RISKS ASSOCIATED WITH REUSE AND REMEDIATION OF FOUNDATIONS

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ABSTRACT

The reuse of foundations is not new and can take many forms, such as reusing existing foundations for increased loading, augmenting existing foundation systems and strengthening existing foundations to mitigate risks imposed by adjacent developments. Many developments in urban settings, particularly in city centres, are redevelopments with rigid geometric constraints that limit reuse options. The challenges become more acute with developments involving significant depths of excavation below the foundations of adjacent structures, services, tunnels and other infrastructure. These challenges can be further compounded by geotechnical and geological imperfections, including unfavourable ground and water conditions, as well as defects in the existing foundation systems.

This paper highlights the key risks associated with reuse and remediation of existing foundations. The risks can be wide ranging depending on the individual project but can be broadly categorised into specific areas, such as design and construction. The paper includes examples of two projects from the UK and UAE, with which the author was actively involved. The examples highlight the process through which the specific risks were identified and how these risks were adequately mitigated.

Keywords: reuse, risk, imperfections, piling, retaining walls

INTRODUCTION

With the ever-increasing demand on limited urban space, and the awareness of the need to reduce carbon footprints, there has been a focus on achieving sustainable solutions through innovative application of engineering knowledge. This provides a great stimulus to expand our knowledge in the area of foundation reuse.

There have been many notable examples of the reuse of existing foundations and retaining walls in recent years, particularly in the commercial sector of city centres. These include Pinnacle / 22 Bishopsgate (Patel, 2015), One Leadenhall (Torial, 2021), Triton Square office block (Pitcher, 2022) and Battersea Power Station (Smith, 2015) in London, UK.

In addition to geometric constraints, the challenge associated with reuse of foundations is often compounded by the presence of imperfections. These imperfections can be of varying natures, including naturally occurring ones, such as solution features (sink holes), scours, and significant variability in stratigraphy and soil strengths across a site. Imperfections may also arise from design deficiencies and poor construction practices. There have been a number of documented cases where imperfections dictated the development and implementation of significant remedial solutions, and in extreme cases, total abandonment of the project, e.g., Shatin Towers in Hong Kong (Poulos, 2009). The Shatin Towers suffered from differential settlements and tilting due to poor pile construction practices, which resulted in piles having soft toes, variable founding conditions and varying lengths within a group. In the case of the Millennium Tower in San Francisco (Menteth, 2022), the foundations are experiencing large and uneven settlements causing the tower to tilt. Remediation works are currently being implemented and comprise installation of new piles. These remediation works exemplify the challenges and risks associated with increasing foundation capacity. The current version of the remedial solution is based on a reduced number of piles and a revised pile installation methodology. During the implementation of the original measures, which involved excavation for a raft extension and pile installation, additional building movements were observed.

This paper presents two case studies that contain aspects of foundation reuse.

1) The Honourable Artillery Company (HAC) – Extension of the Albert Room – London, UK
2) ICD Brookfield Place Tower – FIDC, Dubai, UAE

These case studies were selected to illustrate reuse of foundations in two distinctly different scenarios, both in terms of size of the project and challenges encountered. The HAC project was relatively small, compared to the ICD Brookfield Place Tower, and involved shallow footings. The ICD project was much larger in size and had many more complexities.

CASE STUDY 1 – HAC (London, UK)

Project Background

The project comprised an extension of an existing 25 m long banquet hall, known as the Albert Room. It was to be extended to 53 m in length (14 m extension on either side). The works also included construction of a new, single level basement beneath the Albert Room footprint. The original Albert Room was completed in 1862. The Armoury House building, which houses the Albert Room, is a grade II heritage listed building (Schunmann, 2005).

The northern boundary of the Armoury House abuts the grade I heritage listed Bunhill Fields burial ground. Bunhill Fields (Fig. 1) first started being used as a dedicated burial ground in 1665 and this continued until about 1854. The Bunhill Fields cemetery is the resting place of a number of famous personalities, including John Bunyan, Daniel Defoe and William Blake amongst others, and no disturbance of the ground was permitted.

Fig. 1. Location of Albert Room and Bunhill Fields Cemetery

Geotechnical Conditions

The ground conditions were typical of the London region and comprised variable thickness of fill over river terrace gravel, underlain by London Clay, Table 1.

The water table was approximately 10 m below the existing surface.

Table 1. Summary of geotechnical conditions and adopted design parameters

Material	Description	Approx. layer thickness (m)	Unit weight (kN/m^3)	Drained cohesion c' (kN/m^2)	Internal friction angle (degrees)	Young's Modulus (kN/m^2)
Fill	Very loose to loose sandy gravel / clayey gravel.	3.0	18	-	25	10,600
Sandy Gravel	Medium dense to very dense sandy gravel (River Terrace Gravels)	6.0	20	-	36	75,000
Clay (London Clay)	Stiff to very Stiff, high plasticity Clay	13 (borehole limit)	20	-	22	37,000

Constraints

The key constraints were directly tied to the heritage nature of the Armoury house and the Bunhill Fields burial ground. These constraints imposed headroom restrictions of less than 5 m in the existing Albert Room, and prevented the use of normal height piling rigs and any disturbance of the burial ground.

There was a level difference of approximately 2.4 m between the burial ground and the footing levels supporting the existing masonry load bearing walls of the Albert Room. The new foundation system, required to facilitate the construction of the new single level basement, had to account for the surcharge load from the adjacent higher level of the burial ground – refer to Fig. 2.

In order to optimise the floor / basement space, the original proposal was to underpin (mass concrete) the existing foundations along the northern and the southern sides. However, the approval authorities were concerned that on the northern side (along the Bunhill fields burial ground), there was a risk of bones "dropping into the basement during construction," and consequently, the design had to be revised. The underpinning on the northern side was replaced with a contiguous piled wall, inside the building footprint, to avoid disturbance of the burial ground.

Risks

The main risk considered during design was the lack of sufficient lateral restraint of the foundations. It was important that the new piled wall was designed to support the lateral earth pressures and surcharge from the burial ground above the existing footing level, and also to be sufficiently rigid to prevent excessive lateral deflections and deformation of the listed masonry structure.

The main construction risks included:

- Movement or rotation of existing masonry walls during basement excavation.
- Damage to the existing heritage structure during piling operations.
- Excessive vibrations.
- Inability to support the pile bores during piling.
- Ability of low headroom rig to be able to bore through the dense to very dense gravels.

Solution and Mitigating Measures

The design solution for the southern wall was technically relatively straight forward and consisted of mass concrete underpinning of the strip footing of the existing masonry wall. However, the staging of underpinning required careful consideration.

The solution for the northern wall, which was adjacent to the burial ground, consisted of 135, 450 mm diameter piles at 550 mm centres, where the ground retention and existing footing support were the primary requirements. The pile lengths ranged between 8 m and 12.5 m. Larger 600 mm diameter piles at 750 mm centres were used to support new columns and there were nine of these piles in total. Some 450 mm diameter piles were also used at the southern end – refer to Fig. 3.

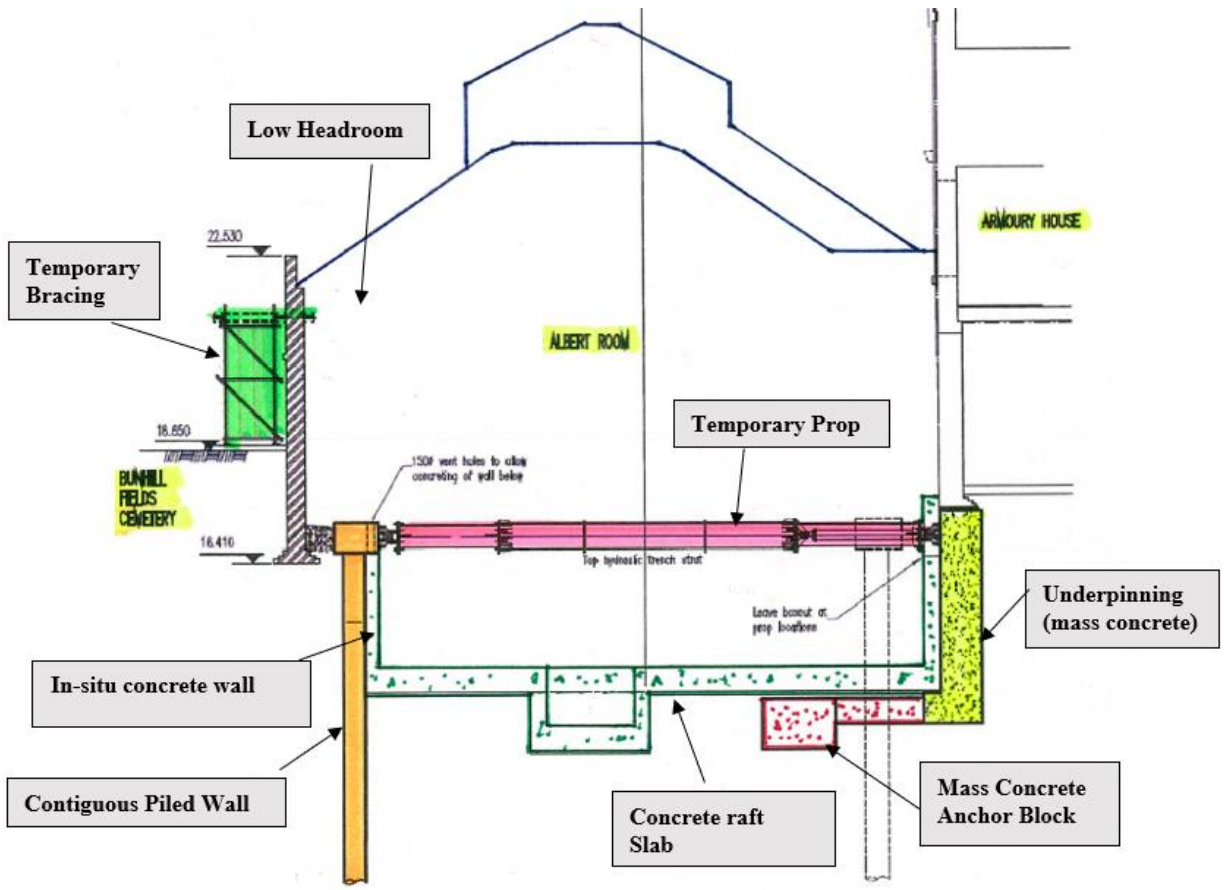

Fig. 2. Indicating low headroom constraints and components of the design solution

The basement excavation depth was approximately 4 m. To control lateral deflections of the contiguous piled wall, and reduce the risk of settlement on the burial ground side, a construction sequence based on employing temporary steel props was adopted to facilitate basement excavations. It was not feasible to use temporary prestressed active anchors into the burial ground, and the basement was not large enough to use a soil berm and raking struts propped off the basement slab. The commercially available WALLAP programme was used to analyse the piled retaining wall. The computed lateral deflections at top of the wall were less than 15 mm and the actual deflections were less than the computed values.

The mitigating measures against potential rotation of the underpinned southern wall and a portion of the eastern wall included construction of mass concrete anchor blocks connected with ground beams, which extended to the base of the underpinned walls – see Figs. 2 and 3.

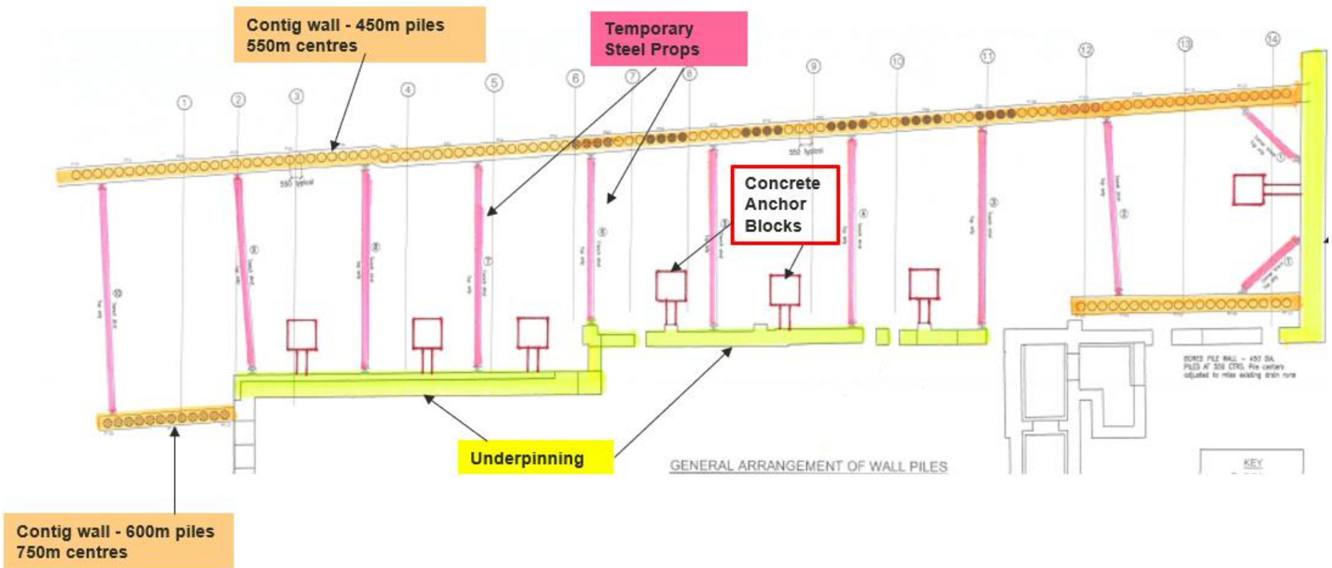

Fig. 3. Plan showing the adopted design solution.

In order to reduce the risk of bore instability during pile drilling and reduce excessive vibrations, the continuous flight auger (CFA) piling technique was adopted. A low headroom piling rig was used to maintain clearance to the heritage roof, see Fig. 4. Figure 5 shows a piled wall installed with temporary props in place. Areas on either side of the existing Albert Room had a greater headroom of 15 m, and consequently, it was possible to use a different rig (14.9 m height) to install piles. The CFA rig was able to drill through gravels and careful execution of works ensured that there were no adverse impact on the heritage elements.

Fig. 4. CFA rig in low headroom mode

Fig. 5. Excavated basement

CASE STUDY 2 – Dubai, UAE

Project Background

The redevelopment of the site in Dubai by ICD Brookfield comprised a 290 m tall tower with a three-level podium. The ICD Brookfield Place development was constructed on a site previously intended as the location of the "Lighthouse Tower" in Plot GB03 (Fig. 7) of the Dubai International Finance Centre. Bored piles and anchored diaphragm walls (D-walls) were installed and completed for the previous development in 2008, but further work was halted due to the Global Financial Crisis in 2009.

The previously constructed enabling works comprised a five-level basement (i.e., a maximum of 21 m below the existing ground level) with temporary anchored retaining walls along three sides (toe level of 23.5 m Dubai Municipality Datum [DMD]), along with all the foundation piles for the intended Lighthouse Tower. The basement was later backfilled when construction ceased in 2009. The anchors, D-walls and piles were left in place, see Fig. 6.

Fig. 6. Completed enabling works (2009) for the previous Lighthouse Tower development.

In order to achieve an attractive frontage to the structure, the construction of a seven-level basement was required to avoid housing above ground parking. This meant a 28 m to 30 m deep basement (at the time of construction the deepest in Dubai) increasing the depth of the existing basement by approximately, by 8 m to 10 m. The new proposed development was larger than the previous Lighthouse Tower development, and included the adjacent Plot GB04, See Fig.7. The ICD Brookfield Place Tower comprised a single tower, in a different location from either of the previously planned twin towers. The bulk excavation level of the new basement was below the toe level of the D-walls of the former basement. The toe level of the new basement level was -36 m DMD. Thus, the completed works for the previous development provided considerable constraints to the construction of the new basement. Particular challenges included the deepening of the basement, the construction of new and deeper temporary retaining walls, pile reuse, and construction in close proximity to adjacent structures.

The main matters that needed to be addressed were the location, extent and form of the new temporary retaining walls to enable the new and deeper basement excavation. Associated with these elements were questions pertaining to the extent of dewatering that would be required and whether or not it might be necessary to remove any of the existing piles to avoid clashing with the new wall.

Due to limitations of the length of this paper, only issues associated with the existing piles have been discussed. However, a detailed account of how the new basement was deepened is provided in a separate paper (Smith et al., 2021**),** including how the strategy for the reuse of the existing D-walls and how the selection of the new temporary wall for the new deeper basement were decided.

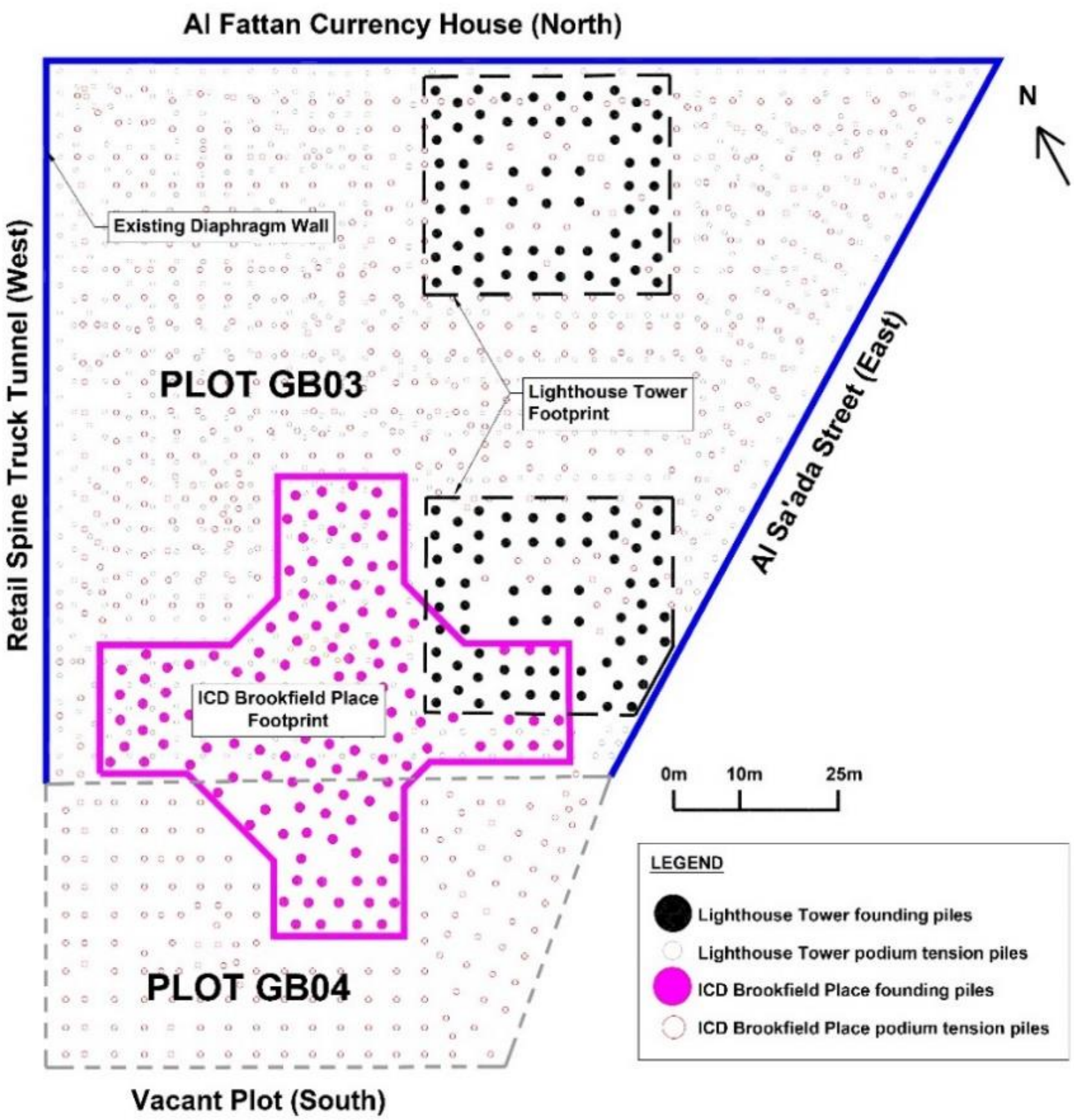

Fig. 7. Shows extents of the former (Lighthouse) and ICD Brookfield Place Tower developments

Geotechnical Conditions

The ground conditions were typical of this area of Dubai. The stratigraphy was relatively uniform across the site and is presented in Table 2.

Table 2. Summary of geotechnical conditions

Material	Description	Appx. Layer thickness (m)
Sand	Silty / slightly silty, medium dense to dense, fine to medium grained SAND	4.5 to 8.8
Dense Sand	Fine to medium grained dense SAND with gravel sized fragments of SANDSTONE	0.3 to 3.34
Sandstone / Calcarenite	Very weak to weak calcareous SANDSTONE/CALCARENITE	23.0 to 27.5
Conglomerate	Weak CONGLOMERATE	1.4 to 9.6
Siltstone	Weak, gravelly, calcareous, interbedded SILTSTONE	>80

Constraints

The main construction constraints for the deepening of the basement were associated with the presence of piles in the previous basement floor, the existing D-walls and the existing high groundwater table. All of these factors interacted in a complex manner.

Additionally, there were several geometric constraints along the different boundaries of the basement that had to be considered in the development of the temporary basement wall, including adjacent piled structures through which the temporary anchors of the existing D-walls had been threaded, and variable spacing between the previously installed basement piles and the existing D-walls.

Risks

The existing piles had the potential advantage of being reused and incorporated into the new development, but at the same time their location presented a significant challenge for the construction of the new retaining wall to deepen the existing basement. In order to create sufficient space to install the new retaining wall, it would have been preferable to extract a number of piles to optimise the dimensions of the new basement. Extraction of piles would also be advantageous where the existing piles clashed with the new structure. Thus, the feasibility of pile extraction had a significant influence over the design option for the construction of the new basement wall in Plot GB03.

The following risks had to be considered for extracting piles:

- Proximity of the pile to be extracted to the existing D-wall (e.g., workability space).
- Pile verticality: even a small deviation from the vertical could result in the pile toe being below the existing retaining wall and adjacent structures.
- Force required to extract piles and ability of the available plant to do so.
- Protection of existing diaphragm wall during pile extraction.
- Control of groundwater (dewatering versus working from a higher level).
- Controlling loss of passive resistance at D-wall toe.

The feasibility of reusing the piles was based on the consideration of the following risks:

- Verticality of existing piles.
- Structural integrity of piles.
- Durability considerations (concrete cover, steel corrosion).
- Load carrying capacity (tensile and compressive).

The existing piles also had the potential to adversely impact the installation of new piles due to physical constraints and generate interaction effects.

Solution and Mitigating Measures

The feasibility assessment to incorporate the existing piles into the new foundation system commenced with the review of the as-built information available on the existing piles. Table 3 summarises details of the existing piles, for the Lighthouse Tower.

Table 3. Details of existing piles

Pile Type	Diameter (mm)	Approximate number of piles	Pile length (m)	Pile Working Load (kN)	
				Compression	Tension
P1	750	680	16	2,600	1,670
P2	900	78	31	9,000	3,000
P3	1500	132	47.3	27,500	2,500

The existing piles ranged from 750 mm to 1500 mm in diameter and 16 m to 47.5 m in lengths. The piles were installed in a grid formation, with centre to centre spacings ranging from 1.5 m to approximately 4 m. Neither the specified limits of verticality nor the values actually achieved for the existing piles and D-walls were known. The ICE Specification for Piling and Embedded Walls, second Edition (2007), Table B1.4, prescribes a maximum deviation from vertical of 1:75 for all types of bearing piles and retaining walls. However, it should be noted that the actual values were not known nor could they be measured.

Clearly, potential deviation of any new wall also had to be taken into account. An assessment was undertaken on the basis that the new wall construction would just satisfy the ICE specification.

In order to assess the scope for locating the new wall, an exercise was undertaken to assess constraints posed by the existing piles. Fig. 8 is an extract from an as-built plan showing a typical short section of the existing wall and the adjacent existing piles. A circle of uncertainty was drawn around each pile on the assumption that it might have the maximum inclination from the vertical permitted by the ICE specification. Two lines at a tangent to these circles formed the boundaries of a corridor along which the new wall could be constructed. These boundaries were then plotted on schematic plans with the distance from the existing walls at an exaggerated scale. Figure 9 shows the plan for the east wall. One of these plans was constructed for each of the walls.

The exercise demonstrated that it should be practically feasible to install an 800 mm thick D-wall around most of the perimeter of the excavation, with only two locations (one can be seen in Fig. 8, towards the south end of the east wall) where there was no easy route between the existing piles, and it necessitated the need for a secant piled wall to avoid the existing piles. However, after exposing the tops of the piles it was feasible to do D-wall installation. The exercise also demonstrated that given the risks associated with the extraction of piles, there was little to be gained from progressing down the pile extraction path.

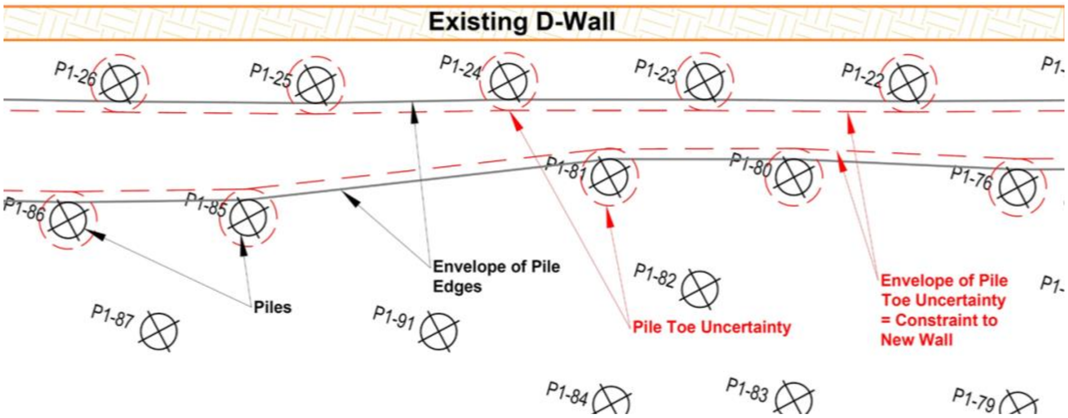

Fig. 8. Assessment of scope for location of new wall based on verticality of existing piles.

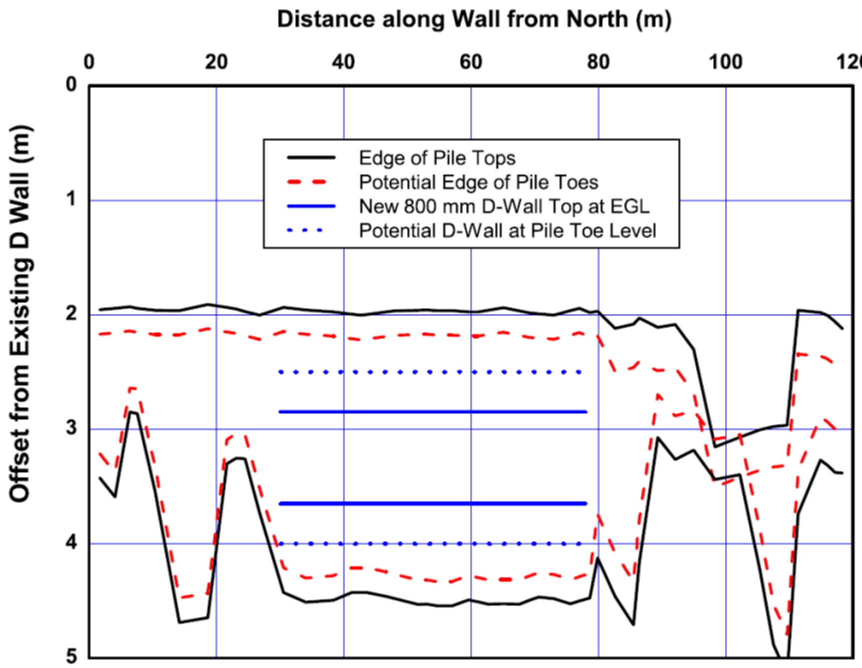

Fig. 9. Scope for installation of new wall along the eastern boundary.

The D-wall solution for the new basement was able to be adopted along all four sides to successfully facilitate the deepening of the basement.

Due to the new proposed basement level being approximately 8 m to 10 m lower than the existing basement level or top of the existing piles, all the piles were required to be cut down. In the case of pile type P1 (750 mm diameter), which was primarily the tension pile, the residual tension capacity of theses piles following trimming down was not sufficient for them to be reused. P1 piles were trimmed down so that their tops were clear of the underside of the new hydrostatic basement slab. However, some of the pile types P2 and P3 were able to be reused.

In order to verify the load carrying capacity and structural integrity of pile types P2 and P3, a targeted testing campaign was developed. The testing regime comprised 15 compressive strength tests (three tests in each of the five selected piles) from concrete specimens taken from 80mm diameter cored samples, three tensile tests of the steel reinforcement and visual inspections for corrosion in the reinforcement. Three out of five piles selected for the verification purposes were cored through their entire lengths and two piles were subjected to load testing.

A total of two compression pile load tests were carried out, one for each of the two larger pile diameters, i.e., 900 mm and 1500 mm. The two selected test piles were subjected to maintained load tests, using a conventional reaction pile system. Two and four reaction piles were used for 900 mm diameter and 1500 mm diameter piles, respectively. Figures 10 and 11 show the typical pile test load set up for the 900 mm diameter pile (two reaction piles). The two pile load test results are presented in Table 4.

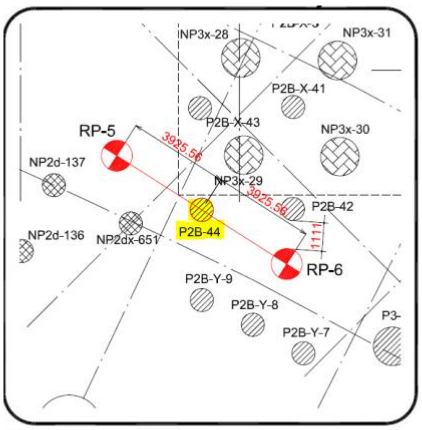

Fig. 10. Pile load test layout

Fig. 11. Pile load test set up with two reaction piles

Table 4 Compressive static maintained pile load test results

Test Pile Ref:	Pile diameter (mm)	Working load (kN)	Test Load (kN)	Vertical settlements (mm)			
				At working load (1st Cycle)		At test load (2nd Cycle)	
				Total	Residual	Total	Residual
P2B-44	900	6,500	9,750	2.20	0.14	3.86	0.72
P3A-10	1500	14,500	21,750	2.48	0.15	4.28	0.33

The concrete compressive strengths and reinforcement tensile strengths were found to be well in excess of 60MPa and 550MPa, respectively. The two maintained load tests performed satisfactorily under the working and test loads. The load versus vertical settlement curve indicated a relatively stiff behaviour, which provided the confidence and assurance for reusing the existing piles. Figure 12 shows the tower under construction and Figure 13 shows the completed ICD Brookfield Place Tower.

Fig. 12. The new tower under construction

Fig. 13. Completed Tower

SUMMARY

Two contrasting case studies have been presented in which foundations were reused.

First, a case study from London, UK, demonstrated the specific constraints imposed by the heritage status of the subject site, the Albert Room, as well as the adjacent listed Bunhill Fields burial ground. Key risks, such as the instability of the existing masonry wall and a 4 m deep excavation to form the new basement without disturbing the adjacent burial ground, were adequately addressed by adopting a stiff contiguous piled wall along the boundary adjacent to the burial ground. The solution relied upon using a temporary steel prop to limit deflections to less than 15 mm, and using a low headroom piling rig in the CFA mode to ensure pile bore stability and low vibrations during piling operations.

The second case study, from Dubai, demonstrated how the presence of an existing 20 m deep basement and foundation piles from a former development posed risks to the new development. The new ICD Brookfield Place development consisted of a single 290 m tall tower with a 10 m deeper basement. A detailed assessment of construction tolerances aided in deciding the alignment and thickness (800 mm) of the new basement D-wall. The reuse of some of the 900 mm and 1500 mm diameter piles from the former development was confirmed through targeted testing. The testing confirmed that concrete compressive and steel tensile strengths were greater than 50 MPa and 550 MPa, respectively. The two static pile load test results indicated relatively stiff load settlement behaviour with settlements under working load of 2.2 mm and 2.48 mm for 900 mm and 1500 mm diameter piles, respectively.

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REFERENCES

ICE (2007) Specification for Piling and Embedded Retaining Walls, 2nd edition. ICE Publications, London, UK.

Menteth. T., 2022. ‘San Francisco’s troubled Millennium Tower continues to sink’ Ground Engineering (GE) (geplus.co.uk), 16 May.

Patel, D., Glover, S., Chew, J., Austin, J.,2015. The Pinnacle- Design and Construction of large diameter deep base grouted piles in London, Ground Engineering, November 2015, pp. 24-31

Pitcher. G., 2022. ‘Pile reuse: Building on past glories’, Ground Engineering (GE) (geplus.co.uk), 14 November.

Poulos, H. (2009). Simulation of the performance and remediation of imperfect pile groups. In W.F. van Impe and P.O. Van Impe (Eds.), Deep Foundations on Bored and Auger Piles. Proceedings of the fifth international symposium on deep foundations on bored and auger piles (BAP V), (pp. 143-154). London, UK: Taylor and Francis.

Schunmann D.,2005. ‘The Honourable Artillery’ new banqueting hall’, Ground Engineering, October, pp.19.

Smith, A., Blackmore, R., Tay. Y., and Sethi, P., 2021. Planning the construction of a deep basement on the site of a previous retained excavation. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering, Vol. 174 Issue 4, August, 2021, pp. 461-472.

Smith. C., 2015. ‘Foundations: Battersea Power Station redevelopment underway' Ground Engineering (GE) (geplus.co.uk), 28 October.

Torial. G., 2021. ‘Piling and foundations: Enhanced design’ Ground Engineering (GE) (geplus.co.uk), 20 September.