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DEALING WITH COMPLEXITIES ASSOCIATED WITH THE APPLICATION OF BORED PILE RETAINING WALLS IN URBAN DEVELOPMENTS

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Engineers and developers have witnessed the growing popularity of deep basement structures as components of urban commercial and residential developments. This is often attributed to inadequate surface parking facilities in urban centres and a somewhat popular belief that underground parking facilities improve both commercial and aesthetic values of urban structures. For many years, diaphragm walls and sheet pile walls have been widely accepted as effective deep excavation support systems for basement construction in urbanised zones. However, these are now being replaced by bored pile retaining walls as preferred earth retention systems of choice on many projects for several reasons, which may include economy, faster installation process, reduced space requirement for machinery and ancillary equipment, flexural rigidity and axial capacities.

However, there are complexities associated with the use of bored pile retaining walls on urban developments. These include; limitations due to vertical and horizontal installation tolerances, which is a major issue when wall is required to provide groundwater cut-off for multi-level basements as tolerances are difficult to achieve for deep piles, uncertainty associated with the prediction of wall axial capacity as piles are rarely tested, difficulties in using temporary props as wall restraint due to restricted working spaces in urbanised zones and difficulties in adopting ground anchors as restraint due to the existence of adjacent sensitive structures in built-up areas. Such complexities have resulted in cases of failure, non-serviceability and abandonment of basement construction schemes. Therefore, overcoming the intricacies associated with the application of bored pile walls is becoming a priority to the present day basement engineer.

This paper is centred on the numerical modelling, design, construction and monitoring aspects of four recent case histories of multi-level basement and marine engineering construction schemes in the United Kingdom and the Republic of Ireland, which involved the use of bored pile retaining walls. The paper highlights the various customised approaches and innovative novel solutions adopted on the sites, to mitigate risks and ultimately overcome the intricacies associated with the earth retention techniques. Simple empirical equations for developing the geometry of secant pile walls for effective groundwater cut-off are presented and validated. An alternative approach for predicting axial capacities of bored pile walls is also presented; this is shown to be considerably less conservative than a common routine design approach, while also in good agreement with observations made in a recent series of laboratory-based scale model tests.

INTRODUCTION

Deep basement structures have been well associated with urban commercial and residential developments for many years. The major reason behind this trend is insufficient surface parking spaces in crowded urban centres. This often compels many developers to incorporate off-street underground parking into their building schemes. In addition, developers and property specialists consider basement parking to add significant market value to urban buildings. Owners of buildings with basement car parks in urbanised zones often make their parking facilities available for public use, to generate additional commercial revenue.

The construction of deep basements routinely requires the installation of temporary or permanent excavation support systems, to enable the construction of the sub-structure. Reinforced concrete diaphragm walls (D-walls) and sheet pile walls have been globally adopted for this purpose for many years. However, in recent years, bored pile retaining walls are increasingly becoming more popular, as they offer some advantages over D-walls and sheet pile walls.

With bored pile retaining walls, installation process is more straightforward, while production rate is higher and for these reasons, they are often adopted as more economical alternatives to D-walls. Reduced space requirement for machineries is another advantage of bored pile walls over D-walls. When compared with steel sheet pile walls, bored pile walls provide higher stiffness, higher flexural rigidity and higher vertical axial capacity, while durability in the long term is of less concern.

However, the application of bored pile walls in urban environment comes with a number of intricacies. In addition to lateral soil and water retention, bored pile walls are now being commonly required to support vertical compressive and/or tension loading from the superstructure. As piles in retaining walls are rarely statically or dynamically load-tested, design methods and assumptions are not routinely verified and as a result, there is a degree of uncertainty surrounding common methods of predicting axial capacities of bored pile retaining walls. In the case of multi-level basement schemes on which groundwater cut-off is required in addition to earth retention, horizontal and verticality tolerances often become major issues to contend with, as it is difficult to achieve required installation tolerances with deep piles associated with multi-level basements.

Also, it is becoming increasingly difficult to restrain bored pile retaining walls with the well-known traditional systems; props and tie-backs. Due to lack of space in built-up areas, many urban construction sites are tight and as contractors are compelled to maximise working space, they often discourage the use of temporary props and braces. Existence of adjacent buildings and sensitive underground structures and services also renders the application of tie-back anchors difficult.

This paper focuses on each of the intricacies outlined above by making references to a number of recent case histories in different geological settings in the United Kingdom and the Republic of Ireland. For each case history, the paper reviews details of the numerical modelling, geotechnical & geostructural design, construction procedure, instrumentation & monitoring, complexities encountered and solutions developed and adopted to overcome the complexities. These are presented in the following sections.

SITE GEOLOGY & GROUND MODELLING

This study focuses on four different sites; site 1, site 2, site 3 and site 4. Site 1 is located in Central London, site 2 is located in Enfield, Greater London, site 3 is located in Dublin, while site 4 is located in Cavan, North Central Ireland. Maps of the United Kingdom and Ireland showing the locations of the 4 sites are presented in figure 1.



Figure 1 – Map of the United Kingdom and Ireland Showing Site Locations.

Stratigraphy on site 1 comprises of up to 3m thick made ground directly underlain by a layer of very soft to soft clay and peat to 9m depth, which in turn overlies a 1m thick layer of medium dense to dense sandy gravel. Firm to stiff to very stiff to hard London clay lies below the sandy gravel stratum at 10m depth and it is underlain by a layer of very dense Thanet Sand at 48m depth. As the site was next to the River Thames on the south eastern boundary, groundwater level was subject to tidal fluctuation and standpipe monitoring results show that water level could be as high as 1m below ground level (bgl). Generalised stratigraphy on site 1 and associated geotechnical design parameters are presented in table 1.

Stratigraphy on site 2 comprises of made ground overlying loose to medium dense to dense sandy Taplow gravel, which is underlain by firm to stiff to very stiff bluish grey London clay. Ground conditions on sites 3 & 4 mainly comprise of made

ground and medium dense gravel overlying Irish glacial till; a mixed material comprising of boulders, cobbles, gravel, sand and clay-sized particles randomly mixed together, an effect of glacial action and deposition. Both London clay and Irish glacial till are overconsolidated materials of low permeability. However, London clay deposits on sites 1 & 2 are of high plasticity, with plasticity indices ranging between 35%-45%, while the glacial till deposits on sites 3 & 4 are of low to intermediate plasticity with plasticity indices varying between 9%-21%. Figures 7c, 9 and 11 show generalised ground conditions on sites 2, 3 and 4 in relation to the excavation support system on each of the sites. Geotechnical design parameters for sites 2, 3 and 4 are presented in tables 2, 3 and 4 respectively. Values of coefficient of earth pressure K_o were generally limited to 1.0 for the overconsolidated materials (after CIRIA Report No. C580, 2003), this was to allow for remoulding and stress relief in the soils; conditions associated with embedded retaining wall installation and excavation.

For the walls on sites 2, 3 & 4, effective wall friction angles δ on the active and passive sides were taken to be $0.67\phi'$ and $0.5\phi'$ respectively (where ϕ' is effective angle of shearing resistance). However, for the wall on site 1, angle of wall friction in the permanent condition was assumed to be zero. This was to allow for the effect of vertical compressive loading on the wall from the superstructure in the permanent condition.

SITE 1

Major Complexities: Horizontal & Vertical Installation Tolerances, Estimation of Male-Female Interlock Thickness & Pile Spacing Required for Effective Groundwater Cut-off and Uncertainty Surrounding Wall Axial Capacity Prediction

Proposed Development & Construction Procedure:

Site 1 was adjacent to the River Thames and the Vauxhall Bridge in Central London. The project was a large scale high rise development comprising of two separate structures on the same site; a 28 storey-commercial tower and an 18 storey-residential block of luxury apartments. The development included a 3-4 level basement; a 3 level-basement over the entire foot print of the site and a 4th level of basement in the centre of the site. Due to the proximity of the site to the Thames River, groundwater level was relatively high and subject to tidal fluctuation. Therefore, in addition to soil retention, groundwater cut-off was a necessity to enable basement construction and a hard-firm secant pile wall was considered to be a potentially effective and economical solution for the first three levels of basement across the site; this is a system of interlocking male and female piles.

Male piles were made of grade 35 structural concrete and reinforced down to depth required for overall wall lateral stability, while the female piles were made of 10 N/mm2 concrete and unreinforced. Both male and female piles were provided with a minimum embedment of 1m in London clay below basement formation level, for groundwater cut-off. Male piles were of 880mm diameter, while female piles were of 900mm diameter. An illustration of the wall geometry is shown in figure 2, while a photo of the secant pile wall is shown in figure 3. However, as the 4th level of basement wholly lies within the London clay stratum, groundwater control was not considered to be a major issue and therefore, a contiguous pile wall was considered to be the most economical and effective solution. Contiguous pile wall comprised of 750mm diameter bored piles spaced at 900mm centre to centre (c/c).

Maintaining a perfect interlock between adjacent male and female piles is often difficult to realise when dealing with multi-level basements, as required horizontal and vertical tolerances are difficult to achieve with many conventional piling rigs, for deep bored piles associated with deep basements. To mitigate this risk, prior to commencement of piling, a guide wall was put in place along the wall alignment, for improved horizontal positional tolerance. In addition, for improved verticality tolerance, male piles were installed with a Cased Secant Piling (CSP) rig, which is a customised version of the traditional Continuous Flight Auger (CFA) drilling rig, equipped with a temporary heavy duty steel casing over the full depth of the pile.

Table 1 – Generalised Stratigraphy & Geotechnical design parameters for site 1 (Vauxhall, Central London)

Depth (m) bgl	**Description**	**N_{spt}**	**Angle of Shearing Resistance ϕ' (°)**	**Young's Modulus E_u or [E'] (kPa)**	**Cohesion C_u or [C'] (kPa)**
0.0 – 9.0	Made Ground & Very Soft to Soft Clay & Peat	1 - 8	25.0	[10000]	[0.0]
9.0 – 10.0	Medium Dense to Dense to Very Dense Sandy Gravel	17 - 41	36.0	[50000]	[0.0]
10.0 – 16.0	Firm to Stiff London Clay	13 – 30	25.0	80000 [65000]	100.0 [5.0]
16.0 – 29.0	Very Stiff London Clay	32 – 44	25.0	120000 [95000]	150.0 [5.0]
29.0 – 48.0	Very Stiff to Hard London Clay & Weak Mudstone Bands	43 – R	25.0	175000 [140000]	220.0 [5.0]
Below 48.0	Very Dense Thanet Sand	R	37.0	[150000]	[0.0]

Table 2 – Geotechnical design parameters for site 2 (Enfield, Greater London)

Stratum	**Bulk Density γ (kN/m^3)**	**E_{50}^{ref} (kPa)**	**E_{oed}^{ref} (kPa)**	**E_{ur}^{ref} (kPa)**	**Cohesion C_u or [C'] (kPa)**	**Angle of Shearing Resistance ϕ' (°)**	**Coefficient of Earth Pressure K_o**
Made Ground	18.0	20000	20000	60000	[0.0]	30.0	0.5
Loose to Medium Dense Gravel	19.0	40000	40000	120000	[0.0]	33.0	1.0
Firm to Stiff London Clay	19.0	40000	40000	120000	100.0 [5.0]	25.0	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 (Dublin Port, Ireland)

Stratum	**Bulk density γ (kN/m^3)**	**Young's modulus E_u or [E'] (kPa)**	**Cohesion C_u or [C'] (kPa)**	**Angle of shearing resistance ϕ' (°)**	**Coefficient of earth pressure K_o**
Made Ground	18.0	[30000]	[0.0]	30.0	0.5
Medium Dense Gravel	19.0	[40000]	[0.0]	32.0	1.0
Stiff Sandy Clay	19.0	60000 [40000]	75.0 [5.0]	32.0	1.0

Table 4 – Geotechnical design parameters for site 4 (Cavan, Ireland)

Stratum	Bulk Density γ (kN/m^3)	Young's Modulus E' (kPa)	Angle of Shearing Resistance ϕ' (°)	Coefficient of Earth Pressure K_o	Pile Shaft 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 [C' = 5.0]	1.0	110.0

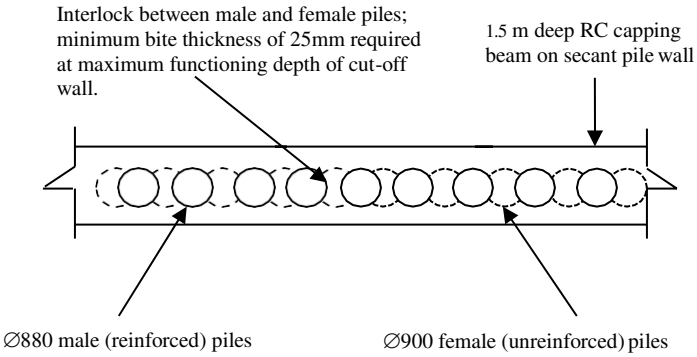


Figure 2 – Layout of hard-firm secant pile wall on Site 1 (Vauxhall, Central London)

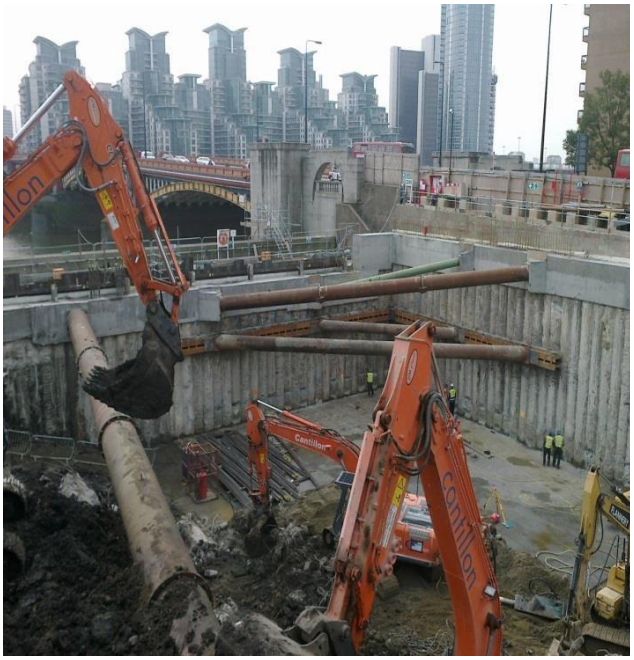


Figure 3 – Hard-firm secant pile wall on Site 1 (Vauxhall, Central London)

The CSP technique improves the achievable vertical tolerance to 1:150, when compared with 1:75 achievable with a conventional uncased CFA bore (after ICE SPERW, 2007). In addition, the customised technique results in better finished wall appearance and reduced overbreak when compared with traditional CFA drilling, while it is also a faster and more economical technique when compared with rotary bored piling.

Economy is another important factor in foundation engineering and construction (especially from a contractor's perspective) and this has to be accounted for in every aspect of wall design and installation. For economical reason, female piles were designed to be installed by traditional CFA drilling; the reduced verticality tolerance was off-set by the commercial benefit derived from the lower hire rate of the CFA rig, when compared with the CSP rig.

The 4th level of basement at the centre of the site, which had to be constructed beneath the excavation level of the perimeter 3 level-secant pile wall, was supported with a contiguous pile wall. This was installed by a Bachy Soletanche Large Diameter Auger (LDA) rotary boring technique. Above the London clay stratum, the LDA rig advanced the drillhole with the combination of an auger and a thick-walled temporary casing, rotating in opposite directions. The temporary casing was required to provide lateral support to the unstable made ground and alluvial deposits above the London clay. In the London clay, hole was drilled with the auger only. This rotary boring technique allowed each hole to be partially concreted up to the formation level of the general 3 level-basement (approximately 11m bgl), while the rest of the hole was backfilled with carefully selected gravel peas up to ground/piling platform level. This approach

allowed for the installation of both the secant and contiguous pile walls from ground/piling platform level, such that excavation for the top 3 levels of basement was done without having to deal with concrete piles projecting from the 4th level contiguous wall in the middle of the site; this also resulted in savings in concrete.

Overall, the basement works package involved the use of three different bored piling techniques for improved efficiency and economy.

Groundwater Cut-off:

On deep basement construction projects where temporary or permanent groundwater cut-off is required in addition to earth retention, secant pile walls have become popular solutions of choice. However, many cases of flooded and/or abandoned secant wall-supported basements have been recorded. Such problems arise because many engineers are unaware of the fundamental principles underlying the accurate estimation of minimum interlock thickness between male and female piles (and ultimately pile spacing) required to ensure an effective groundwater cut-off solution. To provide effective groundwater cut-off with a secant pile wall, many factors would need to be accounted for. These include;

- rig type;
- rig capability;
- installation technique;
- pile size;
- verticality tolerance;
- horizontal positional tolerance;
- depth of sealing stratum;
- minimum required pile embedment in sealing layer.

All of the above factors must be accounted for. Not accounting for just one of the factors could result in significant consequences. While publications with comprehensive specific information on this subject are very rare, field experience has shown that the minimum required interlock thickness between male and female piles may be expressed by the equation;

$$\beta = \delta + \alpha \qquad \text{---------- (1)}$$

Where β = estimated required bite thickness between male and female piles at piling commencement level in mm.

δ = estimated potential gap in mm between male and female piles at pile depth required for cut-off. This is a function of horizontal positional tolerance, verticality tolerance and length of pile L required for cut-off (equivalent to length of female pile).

α = minimum allowable bite thickness in mm between male and female piles at cut-off depth. This may be taken as 25mm (after ICE SPERW, 2007).

δ in equation 1 can be expressed as;

$$\delta = k\,[(\upsilon * L) + h] \qquad \text{---------- (2)}$$

Where υ = average verticality tolerance of both male and female piles in mm/m depth of installation, which is dependent on rig type and piling technique.

h = horizontal positional tolerance in mm at piling commencement level. This is dependent on availability of guide wall prior to the commencement of piling. A conservative value of 25mm may be assumed (after ICE SPERW, 2007).

L = pile length required for cut-off; this is equivalent to depth of female pile toe.

k = a constant, which accounts for the potential horizontal and vertical deviation of immediately adjacent male and female piles in opposite directions. k should be taken to be 2.

Therefore, equation 1 may be rewritten as;

$$\boxed{\beta = k\,[(\upsilon * L) + h] + \alpha} \qquad \text{---------- (3)}$$

The value of β can be used to estimate the required centre to centre spacing of the male piles, provided male and female pile diameters are known. This is illustrated in figure 4.

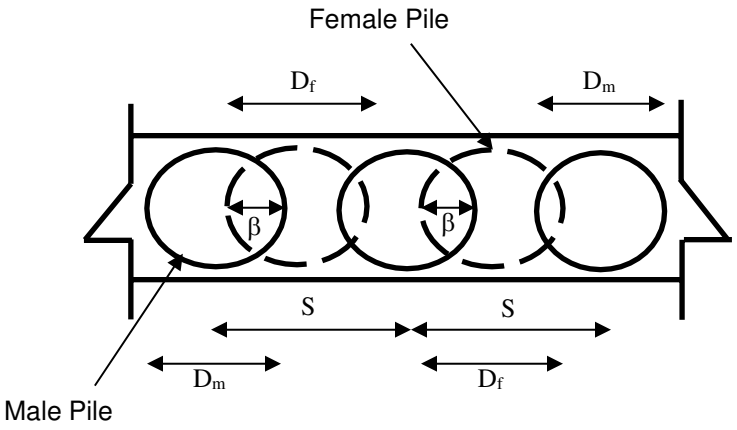


Figure 4 – Secant wall geometry annotation.

In figure 4;

D_m = male pile diameter

D_f = female pile diameter

β = thickness of interlock between overlapping male and female piles

From figure 4, male pile centre to centre spacing S can be expressed as;

$$S = [\frac{D_m}{2}] + [(\frac{D_f}{2}) - \beta] + [\frac{D_f}{2}] + [(\frac{D_m}{2}) - \beta] \quad \text{---------- (4)}$$

$$\therefore S = D_m + D_f - 2\beta \quad \text{---------- (5)}$$

Replacing β with equation 3, equation 5 becomes;

$$\therefore S = D_m + D_f - 2k\,[(\upsilon * L) + h] - 2\alpha \quad \text{------ (6)}$$

Adopting a value of 2 for k as suggested above, equation 6 becomes;

$$S = D_m + D_f - 4\,[\upsilon L + h] - 2\alpha \quad \text{------ (7)}$$

$$\therefore \boxed{S = D_m + D_f - 2[2\,(\upsilon L + h) + \alpha]} \quad \text{------ (8)}$$

By using the project on site 1 (Vauxhall, Central London) as a reference, equations 3 and 8 can be used to estimate the required bite thickness β at commencement level and associated male pile centre to centre spacing S respectively. On the project, secant pile wall retained height was 11m. Basement formation level was in London clay and a minimum embedment of 1m in London clay (sealing stratum) below basement formation level was required for groundwater cut-off.

Therefore, length of pile L required for groundwater cut-off = (11 + 1)m = 12m.

Male piles were installed by CSP technique; verticality tolerance = 1:150 (after ICE SPERW, 2007) and this is equivalent to a deviation of 6.67mm per metre depth of installation.

Female piles were installed by CFA technique with heavy duty augers; verticality tolerance = 1:100 (after ICE SPERW, 2007) and this is equivalent to a deviation of 10mm per meter depth of installation.

Therefore, average verticality tolerance υ of both male and female piles = [(6.67 + 10)/2] mm/m ≅ 8mm/m.

While guide wall was installed along wall alignment prior to start of piling works, horizontal positional tolerance h at piling commencement level can be conservatively taken to be 25mm.

Minimum allowable bite thickness at pile toe can be taken to be 25mm (after ICE SPERW, 2007).

Constant k accounting for potential deviation of overlapping piles in opposite directions can be taken to be 2 as suggested above.

By substituting the above values in equation 3, minimum required bite thickness β between overlapping male and female piles is estimated to be;

β = 2[(8mm/m * 12m) + 25mm] + 25mm ≅ **270mm**

Male pile diameter D_m = 880mm

Female pile diameter D_f = 900mm

Substituting the above values in equation 8, required male pile centre to centre spacing is estimated to be;

S = [880 + 900 – 2[2(8 * 12 + 25) + 25]] mm

S = 1246mm ≅ **1.25m**

Therefore, required centre to centre spacing of male piles is 1.25m c/c, which is approximately equivalent to 1.4D (where D is pile diameter). Installation of the secant pile wall on site 1 was

based on the above figures and the end result was a perfectly dry excavation as can be seen in figure 5 below, despite the wall having to deal with relatively high water level that was subject to tidal fluctuation and more than 10m of cohesionless made ground, very soft to soft alluvial deposits and sandy gravel.

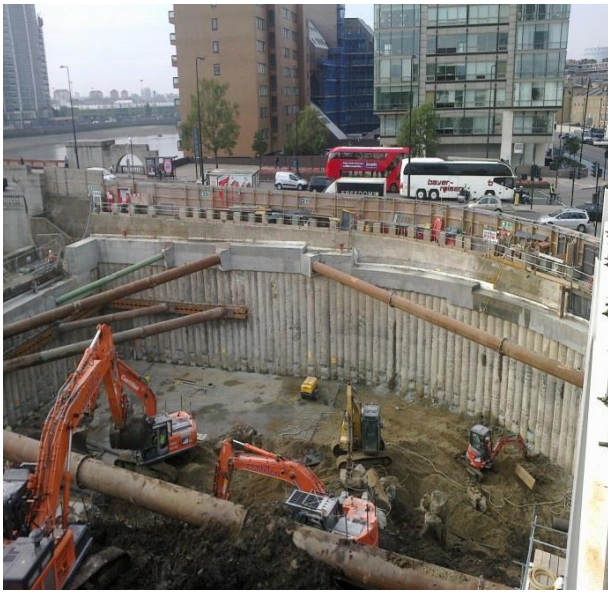


Figure 5 – Perfectly dry excavation was achieved with the secant pile wall on site 1 (Vauxhall, Central London).

Uncertainty Surrounding the Prediction of Wall Axial Capacity:

In addition to earth and water retention, the hard piles in the secant wall on site 1 were required to support vertical compressive column loads from the superstructure. The RC capping beam on the piles was designed to spread the point loads from the columns over a minimum of 3 No. hard piles, such that the maximum estimated service compressive load on any hard pile was 2450 KN.

The estimation of axial capacities of pile retaining walls is an area shrouded in uncertainty because unlike traditional bearing piles, hard piles in bored pile walls are rarely tested. It is common practice among designers to routinely approach this problem by estimating the capacity of a singly acting pile in similar ground conditions and thereafter, factoring down the estimated capacity to 50%-60% of estimated figure, to account for pile group efficiency in the wall. This approach is common for walls in clay. However, Rose & Taylor (2010, 2012 & 2013) carried out some experimental work and parallel finite element modelling on the behaviour of pile groups arranged in grids in clay at the City University London. They discovered that piles on the perimeter are subject to higher loads when compared with the internal piles; this they have attributed to increased soil-pile interaction.

Further to this, they carried out a series of tests on piles arranged in single row, to model the behaviour of perimeter pile groups with no internal piles, whilst also considering several variables that influence pile group behaviour; number of piles, pile spacing, length/diameter ratio and soil strength. From their experimental results, Rose & Taylor (2013) discovered that pile group efficiency of perimeter piles could be as high as 100%, while perimeter pile groups with pile centre to centre spacing of less than 2D (where D is pile diameter) tend to exhibit block type failure. Piles in pile retaining walls are typically spaced at ≤ 1.5D centre to centre.

Based on Rose & Taylor's (2010, 2012 & 2013) test results, at design stage, the wall on site 1 was considered 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 factor N_c in the London clay was reduced by multiplying with a reduction factor f, to account for the existence of non-structural female piles in the wall. The reduction factor f is expressed as;

$$f = \frac{\pi D}{4S} \qquad \text{---------- (9)}$$

Where D = pile diameter and S = male pile centre to centre spacing. This approach produces an estimate of the axial capacity of the wall per metre run. Multiplying this value by the centre to centre spacing of the hard piles yields the vertical capacity of an individual hard pile.

Prior to the construction of the RC capping beam on the wall, a number of hard piles in the wall were tested to 1.5 times the safe working load (SWL) in compression. As tests were carried out before excavation for the basement, additional capacities derived by test piles through skin friction in soils above proposed basement formation level were discounted while assessing measured capacities of the test piles. The load-settlement curves for the non-failed load tests

were extrapolated beyond the maximum test loads, following the analytical approach of Paikowsky & Tolosko (1999), to determine the ultimate vertical capacities of the piles in compression. Load-settlement curve for one of the test piles is shown in figure 6.

Figure 6 shows a reasonable level of agreement between ultimate capacity predicted with the method outlined above and actual measured capacity, while the common industry approach of factoring down the estimated capacity of an equivalent singly acting pile to 50%-60% to account for pile group efficiency is seen to be quite conservative. In addition, the pile group efficiency of the wall was deduced to be approximately 90%, which is in line with Rose & Taylor's (2010, 2012 & 2013) assertion that pile group efficiency of perimeter piles could be close to unity. The 90% pile group efficiency recorded is also noted to be higher than 50%-60% suggested by some workers (e.g. Broms, 2007 & Sowers et al., 1961) for piles spaced at < 1.5D in clay and which are typically adopted in practice.

SITE 2

Major Complexities: Serviceability Requirements & Restrictions to Applicable Wall Restraining Methods

Proposed Development & Construction Procedure:

The project on site 2 was a mixed-use development comprising of a 5m high basement for underground parking. A contiguous pile wall was installed for excavation support. However, as the development was located in a built-up area, a number of wall sections were adjacent to existing single to multi-storey buildings supported on shallow foundations. In order to limit ground and structural movement, whilst also maintaining overall stability, there was a significant necessity to support these wall sections with restraining systems. The supervising engineers for the project had specified maximum allowable wall deflection of 10mm and maximum allowable settlement of adjacent buildings of 5mm.

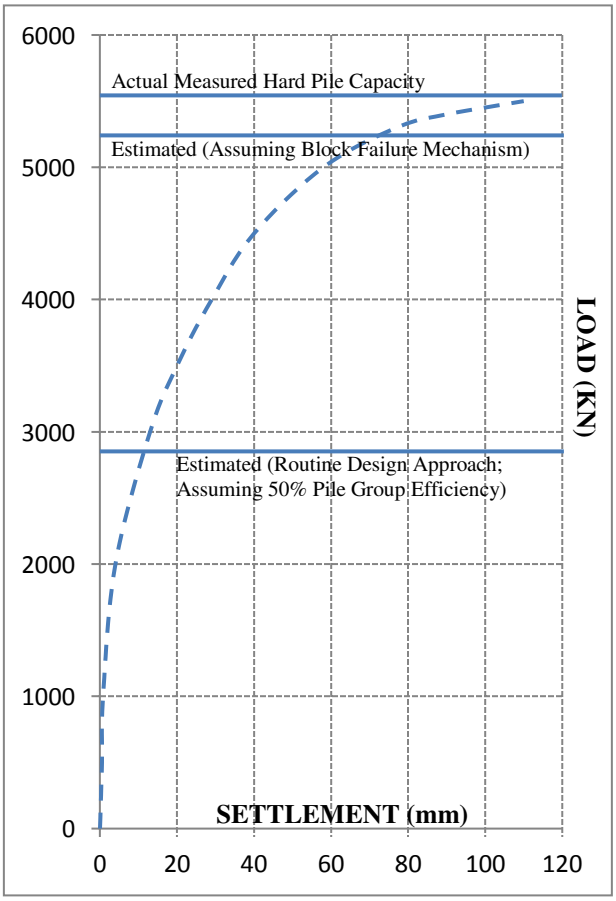


Figure 6 – Extrapolated load-settlement curve for a selected hard pile on site 1 (Vauxhall, Central London).

Tie-back anchors could not be installed as owners of the adjacent buildings were not in support of drilling and grouting beneath their buildings. Also, as it is often the case with developments in urbanised zones, space on the site was limited and because the maximisation of working space was of high priority, the main contractor was not in support of a temporary propping scheme.

As an alternative, the wall was restrained with a system of twin vertical buttress piles installed at 5m intervals behind the wall and rigidly connected to the contiguous pile wall with a RC capping beam. The novel system is illustrated in figures 7a, 7b and 7c.

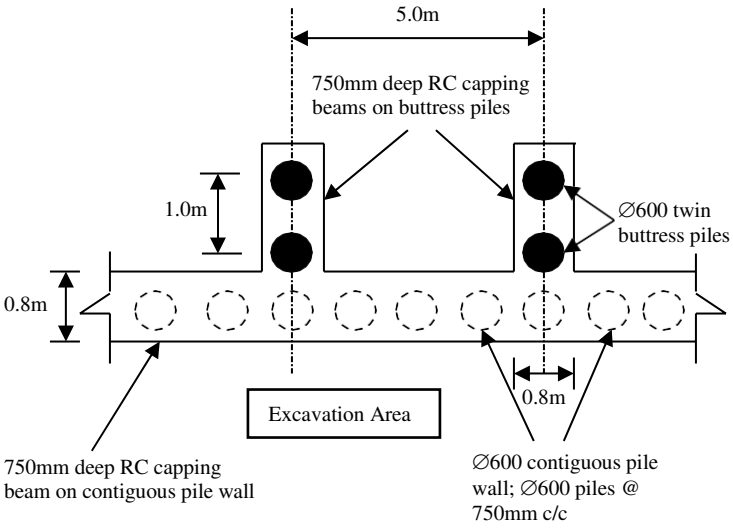


Figure 7a – Layout of contiguous pile wall and buttress piles on site 2 (Enfield, Greater London).

The system of twin buttress piles is based on a push-pull mechanism such that the buttress pile nearer to the wall experiences higher compressive loading than the other, while the buttress pile farther away from the wall is subject to higher tension than the other. The wall relies on the lateral and rotational stiffnesses of the buttress system to limit lateral movement and maintain overall stability. The buttress piles were designed to support tension, compression, bending and shear. The capping beam on the buttress piles was designed to support tension, bending and shear, while the capping beam on the wall was designed to support torsional moment, bending moment and shear.

Wall lateral movement during and after excavation was monitored by optical survey, while tiltmeters were installed on the adjacent buildings to monitor their vertical displacements. Generally, measured wall and building deflections were less than 50% of values predicted with the FE model. Maximum recorded wall deflection was 4mm, while maximum measured building movement was 2mm. In essence, the system was noted to be a safe and effective unconventional solution. See Adekunte, 2013 for more comprehensive details on the FE modelling, design, construction and monitoring of the earth retention system on site 2.

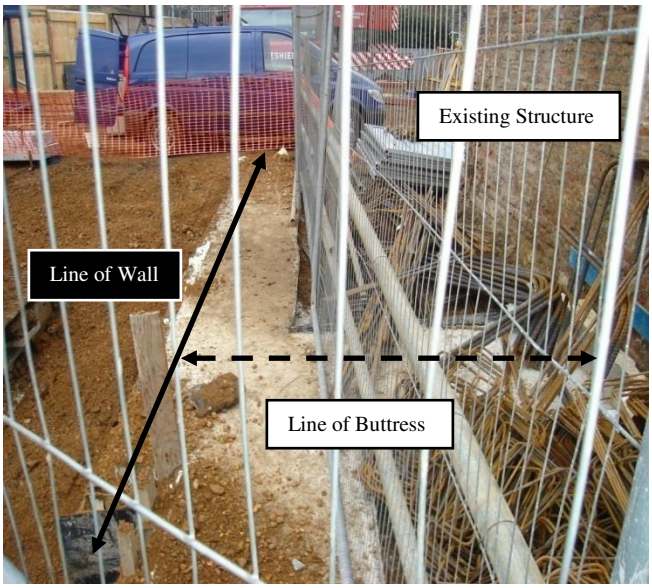


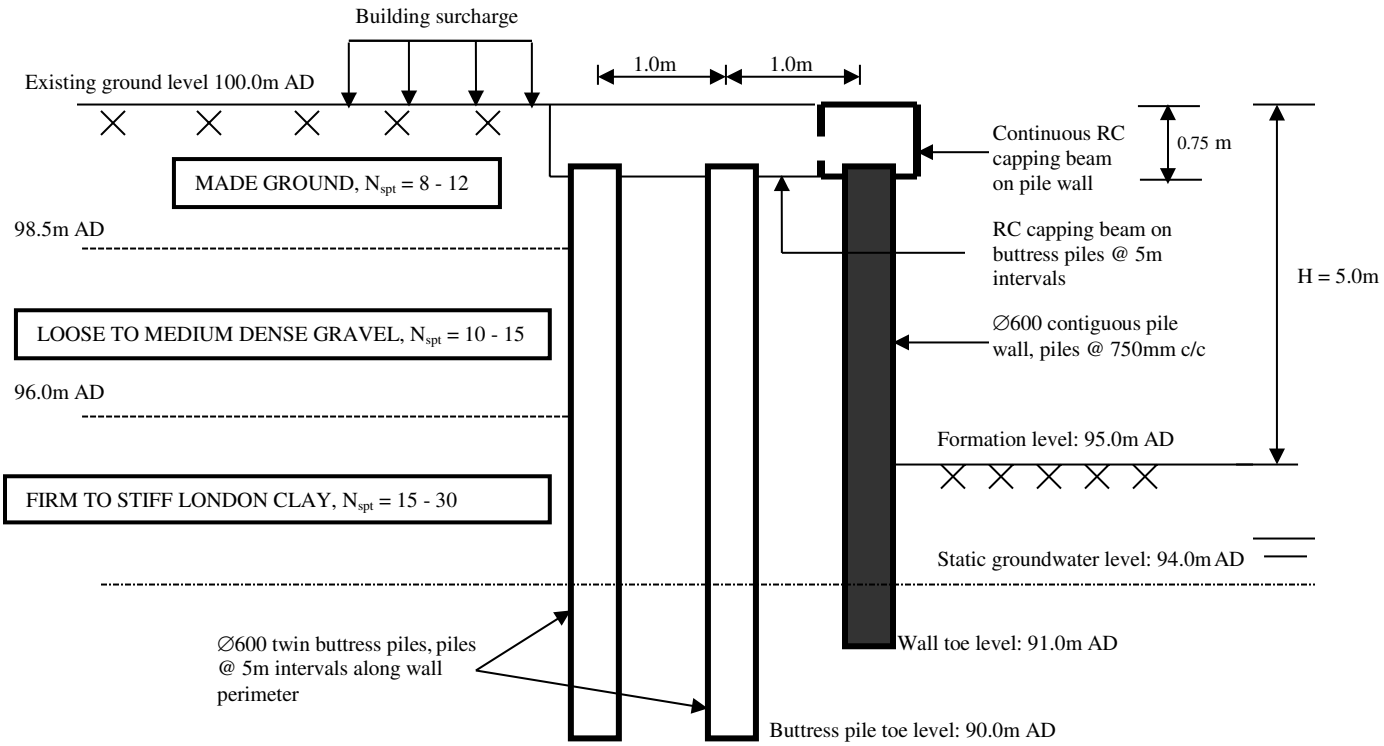
Figure 7b – Typical wall-buttress system on site 2 (Enfield, Greater London).

SITE 3

Major Complexities: Underground Obstruction and Restrictions to Applicable Wall Restraining Methods

Proposed Development & Construction Procedure:

The project on site 3 was centred on the development of a new Roll-on Roll-off (Ro-Ro) ramp for cruise liners coming into Dublin Port. This required the construction of a contiguous pile wall of up to 7m retained height to accommodate the link span bridge. Wall sections with retained heights of less than 6m were designed as free standing cantilever. However, due to the existence of old buried caissons and walls below 10m depth in the 6m-7m high zone, sufficient embedment could not be achieved to allow the wall in this zone to act as free standing cantilever. Therefore, this area of wall had to be supported with a restraining system.



N_{spt} – Average corrected standard penetration test N value, R* - Refusal.

Figure 7c – Typical section of earth retention system on site 2 (Enfield, Greater London).

Due to the existence of sensitive underground services behind the wall, as well as access constraint for an anchoring rig, tie-back anchors were inapplicable. In addition, temporary props were inapplicable as the wall required a permanent restraining system, while no restraint could be allowed in front of wall as the link span bridge had to be accommodated in front of the wall. As an alternative, a novel restraining solution was developed and adopted. This comprised of a permanent lightly reinforced concrete counterweight structure dowelled to the contiguous pile wall at 3 levels; just above formation level, mid-height and capping beam level. An illustration of the system is shown in figure 8.

The construction involved the simultaneous excavation of both sides of the wall, while shoring the excavation behind wall with the braced timber formwork for the counterweight; an arrangement similar to a trench-boxing system. This was followed by the pouring of the concrete for the counterweight.

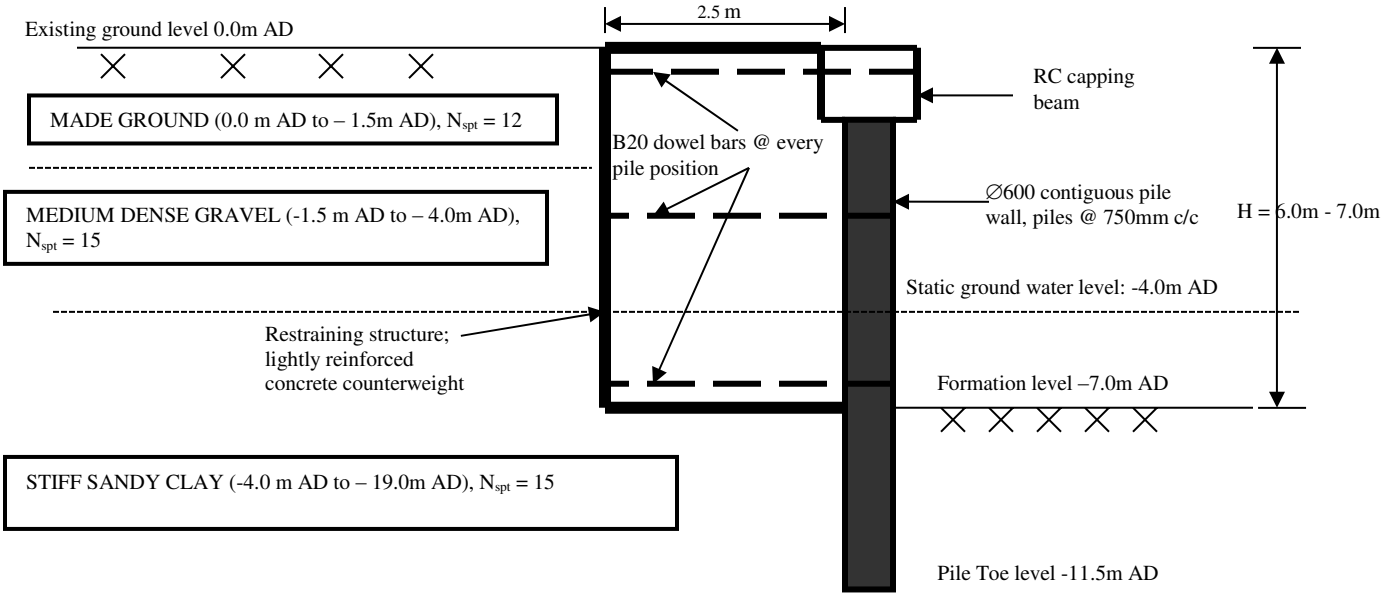
In order to limit active thrust behind wall during the pouring, setting and hardening stages of the concrete, two important decisions were made;

- the concreting of the counterweight structure was limited to a maximum pour of 2m over a 24 hour-period;
- concrete was mixed with rapid hardening cement.

The counterweight structure serves two main functions;

- it provides additional gravitational support to the contiguous pile wall;
- it increases the flexural rigidity and stiffness of the wall above formation level and thus limiting potential wall lateral movement.

Wall lateral deflection was monitored by optical survey over a 7 month-period. Generally, measured deflections were less than 40% of numerical analysis predictions. A maximum wall lateral displacement of 12mm was recorded.



N_{spt} – Average corrected standard penetration test N value.

Figure 8 - Typical section of earth retention system on site 3 (Dublin Port, Ireland).

However, it was observed that almost 60% (or 7mm) of this cumulative movement occurred during the pouring and hardening stages of the concrete for the counterweight. This observation justifies the above steps taken to limit active thrust from fresh concrete. It is presumed that higher and possibly non-serviceable movement could have occurred if this factor had not been accounted for. Figure 9 shows the finished wall. See Adekunte et al. (2010) for more in-depth details of the numerical modelling, design, construction and monitoring of this novel solution.

SITE 4

Major Complexities: Serviceability Requirements & Restrictions to Applicable Wall Restraining Methods

Proposed Development & Construction Procedure:

The project on site 4 (Cavan, Ireland) was a multi-storey commercial development comprising of a single to triple level basement. As groundwater control was not an issue on the site, contiguous pile wall was considered to be the most economical and effective excavation support system for the site. The wall was designed to act as a free standing cantilever in the single level basement area. However, due to the close proximity of an existing hotel and some commercial buildings to the double-triple level basement area, serviceability within and outside the site was a major concern. To mitigate this risk, it was important to support this section of wall with a restraining system. The supervising Engineer for the project had specified a maximum allowable wall deflection of 15mm in wall sections adjacent to existing structures.

Owners of the adjacent properties did not approve the use of ground anchors, while the main contractor was not in support of the use of temporary props and braces, so as to have adequate working space within the basement area. As an alternative solution, a novel wall restraining system was conceptualised and applied. This method involved the installation of steeply raking bored tension piles at 5m intervals behind the wall. These buttress piles were connected to the contiguous pile wall by a reinforced concrete capping beam. Figure 10 shows a typical section of the wall and the restraining system. The buttress piles serve two principal functions;

- the dead weight of each buttress pile provides additional gravitational support to the wall;
- the initial deflection of the wall at the start of excavation mobilises skin friction around the shafts of the buttress piles. The mobilised shaft resistance limits the movement of each buttress pile and the overall deflection of the wall as a whole.



Figure 9 – Finished wall on site 3 (Dublin Port, Ireland).

Buttress piles were designed to support tension only, while the capping beam was designed to support torsion, shear and bending moment. 6 No. inclinometers were installed in selected piles to monitor wall deflection. Figure 11 shows the contiguous pile wall after completion of bulk excavation.

As illustrated in figure 12, monitoring results show that in the single level basement area, which was a free standing cantilevered section, with a maximum retained height of 4m, a maximum lateral deflection of 12mm was recorded. However, in the double to triple level basement area, where the maximum retained height was almost 3 times the single level basement height, a much lower displacement of 6mm was measured. This observation shows how effective the novel wall restraining solution was, in respect of serviceability and overall stability.. See Adekunte (2008) for more detailed discussions on numerical analysis, design, construction and experimental results recorded on site 4.

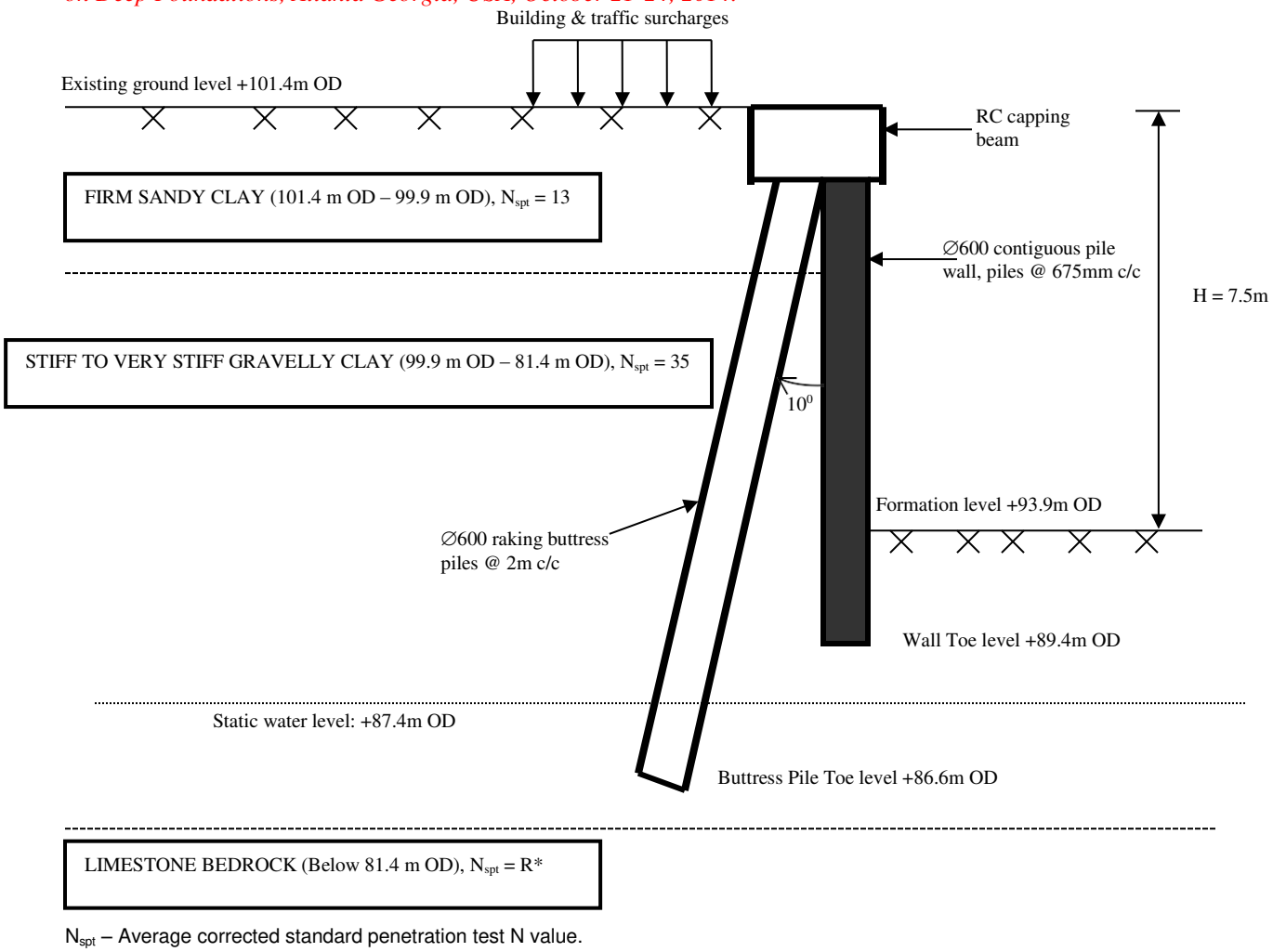
CONCLUSIONS

Intricacies associated with the application of bored pile retaining walls in urban environments have been identified. These include vertical & horizontal installation tolerances, accurate estimation of interlock thickness & pile spacing required for effective groundwater cut-off, uncertainty surrounding the prediction of axial capacities of pile retaining walls, strict serviceability requirements, restrictions to applicable wall restraining methods and difficulties posed by underground obstructions. All the common complexities have been highlighted by direct references to recent real life case histories, while alternative solutions have also been presented.

The dearth of publications with comprehensive specific information on the estimation of minimum interlock thickness and pile spacing for effective groundwater cut-off, to acceptable levels of accuracy is highlighted. Simple empirical equations for estimating minimum bite thickness and pile spacing for groundwater cut-off have been presented and validated by reference to a recently completed multi-level basement project.

Estimation of axial capacities of bored pile retaining walls is an area covered in uncertainty as walls are rarely tested, while existing knowledge of pile group efficiency of multi-grid pile groups is not particularly applicable to perimeter pile groups, such as bored pile retaining walls. An alternative approach for estimating the vertical capacities of pile retaining walls has been presented. This has been shown to be considerably less conservative when compared with a common routine design approach, while predictions of the method are shown to be in reasonably good agreement with the results of a recent research-based small scale model tests on perimeter piles.

Three alternative means of restraining pile retaining walls have been presented. These have been shown to be quite safe and effective at overcoming the shortcoming of traditional propping schemes and tie-back anchorage.



N_{spt} – Average corrected standard penetration test N value.

Figure 10 - Typical section of earth retention system on site 4 (Cavan, Ireland).



Figure 11 – Contiguous pile wall on site 4 (Cavan Ireland).

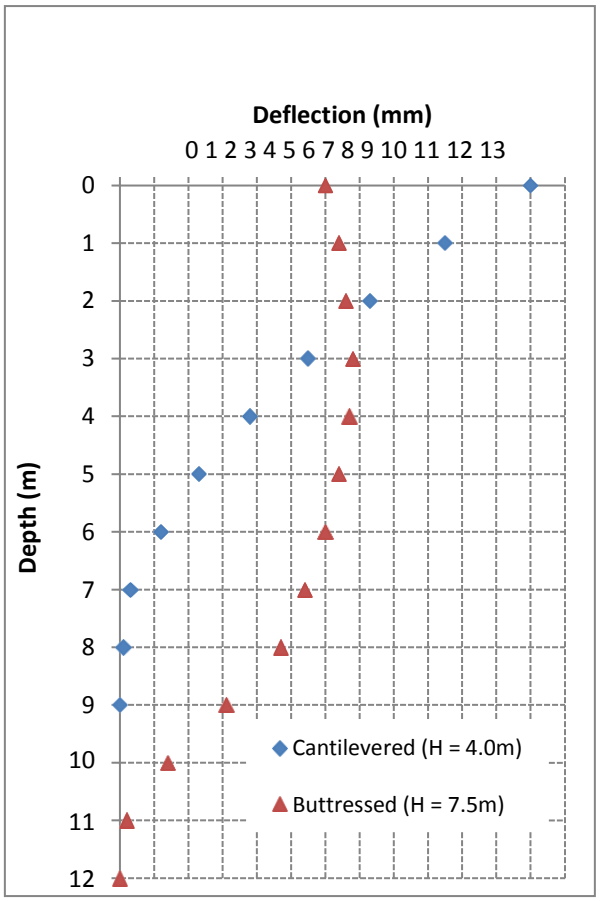


Figure 12 – Comparison of deflections in the free standing cantilevered section with wall movement in the buttressed area of site 4 (Cavan, Ireland).

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