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# NON-CONVENTIONAL PRACTICE: RECENT CASE HISTORIES OF THE USE OF VERTICAL BUTTRESS PILES IN DEEP EXCAVATION SUPPORT

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## ABSTRACT

A number of novel solutions have been proposed in recent years as alternatives to props and tie-backs in deep excavation support e.g. Adekunte (2008), Deschamps et. al. (2008), Adekunte et al. (2010) and Adekunte (2014). A number of these non-standard restraining solutions involve the construction of permanent or temporary vertical or raking buttress piles that are rigidly connected to the excavation support wall at regular intervals along the wall run.

This paper chiefly focuses on the design and construction of twin-vertical buttress piles as restraining systems behind or in front of embedded retaining walls. The system essentially relies on the push-pull mechanism that develops between the twin piles, to maintain the overall stability of the excavation support. The paper's main areas of concentration include; finite element modelling, geostructural design, construction procedure, instrumentation and monitoring that are associated with the application of twin-buttress pile systems. These are supported with project details of three recently completed high-end luxurious residential developments in Greater London, England, on which the twin-buttress pile system was used to restrain the excavation support walls for the basements on the sites.

The mode of application of the system varies across the three sites. While the system was adopted as a long term solution behind the retaining wall on one of the sites, it was designed and constructed for temporary use in front of the retaining walls on the other two sites. Instrumentation and monitoring results from all the sites show the non-traditional system to be safe and effective, whilst having significant beneficial effects on construction programme, sequence of works and project cost.

**Keywords: excavation support, retaining wall, buttress piles, push-pull mechanism, finite element modelling, geostructural design, instrumentation, monitoring.**

## INTRODUCTION

It is becoming increasingly difficult to restrain bored pile retaining walls in urban areas with the well-established traditional systems; props and tie-backs. Due to inadequate space in built-up areas, many urban construction sites are tight and as contractors are compelled to maximise working space, they often prefer to avoid the use of temporary props and braces. In addition, the existence of neighbouring buildings and sensitive underground services in built-up areas often makes the application of ground anchors difficult.

For the above reasons, the use of twin buttress pile systems (TBPs) is increasingly becoming popular on urban sites, as suitable alternatives to conventional props and ground anchors. This alternative solution essentially involves the construction of twin vertical bored piles at right angle to the main retaining wall line, at regular intervals along the wall. The TBPs are connected to the main pile retaining wall by reinforced concrete capping beams. If the TBPs are only required for temporary use, they may be installed in front of

the pile wall (i.e. within proposed excavation area), such that they may be trimmed off once the permanent reinforced concrete slabs in front of the pile wall achieve sufficient structural strength. For TBPs to be adopted as permanent solutions, they need to be installed behind the retaining wall (i.e. outside the excavation/basement area), where they would be out-of-sight and non-obstructive in both temporary and permanent conditions.

The choice of suitable buttress pile size and spacing is typically an iterative process, with the underlying aim of striking a reasonably good balance between efficiency and economy. However, it is advisable to adopt a buttress pile size that is equivalent to the diameter of the piles that make up the main pile retaining wall, so as to limit differential behaviour between the pile wall and the TBP systems. In addition, the gap between the two piles in a TBP system must be at least 3D (where D is pile diameter), so as to limit the interaction between the twin piles, especially below excavation level.

This paper chiefly focuses on the design and construction aspects of three separate sites on which TBPs have been adopted as temporary and/or permanent restraining systems. The paper accounts for the associated numerical modelling procedures, geotechnical/geostructural design approaches, construction procedures and performance monitoring.

## GROUND CONDITIONS & MODELLING

This study focuses on three separate sites; site 1, which is located in Chiswick, West London, site 2, which is located in Hadley Wood, Hertfordshire and site 3, which is located in Virginia Water, Surrey. Stratigraphy on site 1 comprises of 2m thick made ground underlain by a 7m thick layer of medium dense to dense river terrace deposit, which in turn overlies a stiff to very stiff London clay stratum that extends down to the limit of ground investigation at 20m depth.

The river terrace deposit is chiefly made up of pebble-gravel and sand. These were deposited 500,000 years ago when the River Thames was diverted from its original course to its current channel. The London clay stratum is essentially an overconsolidated material of low permeability and high plasticity, with measured plasticity indices ranging between 40%-50%. Monitored highest static groundwater level over the winter months was 4m below ground level (bgl).

Stratigraphy on site 2 mainly comprises of stiff to very stiff London clay. Ground conditions on site 3 comprise of 2m of made ground overlying alternating layers of very soft to soft to firm sandy clay and loose to medium dense sand, which are underlain by stiff to very stiff sandy clay at 8m depth. Measured overconsolidation ratio (OCR) of samples of the stiff to very stiff clay reduces from 60 at 8m depth to 20 at 20m depth. Measured coefficient of earth pressure $K_o$ varies between 1.5-2.5. Generalised stratigraphy and associated geotechnical design parameters for sites 1, 2 & 3 are presented in tables 1, 2 & 3 respectively. In tables 1, 2 & 3, $N_{spt}$ = standard penetration test N value, z = depth below top of stratum.

In order to allow for the influence of stress relief and remoulding during excavation and basement construction, coefficients of earth pressure $K_o$ for the overconsolidated materials were limited to 1.0, in accordance with the recommendations of the CIRIA report No. C580 (2003). The clay layers were modelled with total stress parameters in the temporary condition. In the permanent condition, the clay layers were assumed to exhibit drained behaviour and thus modelled with effective stress parameters. All other soil layers were conservatively assumed to exhibit drained behaviour in both temporary and permanent conditions and thus modelled with effective stress parameters only.

Coefficients of active and passive pressure $K_a$ and $K_p$ were deduced from Caquot & Kensel's (1984) chart. Effective wall friction angles $\delta$ on the passive and active sides were generally taken to be $0.5\phi'$ and $0.67\phi'$ respectively (where $\phi'$ is the effective angle of shearing resistance).

## SITE 1 (CHISWICK, WEST LONDON)

**Proposed Development:** the project on site 1 was centred on the redevelopment of the pre-existing surface car park on the site into 6 units of town houses, with an underground car park spanning across the entire site area. It was proposed to support the 4m deep excavation for the basement car park with a contiguous bored pile retaining wall. The pile retaining wall was also required to function as a permanent component of the finished basement structure. Typical section of the earth retention system on site 1 is shown in figure 1. The TBPs were only required for temporary use and therefore, they were designed and installed in front of the pile wall (within proposed excavation/basement area), while they were subsequently trimmed down to depths below basement level once the basement slab and ground floor slab were constructed.

**Geotechnical/Geostructural Design:** the design for the deep excavation support system involved 4 main stages, which are described below.

- Ultimate Limit State (ULS) Analysis: this essentially involved the use of factored soil parameters to estimate the required embedment of the pile retaining wall, for overall stability to be maintained, whilst considering the pile wall to be restrained at capping beam level. The analysis was carried out with CADS PWS 5.6 modelling programme for pile retaining walls. The analysis also provides information on ultimate bending moments and shear forces on the pile wall.

- Serviceability Limit State (SLS) Analysis: this involved the use of unfactored soil parameters to estimate the potential lateral displacement of the pile wall, as well as service bending moments and shear forces on the pile retaining wall. This analysis has also been carried out with CADS PWS 5.6 modelling programme for pile retaining walls.

- Geostructural Finite Element Analysis of Pile Wall-Twin Buttress Pile System: the potential behaviour of the pile wall-buttress system was analysed by finite element (FE) modelling. This involved the use of unfactored soil parameters. The behaviour of the pile wall–buttress pile system is a complex soil-structure interaction problem that cannot be adequately catered for with simple empirical structural or geotechnical calculation methods. The geostructural assessment of such an earth retention system needs to consider not only the separate capacities of the individual components of the system, but their combined capacities and interaction under service loading e.g. the push-pull mechanism between twin buttress piles, load transfer between the pile wall, capping beam and buttress piles, etc.

  Based on the above, the finite element method was considered to be a more appropriate solution. The potential behaviour of the pile wall–buttress system on site 1 was analysed with PLAXIS 2D finite element modelling programme, which links the pile wall, buttress system, existing neighbouring structures, ground and ground water into a single soil-structure model. The FE model was based on very fine 16-node meshes, while soil parameters used have already been presented in table 1. The piles and capping beams were modelled as beam elements, while the permanent reinforced concrete (RC) slabs were modelled as fixed-ended anchors. Analysis was based on unfactored loads and soil parameters and it did provide information on potential ground movement, pile vertical & lateral deflections, pile & beam bending moments, shear forces and axial forces. A sample exaggerated deformed mesh is shown in figure 2, while a copy of the output file illustrating the predicted ground movement within the vicinity of the site is shown in figure 3. In addition, the main results of the FE analysis are summarised in table 4. Service forces and moments shown in table 4 were subsequently factored up to arrive at ultimate (design) values for the structural design of the twin buttress pile system.

**Table 1 – Generalised stratigraphy & geotechnical design parameters for site 1 (Chiswick, West London)**

| Depth (m) bgl | Description | $N_{spt}$ | Angle of Shearing Resistance $\phi'$ (°) | Cohesion $C_u$ or [C'] (kPa) | Young's Modulus $E_u$ or [E'] (kPa) |
|---|---|---|---|---|---|
| 0.0 – 2.0 | Made Ground/Firm Slightly Sandy Clay | 13-32 | 30.0 | [0.0] | [30000] |
| 2.0 – 9.0 | Medium Dense to Dense Sand & Gravel | 18-42 | 37.0 | [0.0] | [75000] |
| Below 9.0 | Stiff to Very Stiff London Clay | 28-40 | 25.0 | 90 + 8z [5.0] | 72000 + 6400z [57600 + 5120z] |

**Table 2 – Geotechnical design parameters for site 2 (Hadley Wood, Hertfordshire)**

| Stratum | Bulk Density $\gamma$ (kN/m$^3$) | $E_{50}^{ref}$ (kPa) | $E_{oed}^{ref}$ (kPa) | $E_{ur}^{ref}$ (kPa) | Cohesion $C_u$ (kPa) | Angle of Shearing Resistance $\phi'$ (°) | Coefficient of Earth Pressure $K_o$ |
|---|---|---|---|---|---|---|---|
| Stiff to Very Stiff London Clay | 19.0 | 76000 + 6400z | 76000 + 6400z | 228000 + 19200z | 95 + 8z | - | 1.0 |

$E_{50}^{ref}$ = secant stiffness in standard triaxial test, $E_{oed}^{ref}$ = tangent stiffness for primary oedometer loading, $E_{ur}^{ref}$ = unloading/reloading stiffness.

**Table 3 – Geotechnical design parameters for site 3 (Virginia Water, Surrey)**

| Depth (m) bgl | Stratum | Bulk density $\gamma$ (kN/m$^3$) | Young's modulus $E_u$ or [E'] (kPa) | Cohesion $C_u$ or [C'] (kPa) | Angle of shearing resistance $\phi$' (°) | Coefficient of earth pressure $K_o$ |
|---|---|---|---|---|---|---|
| 0.0 – 2.0 | Made Ground | 18.0 | [30000] | [0.0] | 25.0 | 0.5 |
| 2.0 – 8.0 | Very Soft to Soft to Firm Sandy Clay/ Loose to Medium Dense Sand | 19.0 | [40000] | [0.0] | 28.0 | 1.0 |
| Below 8.0 | Stiff to Very Stiff Sandy Clay | 19.0 | 120000 [96000] | 150.0 [5.0] | 25.0 | 1.0 |

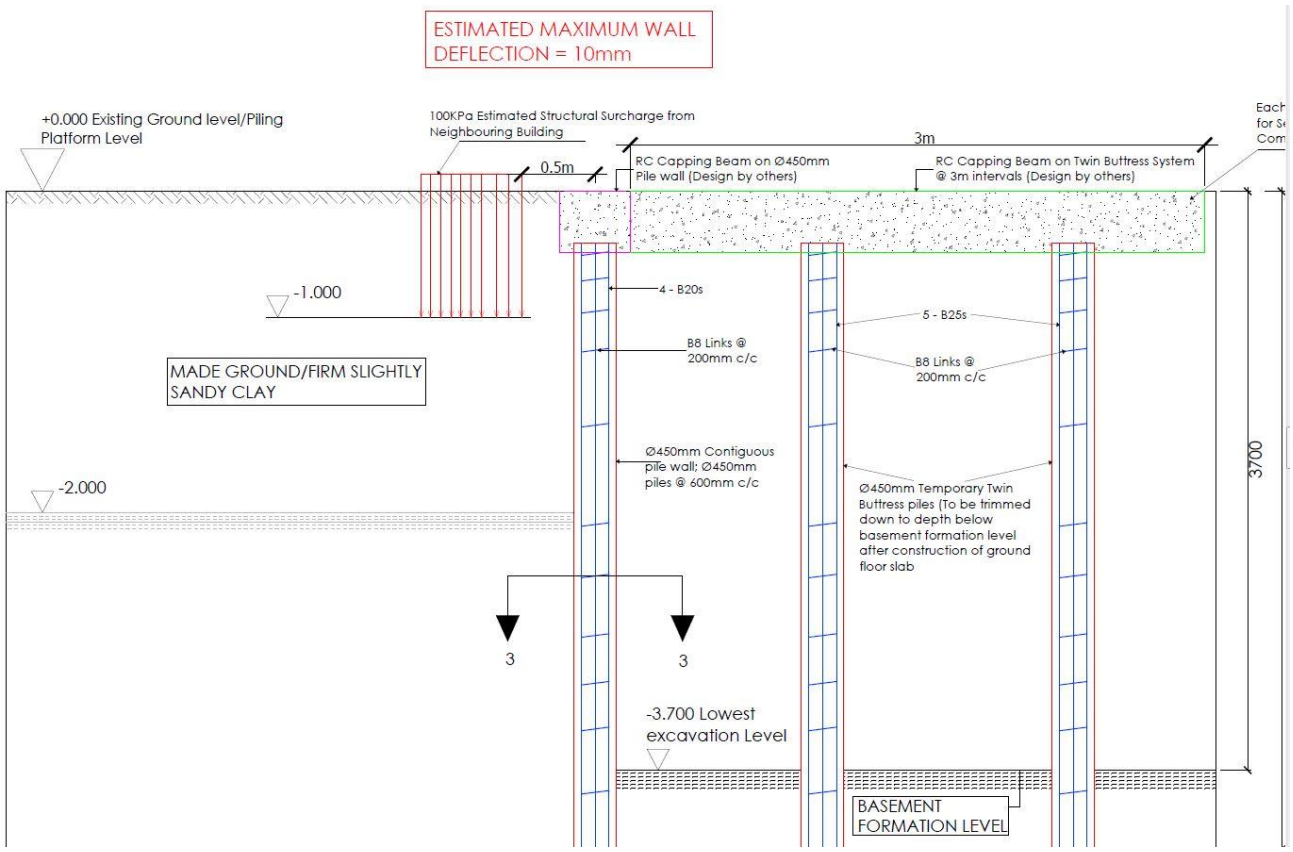



**Figure 1 – Typical section of the earth retention system on site 1 (Virginia Water, Surrey).**

Essentially, the FE model had predicted the maximum ground movement within the vicinity of the site and the worst case lateral deflection of the pile wall to be 7.5mm and 4mm respectively. Also, the maximum displacement of the buttress piles was estimated to be 3.5mm.

- Capacities of Twin Buttress Pile System and Pile Wall under Vertical Loading: the FE output files show the buttress piles to be subject to axial tension and compression during and after bulk excavation (see table 4). Also, the FE analysis results show the pile retaining wall to be subject to axial compression and tension. The designs for the pile wall and the twin buttress pile systems to support vertical axial loading were based on the traditional bearing capacity approach for axially loaded piles. However, the pile wall was assumed to act as a continuous deep strip footing below basement formation level, surrounded by a block of soil, with the assumption of a block type failure mechanism in the ultimate state. In addition, the bearing capacity factors $N_c$ & $N_q$ in the cohesive and cohesionless strata respectively were reduced with a factor $f$, to account for the existence of gaps between piles in the contiguous pile wall.

  The reduction factor $f$ is expressed as;

$$f = \frac{\pi D}{4S} \qquad \text{---------- [1]}$$

where D = pile diameter and S = pile centre to centre spacing. The above approach produces an estimate of the axial capacities of the pile wall per metre run; see Adekunte (2014) for more detailed information on this methodology.

- Structural Designs for Pile Wall, Twin Buttress Pile System and RC Capping Beam: based on the worst case results of the different sets of analysis described above, structural designs for the pile wall, TBPs and RC capping beams were carried out in accordance with the recommendations of the BS 8110 (1997).

**Construction Sequence Adopted on Site:** the sequence of construction followed on site 1 is presented below:

1. Install ∅450 piles @ 600mm c/c from piling platform level to specified depths, to form contiguous pile wall.
2. Install ∅450 twin buttress piles at 90° angle to pile wall run, at 3m intervals in front of pile wall (proposed bulk excavation/basement area).
3. Break down contiguous piles and buttress piles to 75mm above proposed soffit levels of RC capping beams.
4. Construct RC capping beams on contiguous pile wall and buttress piles. The capping beams on buttress piles must be rigidly connected to capping beam on pile wall.
5. Carry out bulk excavation to basement formation level (approx. 4m dig).
6. Construct basement slab and dowel into pile retaining wall.
7. Construct RC liner wall in front of pile retaining wall.
8. Construct ground floor slab.
9. Cut down buttress piles to depths below basement formation level.
10. Construct superstructure.

**Performance Monitoring of Pile Wall-Buttress System:** upon completion of the installation of the pile wall, the twin buttress pile systems and the RC capping beams, the pile wall was instrumented with survey targets, while the movements of the targets were monitored by optical survey during and after bulk excavation. The instrumentation and monitoring procedure followed is concisely outlined below:

- after pile installation and capping beam construction, shot fix nails were installed as targets at selected positions on capping beams;
- initial coordinates of targets were recorded using survey instrument;
- after recording initial coordinates, readings of coordinates were taken at 1 working day-interval during excavation and every two weeks after completion of excavation;
- monitoring continued over a period of 6 weeks after completion of bulk excavation until the basement and ground floor slabs achieved sufficient structural strength.

Monitoring results show maximum pile wall lateral deflection to be 2 mm, which was approximately 50% of the theoretically estimated potential wall lateral deflection, as described earlier in this section. As overall wall movement was relatively negligible, no serviceability issues were recorded on the adjacent buildings, which were located at relatively close proximity to the excavation area.

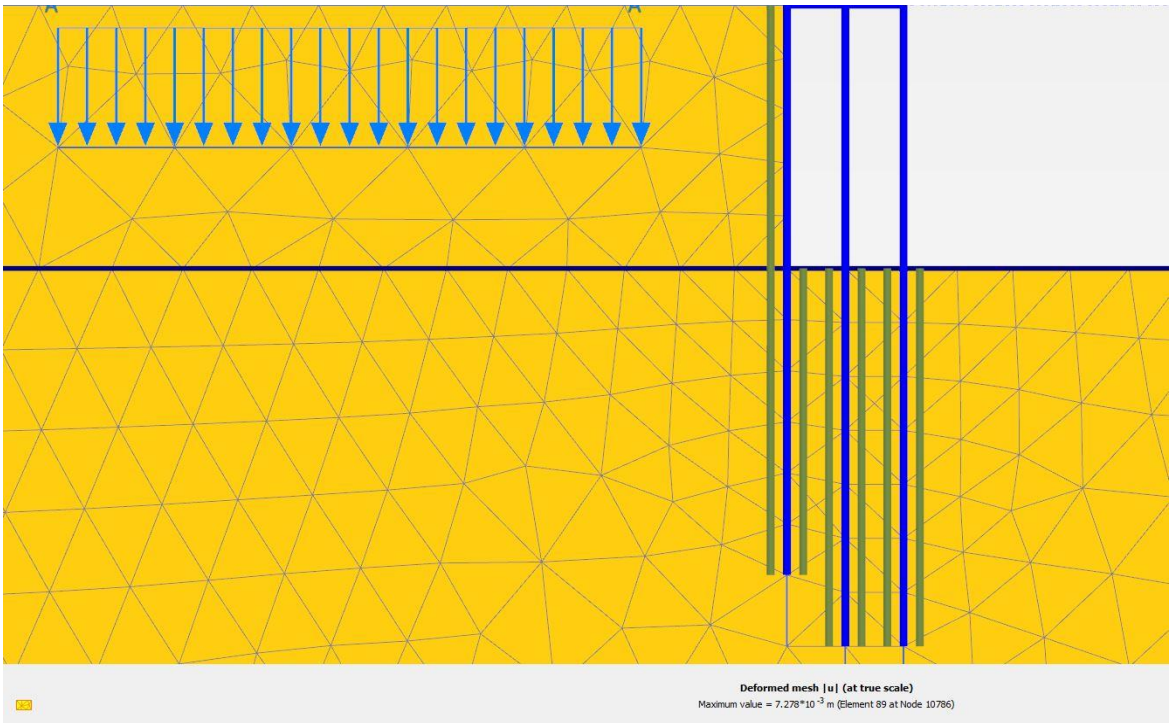



**Figure 2 – Typical deformed FE mesh for site 1 (Virginia Water, Surrey).**

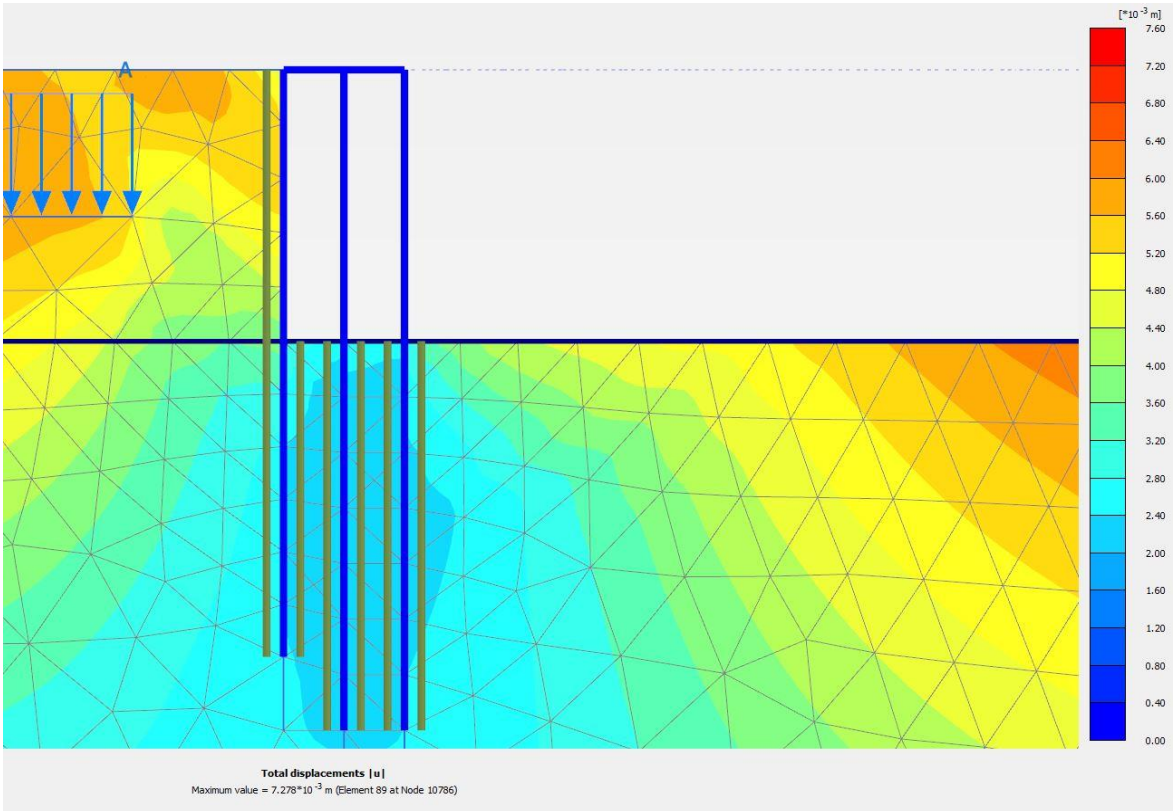



**Figure 3 – FE prediction of ground movement within the vicinity of site 1 (Virginia Water, Surrey).**

| Description | Estimated Value |
|---|---|
| Maximum Ground Movement (mm) | 7.5 |
| Maximum Pile Wall Lateral Deflection (mm) | 4.0 |
| Maximum Displacement of Buttress Piles (mm) | 3.5 |
| Maximum Settlement of Neighbouring Foundations (mm) | 6.5 |
| Maximum Compressive Loading on Buttress Piles (KN/m) | 40.0 |
| Maximum Tension Loading on Buttress Piles (KN/m) | -10.0 |
| Maximum Bending Moment on Buttress Piles (kNm/m) | 40.0 |
| Maximum Shear Force on Buttress Piles (KN/m) | 50.0 |
| Maximum Axial Loading on Buttress Capping Beam (KN/m run of wall) | 15.0 (compression) |
| Maximum Bending Moment on Buttress Capping Beam (kNm/m run of wall) | 10.0 |
| Maximum Shear Force on Buttress Capping Beam (KN/m run of wall) | 20.0 |

**Table 4 – Summary of FE analysis results for site 1 (Virginia Water, Surrey).**

## SITE 2 (HADLEY WOOD, HERTFORDSHIRE)

**Proposed Development:** the development on site 2 comprised of two separate 4 storey-residential blocks of luxury apartments; Blocks A and B, with separate single level basements for both blocks, which were required for underground parking, communal gyms and cinemas. It was proposed to support the 5.5m deep excavation for the basements with contiguous bored pile retaining walls. Typical section of the earth retention system on site 2 is shown in figure 4. In contracts to the solution adopted on site 1, the TBPs on site 2 were installed behind the pile retaining wall (i.e. outside the proposed excavation/basement area); this approach offered the additional benefits completely maximising the available working space in the basement area, while there was no requirement for the trimming down of buttress piles after basement construction and ultimately, the solution was advantageous to the main contractor's construction programme.

**Design, Construction Sequence & Performance Monitoring:** the design approaches, construction sequence, instrumentation and monitoring procedures adopted on site 2 were similar to those adopted on site 1 as already described in the penultimate section of this paper. Piles for the main pile wall were 8m long, while the buttress piles were 9m long, as illustrated in figure 4 overleaf.

Instrumentation and monitoring results show maximum pile wall lateral deflection to be 4 mm, which was less than 60% of the theoretical prediction made with the FE model for the earth retention system on the site. Overall, the TBPs were found to be quite efficient, as no serviceability issues were recorded within and outside the site, while they did allow the main contractor to make a total savings of 4.5 weeks in construction time, when compared with the Supervising Project Engineer's original proposal for the use of temporary flying struts and corner braces for the basement scheme.

## SITE 3 (VIRGINIA WATER, SURREY)

**Proposed Development:** the development on site 3 was centred on the construction of a new underground car park next to the west wing of an existing 3 storey-mansion. A contiguous pile retaining wall was required to provide lateral support for the 2.3m-6.0m deep excavation for the new subterranean structure. The pile retaining wall comprised of ∅350 piles @ 500mm c/c. The twin buttress pile system for the pile wall was installed in front of the pile wall and the system comprised of ∅450 twin buttress piles installed at right angles to the pile wall run, at 3m intervals in front of the wall. Figure 5 shows the layout of an installed buttress pile system in relation to the pile wall run, while figure 6 shows the pile wall and buttress pile system after completion of bulk excavation.

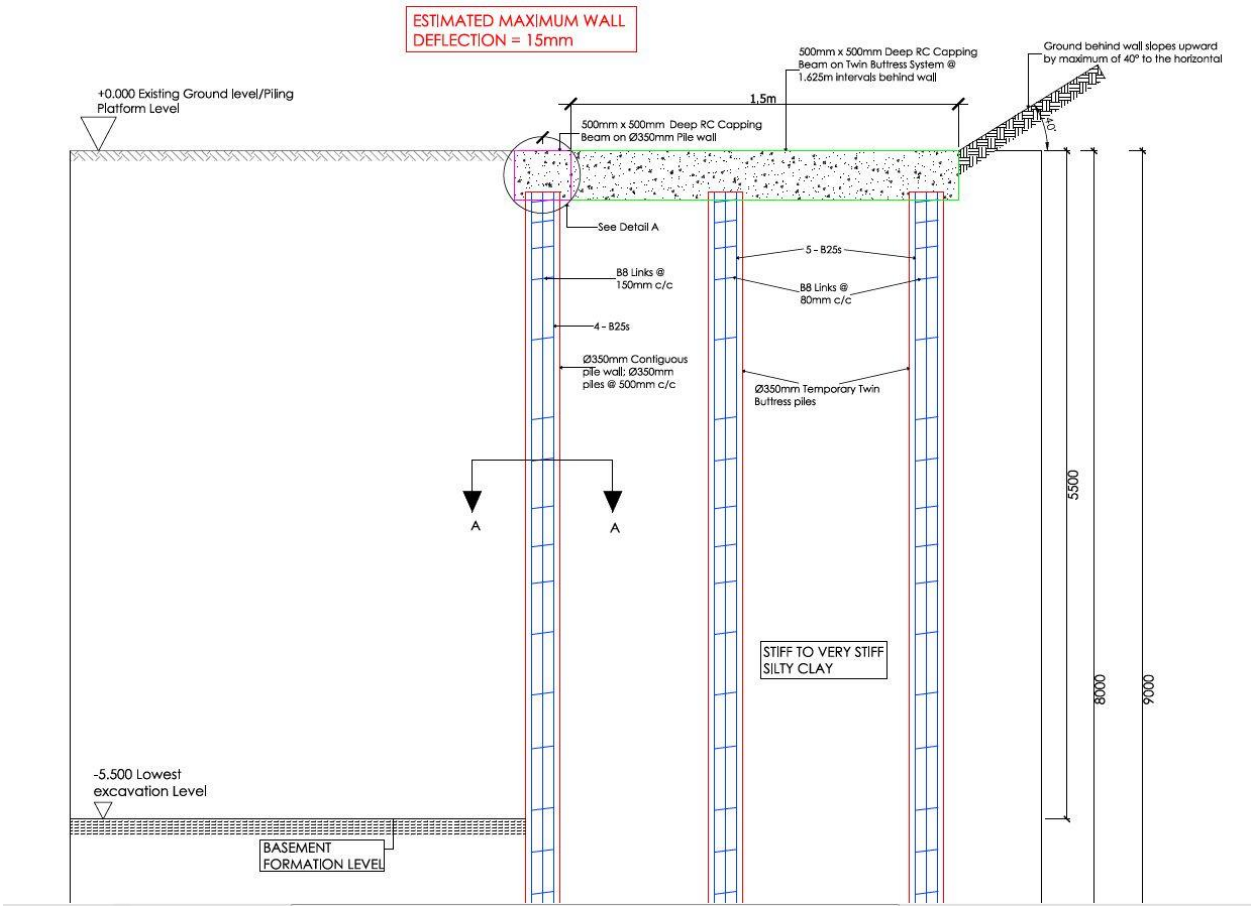



**Figure 4 – Typical section of the earth retention system on site 2 (Hadley Wood, Hertfordshire).**



**Figure 5 – Layout of pile wall, as well as twin buttress piles installed at 90° angle to wall run on site 3 (Virginia Water, Surrey).**

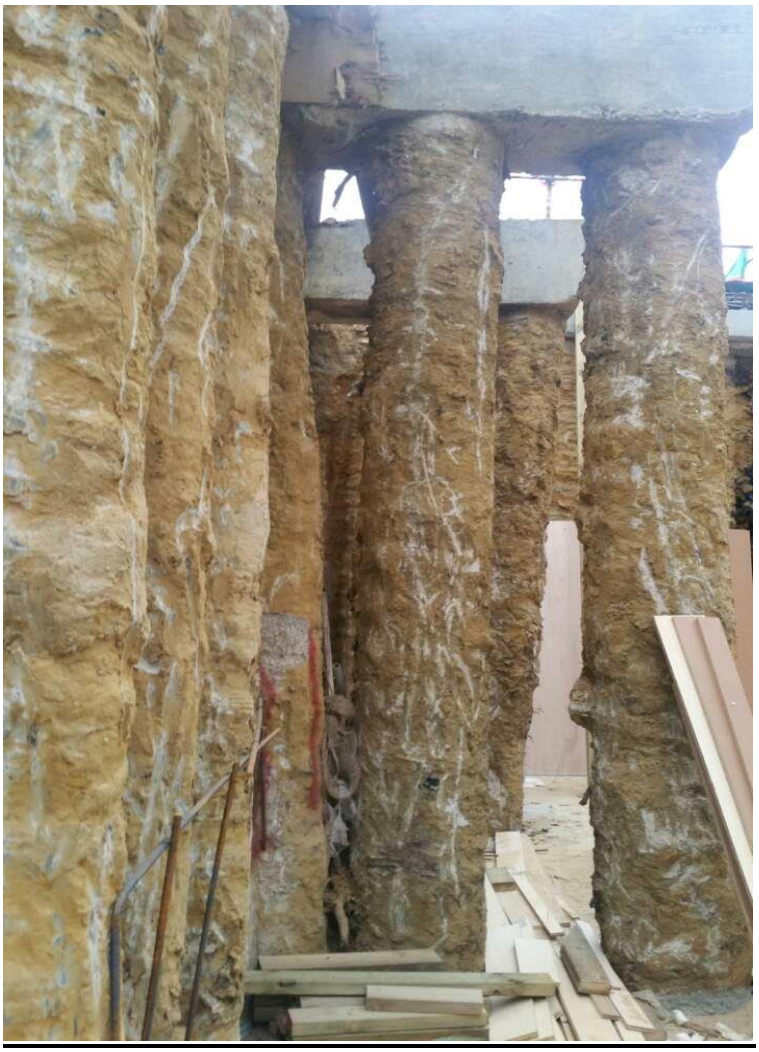


**Figure 6 – Installed twin buttress pile systems, after completion of bulk excavation for basement on site 3 (Virginia Water, Surrey).**

In similarity with sites 1 & 2, on site 3, the TBPs were efficient at limiting the overall movement of the pile retaining wall. Maximum pile wall lateral movement of 9mm was recorded in the deepest area of the basement (6.2m excavation zone). No serviceability issues were recorded on the immediately adjacent substantial building during and after bulk excavation.

## SUMMARY & CONCLUSIONS

Detailed information on the numerical modelling, design, construction, instrumentation and monitoring of pile retaining walls restrained with twin buttress pile systems (TBPs) on three separate basement schemes have been provided. The paper clearly lays out the typical construction sequencing and performance monitoring procedures involved with the application of the non-traditional system and these are supported with excerpts of project drawings and site photographs.

Essentially, the TBPs have been found to be quite efficient at limiting wall movements and maintaining overall stability on all the three sites covered in the study. Monitored pile wall deflections were generally

much lower than theoretical design estimates, while no serviceability issues were recorded on neighbouring properties or within the sites.

The main benefits of the TBPs over traditional propping and ground anchorage include; maximisation of working space within excavation areas (an advantage over props) and the significantly reduced potential of intrusion into neighbouring properties (an advantage over tie-back anchorage). In addition, as the TBPs are installed concurrently with the pile retaining wall, with the same equipment, they offer significant benefits towards the contractor's construction programme/work schedule.

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