage of construction were typically 27-38% of the estimated values. At the design stage, the wet concrete had been modelled as a loose, cohesionless material with effective stress parameters. However, the rapid-hardening cement used may have allowed the concrete to behave as an undrained, cohesive material, significantly reducing active pressures to values much lower than design estimates.

After the counterweight structure had been installed, an additional movement of 5mm was recorded over a period of seven months. Observations between the fifth and seventh month following counterweight construction showed no significant wall movement.

Overall, a maximum wall deflection of 12mm was recorded over an eight-month period. This falls below the maximum allowable movement of 25mm specified by the supervising engineers for the project. It also shows the effectiveness of the restraining scheme in terms of serviceability and overall stability.

Observations of wall movement on site 2 have also shown the design estimates of wall deflection to be conservative, which may be attributed to the numerical modelling of the glacial till at the design stage. The till had been modelled as a drained material with effective stress parameters. However, it is a possibility that the actual active pressures mobilised behind the wall may have been overestimated in the serviceability analysis.

Despite the retained height in the buttressed section being higher than that of the free-standing, cantilevered section, the measured wall deflections in the buttressed section are much lower than the wall movements in the free-standing, cantilevered area. Maximum lateral deflection in the buttressed section was 7mm, while it was 12mm in the free-standing cantilevered section. These observations show the unconventional wall restraint method to be quite effective at limiting wall movement, while maintaining overall stability.

Conclusions

In general, monitoring results have shown that the novel wall-restraint methods used on both sites have been effective, and thus offer useful alternatives to contractors and designers where conventional methods are inapplicable.

When compared with propping and bracing, both methods allow for the maximisation of working space in front of the wall and also prevent intrusion into third-party properties adjacent to site boundaries. Unlike props and anchors, both solutions also provide additional gravitational support to the wall.

Ground-anchorage projects typically require the placement of special orders for tendons, couplers, plates, nuts, spacers and other accessories, which are relatively expensive and associated with longer fabrication and delivery times when compared with ordinary reinforcement bars. In contrast, employing raking buttress piles only requires the use of ordinary, high-yield reinforcement bars and concrete, which are cheaper and more readily available to the contractor.

Taken from A Adekunte, P O'Hara, and A Dennany (2010). Novel Methods of Restraining Embedded Retaining Walls. Proceedings of the Deep Foundations Institute's 35th Annual Conference on Deep Foundations, Hollywood, US, October 2010