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Re-use of existing piles, Belgrave House, London

La reutilization des pieux existants, Belgrave House, a Londres

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ABSTRACT
This paper describes the application of re-using piled foundations to a development site in London, Belgrave House. The project involved the demolition of an existing seven-storey building followed by the construction of a new six-storey building. The existing building was founded on straight-shafted and under-ream piles in London clay.
The paper discusses the advantages of re-using the existing piles, highlights the principal issues identified and describes the strategy adopted to confirm that piles are suitable for re-use. Investigation works comprised visual inspection of piles under the existing building, integrity testing on piles both before and after demolition of the building and laboratory testing on cored samples.

RÉSUMÉ
Cette publication décrit la réutilisation des pieux existants dans le cadre du redévelopement d'un site (Belgrave House) à Londres. Le projet implique la démolition du bâtiment existant qui comprenait sept étages, suivie par la construction d'un bâtiment neuf de six étages. Le bâtiment existant était fondé sur des pieux à base élargie, ancrés dans l'argile de Londres.
Cette publication adresse les avantages de la réutilisation des pieux existants, souligne les principales difficultés et décrit la stratégie adoptée afin de confirmer l'adéquation des pieux pour leur réemploi. Les investigations ont compris l'auscultation visuelle des pieux sous le bâtiment existant, des tests visant à vérifier l'intégrité des fondations avant et après démolition du bâtiment et des essais en laboratoire sur des échantillons carottés.

1 DESCRIPTION OF THE PROJECT

The site is located on Buckingham Palace Road and is opposite Victoria Railway Station in London. The site is approximately rectangular (110m long and 43m wide) in plan and is bounded by Buckingham Palace Road to the east, Eccleston Place to the west, Phipps Mews to the north and Chantry House to the south. The London Underground Circle and District line passes at shallow depth along the Western boundary of the site. In addition, to the east of the site is the possible route of the second Crossrail line at a depth of approximately 36m below ground level. Figure 1 shows the location of the site and the nearby underground tunnels.

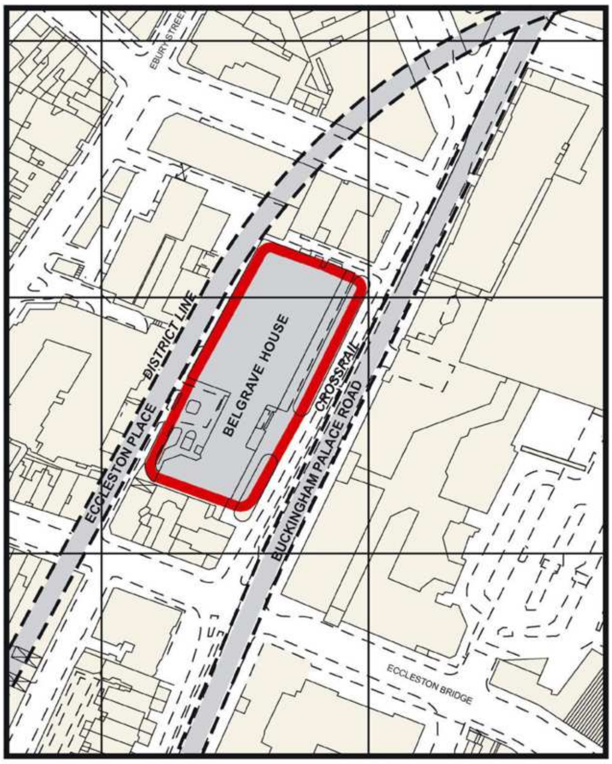

Figure 1: Site Location Plan

The Belgrave House project involved the demolition of an existing seven-storey concrete framed structure with a single-storey basement followed by the construction of a new office building. The new development comprises a six-storey steel framed structure with a single-storey basement (Figure 2).

Figure 2: New Structure

Whitbybird design responsibilities included:

- demolition of the existing building, including basement and piles caps with the retention of all the existing piles

and parts of the existing retaining walls below street level
- installation of reinforced concrete transfer slab, pile caps and retaining walls
- new underground drainage pumped and discharged to the sewers using new and existing connections
- seven-storey composite steel and concrete superstructure supported on new and existing piles
- mechanical and electrical services
- design of the structure within the constraints set out by London Underground Limited and Cross London Rail Links Limited

Building works on site started with demolition in November 2001 and the construction was completed by winter 2003.

2 GROUND CONDITION

Ground conditions comprise made Ground underlain by River Terrace Deposits and London Clay with the groundwater level at about a meter above the top of London Clay. Table 1 summarises the ground conditions as encountered from the site investigation.

Table 1: Summary of Ground Conditions Encountered

Stratum	**Typical description of material**	**Top of stratum (m bgl)**	**Thickness (m)**
Made Ground	Brown very sandy gravel FILL with cobble and boulder size fragment of concrete	Ground level	1.7 to 8.2
River Terrace Deposits	Medium dense to very dense sandy or very sandy GRAVEL	3.3 to 8.2	1.2 to 4.8
London Clay	Stiff, very stiff grey very closely fissured sandy CLAY	7.5 to 9.4	Greater than 45

3 FOUNDATION CONSTRAINTS

The presence of existing piled foundations at Belgrave House imposed restrictions on the design of the new building. The existing piles which were constructed in the early 1970's are generally under-ream piles up to 2800mm in diameter with a corresponding 1200mm shaft diameter. The piles supporting the original lift cores are 600 and 900mm in diameter. The existing pile layout plan is shown in Figure 3.

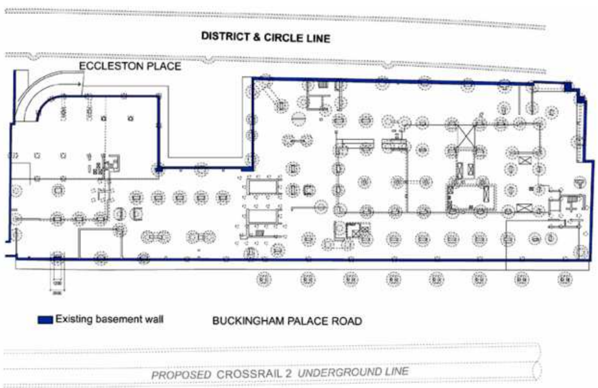

Figure 3: Existing Basement Plan

If the design strategy ignored the existing piles then the new piles would need to avoid the existing pile locations. However this criterion is unlikely to be satisfied at all grid locations, as some of the existing under-reamed piles would provide obstruction to the new piles. Installing additional piles in between the existing ones restricts flexibility in any future development; as the number of piles increases, the option of avoiding them will become increasingly expensive.

Removal of old piles could have been achieved by over-coring the pile shaft and progressively removing the piles. Removal of the under-reams is an expensive operation. The piles at Belgrave House are not reinforced to their full depth and hence progressive removal of piles from the ground would have been problematic. Pile removal involves drilling and the use of water may cause the clay to swell and hence result in a reduction in the frictional capacity of the new piles installed at or near the same location.

Reuse of existing foundations is generally more beneficial and certainly more cost-effective than ignoring and/or removing them. For the successful reuse of existing piles it is important to understand how the present piles are likely to behave when subjected to unloading by demolition of the present building and reloading by the new development.

As the existing column grid at Belgrave House did not coincide with the new column grid, new structures (e.g. transfer slab, transfer beams, capping beams, etc.) were needed to distribute loads to the existing piles. In few areas where this was not possible or if there was a risk of over-loading the existing piles then isolated new piles were installed. The impact of differential movements between the existing and new piles was assessed where isolated new piles had to be used.

4 STRATEGY FOR THE REUSE OF PILES

4.1 *The Principal Benefits*

As part of the development of options for the new building the following foundation options were considered:
(i) New piled foundations,
(ii) raft foundation,
(iii) re-use of the existing piles.

The principal advantages of re-using existing piles were identified at the outset as follows:
- Reduction in the construction programme;
- Less risk to the development arising from obstructions in the ground;
- Lower foundation costs.

However with these advantages there were also a number of issues that had to be addressed in order to make the re-use of Piles a viable foundation option:
- Quality of design information on the existing piles
- Integrity of existing Piles
- Design Responsibility
- Impact of demolition of existing building on the Piles
- Design Life of the Piles
- Insurance Concerns
- The new column grid location in relation to the existing piles (risk of overloading)

The following section highlights the issues that need to be addressed for reuse of the piles and describe the strategy adopted to confirm that the piles are suitable to be used for the new development.

4.2 *Quality of Existing Information*

The consulting engineers for the original building were Ove Arup & Partners. Whitbybird enquired about the possibility of obtaining original design information on the project and later commissioned Ove Arup Ltd. to carry out a search of their archive department and consequently copies of the original design drawings for the period 1972-1976 were obtained. Although these were design drawings rather than as built drawings the information provided was found to be very useful in assessing the feasibility of re-using the existing piles.

The drawings showed the piles used for the existing building were of four types, three of which were under-reamed (Figure 3). The pile sizes ranged from 600mm diameter straight-shafted piles up to 1200mm diameter piles with 2800mm under-reams. Piles were generally shown to be 20m in length. The pile toe levels were shown at –19m O.D with requirements that the piles are founded a minimum depth of 16m into London clay, a minimum concrete strength of $25N/mm^2$ and a total reinforced length of 10m. A safe working load factor of 2.5 was used in the design of the piles. The Ove Arup & Partners drawings dated May 1976 and it was assumed that construction followed soon after this date.

There were only a small number of contractors that were able to construct under-reamed piles in the 1970s. The original designers were contacted to establish whether any "as built" records or "pile supervision close out" reports were available. The archive search showed that the piling contractors were Cementation Piling and Foundations Ltd and after consultation with them we were not able to find any of the original piling records.

At this stage in the design development there were no obvious reasons to believe that the piles were not constructed as shown on the drawings. The Super Structure design drawings agreed well with the visual inspection. There was a risk that the geometry of the piles would be different to that shown, as occasionally piling contractors will offer an alternative design by considering the type of plant or materials they have available at the time. Notwithstanding possible differences in pile geometry, the pile safe working loads were unlikely to differ from the design, unless the piling contractor became involved in redefining the structural load paths, which is very unlikely, indeed unrealistic. Therefore it was reasonable to conclude that the load carrying capacity of the piles is as shown on the drawings.

Table 2: Results of Back Analysis of Existing Piles

Pile Type	Shaft diameter [mm]	Under-ream diameter [mm]	Safe Working Load (SWL)	
			Back-analysed[1] [kN]	Original [kN]
100	600	-----	1180	1170
200	900	2300	4230	3850
300	1200	2600	5400	4970
400	1200	2800	6000	5580

4.3 *Load Carrying Capacity of Existing Piles*

Based on the original design drawings and our understanding of the ground conditions at the site the load carrying capacity of the existing piles were back analysed in order to confirm the pile load capacities shown on the original drawings. Table 2 summarizes the back analysed pile safe working loads and compares them with the original safe working loads quoted in the original drawings.

(1) Back analysed SWLs based on the following assumptions:

a)

$$SWL = \frac{Q_{bf} + Q_{sf}}{2.5} \; or \; \frac{Q_{bf}}{3.0} + \frac{Q_{sf}}{1.5} \qquad (1)$$

Qsf = Ult capacity of the Pile Shaft
Qbf = Ult capacity of the Pile Base

(b) $\alpha = 0.50$
(c) Shaft load developed in 16m depth of London Clay
(d) Moderately conservative strength profile in London Clay defined as:

$$C_u = 85 + 6.8z kN/m^2, \qquad (2)$$

with z measured from 0mOD.

Good agreement was obtained for the 600mm diameter straight-sided piles; for the under-reamed piles the back-analysed safe working loads were 7-10% greater than the original design safe working loads.

The availability of the original information was an important aspect in the project because the size and location of piles could be considered in designing the foundation scheme without significant intrusive investigations/inspections being carried out. If the original drawings were not obtained then sufficient information would not have been available at the scheme design stage and this would have been unacceptable, because of delays in the construction programme and significant uncertainty in the design until part way through the construction process.

A visual inspection of the existing building showed no evidence of building settlement and hence the existing foundations had performed adequately.

4.4 *Load-Settlement response of Existing Piles*

The effect of the construction processes arising from the building redevelopment on the existing piles can be compared to a loading test on an under-reamed pile. The demolition stage is comparable to an unloading cycle of a pile in a maintained load test and the construction of the new structure is analogous to the second load cycle of a pile load test. Figure 4 shows an idealised load–settlement response of a pile in London Clay.

If the new loads exceeded the original Belgrave House pile loads then there would be a net permanent settlement, and a reduc-

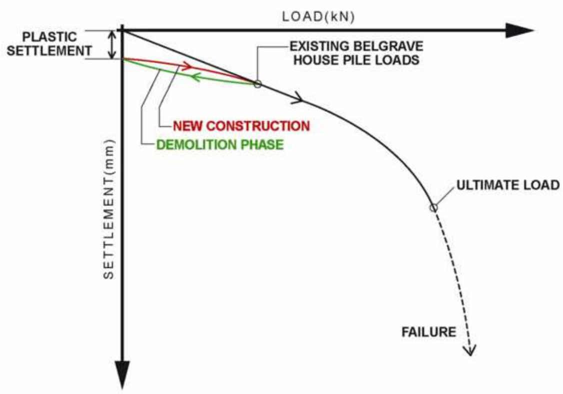

Figure 4: Load settlement response of a pile

tion in pile stiffness. This would become progressively worse as the pile ultimate load is approached whereupon, the stiffness of the pile reduces rapidly and excessive settlement will be observed which would subsequently lead to pile failure. To overcome potential problems with excessive settlement the piles were linked by a new transfer slab which redistributed loads avoiding the risk of failure of the piles and where there was a risk of over-loading the piles new ones would be installed.

Fleming and Salter (1962) reported on the monitoring of a large diameter under-reamed pile load test in London Clay. The pile was founded on London Clay of similar strength to that at Belgrave House; the load test was carried out on a 1.2m shaft diameter, 10.7m deep and 3.2m under-ream diameter pile. The load test results showed the safe working load of 4800 kN resulted in 10mm settlement (5mm of which was irrecoverable plastic settlement), and at 1.5 times safe working load (7200 kN) the pile settled by 33mm, half of which was again noted to be plastic settlement.

The virgin settlement of the Belgrave House under-reamed piles at the design safe working load was calculated and a maximum settlement of 10mm was predicted at the design safe working load of 5580kN (400 series piles). It was expected that the demolition of Belgrave House would result in small pile head uplift. Subsequent construction of the new building would then result in reloading of the piles; the ground would respond elastically and be expected to settle approximately by the same amount as it previously heaved.

The existing pile loads at Belgrave House can be considered to consist of two components: the building dead load plus a realistic live load (which is some percentage of the design live load). The new design pile loads generally did not exceed the existing pile loads and the resulting settlement was expected to be small. If the proposed pile loads exceeded the existing loads, but stayed below the Original Safe Working Load then a further settlement of few millimetres was predicted. The maximum pile settlement upon reloading was expected to be no more than 10mm. None of the piles were to be loaded above the Original Safe Working Load.

A maintained Pile Load Test was considered to be carried out on a pile after demolition of the building. The aim of the test was to confirm the load-settlement behaviour of the piles upon reloading. However, this was not undertaken because of time, logistical problems and cost constraints.

4.5 *Impact of Demolition on Existing Piles*

Concerns were raised about the Piles could become damaged during the demolition of the building. The main cause of damage to the piles was thought to be from the demolition of the basement slab and pile caps. Care had to be taken during the demolition works so as not to subject the piles to tensile forces.

The section of the piles most susceptible to cracking was just below the reinforcement cage, some 8.5m below the top of the pile. Pile integrity testing was recommended to detect anomalies/cracks but the tests cannot confirm crack size, however it is also accepted that the anomaly results from such tests could be doubtful, but due to its cost-effectiveness, rapid and non-intrusive nature these tests were carried out as a matter of course on the exposed piles. If the results showed that the piles are damaged then this could have influenced the construction programme and may resulted in the need for the installation of further new piles. The structural arrangement for the new building was specifically set to avoid eccentric or horizontal loads being applied to existing piles: hence the integrity of the pile reinforcement did not influence the design of foundation or the reinforcement needed to be tested for corrosion.

4.6 *Assessing Pile Deterioration for the Design Life of the Building*

Visual inspection and laboratory tests were commissioned to show that the piles had not deteriorated beyond their original design life. Harmful processes that can affect piles are sulphate attack on the concrete and reinforcement corrosion. Representative core samples from the piles were taken particularly in the Terrace Gravels where there is a higher risk of damage due to exposure to groundwater. Microscopic examinations were carried out to closely investigate for any evidence of concrete deterioration.

The exposure conditions for the new building will not subject the piles to a more harmful environment than has existed to date and hence if the exposed piles show no sign of sulphate attack over the past thirty years then the piles are unlikely to suffer such attack over the design life of the new building.

4.7 *Risks*

Inspection of existing piles for damage or deterioration is only carried out on a relatively small number of piles and provides valuable information; the integrity tests provide further assurance on the existing piling. However these cannot guarantee that all deficiencies are detected.

The re-use of existing piles also raised the question of who was to be responsible for the design. In traditional piled foundation contracts the designer is responsible for specifying the performance required of the piles whilst the specialist contractor is responsible for the design of the piles to meet the defined criteria. However, in this case the contractor responsible for the installation of the pile was understandably reluctant to warrant the piles for the change in use and increased design life.

It is most unlikely that warranties will be provided by any party to guarantee the future performance of the existing piles in the new development. As the lead engineer on the project whitbybird took the appropriate professional processes and decisions to conclude whether the piles are adequate for the new development. Although reuse of existing foundations for a refurbished building structure is an established practice, attitudes to insurance vary and there was a risk that the building insurers will adopt a view that the reuse of the piles is an increased risk over the installation of a new pile foundation which could have had financial implications for our client. We recommended that early discussions to take place between our client and their insurers to quantify and mitigate against this risk, together with a detailed testing programme, as described below in Section 4.8.

There was always a risk that the investigation results on the existing piles would show that some of the piles were not suitable for re-use. To address this risk an acceptancy strategy was developed to manage the results of the investigation, also an alternative foundation design was schemed up in case the existing piles could not have been re-used. The above outlined design risk was further addressed by ensuring that the investigative works on the existing piles were carried out, prior to and during the demolition works and well ahead of the start of the piling contract works.

4.8 *Testing Programme & Acceptance Strategy*

4.8.1 *Testing Programme*

When piles are constructed for new developments in London, it is normal practice to carry out integrity testing on all the piles and load test 1% of the piles for piles designed with a factor of safety of 2.5. The London District Surveyor's (Office) also advises that a sacrificial pile be tested (Preliminary Pile Load Test) to near failure, if a lower factor of safety of 2.25 (Constant Rate of Penetration Test) or 2.0 (Maintained Load Test) is to be used in the design.

On the same basis, a number of the existing piles at Belgrave House were examined to establish their integrity and to confirm the pile design assumptions. Based on the currently available data and considering the number, type and depth of penetration of the existing piles that were to be re-used it was concluded that 5% of the piles i.e. six in total, is an appropriate number to be tested in order to provide confidence in the integrity of the piles at this site.

Figure 5 shows the existing piles and observation test pit locations for exposing and testing the existing piles. The testing programme included the following:

- confirm pile diameter
- confirm concrete integrity and absence of any defects
- confirm pile toe level
- carry out crush cylinder and microscopic examination on cored sample to confirm structural load carrying capacity and durability

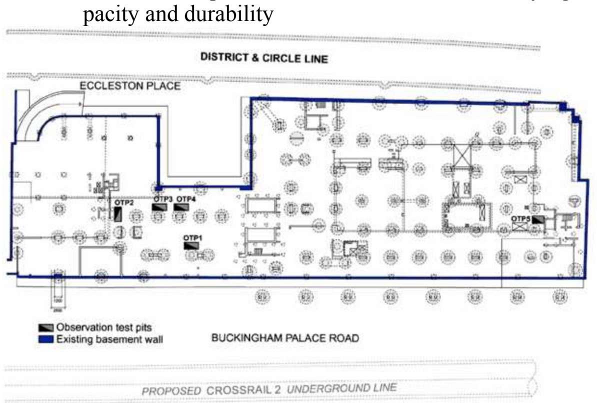

Figure 5: Observation Test Pits – Existing Basement Plan

Depending on the findings from the testing programme it was considered that there may be a need to selectively increase the number of piles tested.

Where pile heads were exposed after demolition, then integrity testing was carried out on all the exposed piles.

It was also recommended that selected column/piles are surveyed using precise level targets and that their load-settlement response to demolition and re-construction is monitored.

4.8.2 *Acceptance Strategy*

An acceptance strategy was introduced whereby if test results confirm the design assumptions and show the piles have not deteriorated with time, or been damaged, then the piles are suitable for reuse.

However, if the results showed that some of the piles were not as originally envisaged in the design, further action would have been needed to be taken. This could have included one or more of the following:

- Increase sample size to 10% of the existing piles
- Conduct a maintained load test on the affected pile or piles
- Downgrade the load carrying capacity of some of the piles
- Install new piles

5 INVESTIGATION WORKS

5.1 *Inspection of existing piles*

The pile inspection comprised excavating observation pits next to selected columns to expose the range of pile types identified on the drawings; the excavation was then extended to expose the piles to depth. One of the reasons that the pits were excavated was to verify the size and location of the piles with the information shown on the original design drawings. Figure 5 shows the location of the observation pits.

The observation pits were excavated manually due to the limitations of space and access from working in the basement of the existing building (Figure 6). The pits were also dug during the soft strip demolition of the building and this resulted in additional health and safety considerations.


Figure 6: Basement of Existing Building

The number of piles inspected was chosen to be 5% of the total number of piles that were to be re-used (i.e. 6 Number), however as part of this investigation seven piles were inspected and tested although about one in twenty piles were located and tested, the frequency of inspection could be considered relatively high when compared to the typical working pile test on new piles which is normally one in a hundred.

A total of five observation pits were excavated at basement level to a maximum depth of 3.9m below structural slab level. The piles exposed in the trial pits were visually inspected. No obvious defects in the piles were noted. Table 3 below details the piles revealed in the trial pits using the original pile design reference numbers.

A core was taken through each of the piles to confirm the diameter of the piles and for carrying out laboratory tests. Table 3 summarises the length of cores recovered from each of the piles. The cores were recovered in several pieces and the lengths of cores recovered showed the approximate diameter of the pile is greater than the design pile diameter shown on the drawings. The cores were generally taken in the gravels and the greater diameter encounter indicates the size of the piles in the cased section and the over-breaks.

The following in-situ and laboratory tests were carried out on the existing piles:

- Low Strain Integrity Test
- Microscopic examination
- Chloride content test
- Sulphate test on ground and groundwater
- Crush Cylinder test on cored samples

Table 3 – Summary of Investigation Results

Trial Hole	Piles Revealed	Design Pile Shaft Diameter (mm)	Core Length/ Pile Diameter (mm)	Core/Test Level (mOD)
OTP1	P216	900	500*	-0.975
OTP2	P181	600	700	-0.875
	P182	600	690	-0.875
OTP3	P315	1200	1400	-0.175
OTP4	P316	1200	1350	-0.285
OTP5	P103	600	690	-1.975
	P222	900	1000	-1.125

*full length pile core not obtained

5.2 *Integrity Testing*

Integrity tests were carried out in the trial pit on all the piles inspected. This required the preparation of the side of the pile by forming a small notch (Figure 7).

Figure 7: Integrity Testing carried out on Exposed Piles

This then enabled sonic type integrity tests to be carried out. These tests were performed to assess the condition of the pile and to determine if there were any significant defects in the piles below ground level. The tests were also carried out to provide an estimate of the length of the pile and to compare this with the original design length.

Integrity testing was performed on all seven piles revealed in the observation pits. The tests concluded that there was no evidence of major detrimental impedance changes for the piles and therefore, within the known capabilities of the test, the piles appear to be of a satisfactory integrity with no obvious indication of pile defects.

Table 4 details the length of piles estimated from the pile integrity testing carried out. The table also lists the calculated pile toe levels. The original pile design drawings indicated a pile toe level of -19mOD for all of the piles.

Table 4 – Summary of Integrity Test Results

Observation Pit (No)	Piles Revealed	Pile Length Below Test Level (m)	Estimate Pile Toe Level (mOD)
TP106	P216	18.20	-19.18
TH1	P181	20.02	-20.90
	P182	20.00	-20.88
TH2a	P315	18.20	-18.38
TH2b	P316	19.00	-19.29
TH4	P103	19.80	-21.78
	P222	18.60	-19.73

The integrity testing revealed the toe level ranging from -18.38 to -21.78mOD. Only one of the seven piles recorded a pile toe level less than the design pile toe level. The test calculates the depth of pile to an accuracy of approximately 10% and therefore, the results of the integrity testing and observations made in the pits were considered to verify the design information.

Integrity testing was also carried out after demolition of the building. The main purpose of these tests was to determine whether the demolition process had resulted in damage to the piles.

The tests were carried out on exposed piles for construction of the transfer slab. If the piles were not exposed, for example where the main works did not require the removal of existing pile caps or slabs, then they remained untested. Approximately 75% of the existing piles were tested. Of these all the under-reamed piles were found to be of good structural integrity. Few of the 600mm diameter straight-shafted piles failed the tests, the results showed cracks at 2-3m below ground level. It was unclear whether the demolition works caused the cracks or whether they were present before the works commenced. Where necessary the pile was excavated and inspected or the new structure was re-designed to span over the cracked piles.

5.3 *Laboratory Inspection and Testing*

Sub samples of cores taken from the piles were visually examined and tested. Petrographic examination was carried out to assess the quality of the concrete and examined for signs of abnormalities or deterioration such as sulfate attack. The laboratory test results were interpreted in order to confirm the durability of the piles over the design life of the new building. Laboratory tests were also carried out on the concrete core specimens to determine their strength. The aim of these tests was to compare the actual strength of the concrete sections with the original design strength of the concrete.

5.3.1 *Microscopic Examination*

Sandberg Consulting Engineers were commissioned to carry out the microscopic examination and laboratory testing on the concrete core samples. The visual inspection of the cores showed no evidence of cracking or honeycombing and the coarse aggregate distribution was found to be even or fairly even.

Microscopic examinations revealed the concrete comprises natural or partially crushed natural gravel, natural sand with a Portland cement binder. Portlandite (a form of calcium hydroxide) was seen to be lining or infilling voids and gaps along aggregate margins and some thaumasite deposits were also observed in the surface zone. The abundance of Portlandite within the Cement matrix and voids is indicative of wet conditions. The thaumasite present is indicative of some incipient very small scale sulphate attack. The thaumasite form of sulphate (TSA) attack requires the following to be present simultaneously:

- Presence of sulphates and/or sulphides in the ground
- Presence of mobile groundwater
- Presence of Carbonate, generally in the coarse and/or fine concrete aggregate, but occasionally originating in soil or groundwater
- Low temperature generally below 15 °C

Apart from a small amount of shell noted in the fine aggregate, no sources of carbonate were identified in the concrete. Thaumasite can form in the voids and cracks on the concrete surface however in the observations made only a slight surficial weakening had occurred and given the age of concrete and the amount of thamasite identified it was considered unlikely that this will pose a significant threat to the durability of the piles.

5.3.2 *Chloride Content*

The chloride contents of the concrete core samples ranged from 0.15% to 0.53% by weight of cement. It is generally accepted that there is a threshold concentration of chloride ion which must be exceeded before significant corrosion of the embedded reinforcement occurs, however this threshold value is dependent on several factors such as cement type and source of chloride ions. BS 5328:Part 1:1997 requires that normally manufactured concrete should have a chloride ion content not exceeding 0.40% by weight of cement.

Two of the results showed values (0.41% and 0.53%) just higher than the 0.40% limit for an assumed mean cement content of 14%. It was concluded that the most likely source is the use of chloride bearing aggregate in the mix and considering the fact that the actual cement content is very likely to be higher than the assumed value of 14% (and this is quite possible given high compressive strength results of 67.5N/mm^2 and 80N/mm^2) then the chloride content expressed as a percentage of cement content is likely to be lower.

The re-use of existing piles in the new scheme were not designed to take eccentric or horizontal loading and therefore integrity of the pile reinforcement was purposely avoided to remove the requirement.

5.3.3 *Concrete Compressive Strength Test*

The strength of the concrete quoted on the original design drawings was relatively low 25N/mm^2. The safe working capacity of some of the under-ream piles in terms of soil failure was almost the same as the safe capacity of the concrete against compressive failure. For modern piles the design concrete strength is normally in the region of 35-40N/mm^2. The concrete strength of all seven cores is listed in Table 5 as estimated in-situ cube strengths. The core sample from pile P216 returned estimated in-situ cube strength of 17.5 N/mm^2. The other six tests produced concrete strengths significantly greater than the original minimum concrete strength requirement of 25N/mm^2. BS 6089: 1981 acknowledges that estimated in-situ cube strengths can be less than standard cube strengths. The material factor of 1.5 applied to the standard cube strength of 25N/mm^2 gives a design strength of 16.6N/mm^2 for the in-situ concrete. Therefore, the lower estimated in-situ cube strength is acceptable.

Table 5– Summary of Concrete crushing test results

Trial Hole	**Piles Revealed**	**Design Pile Shaft Diameter (mm)**	**Estimated in-situ cube Strength (N/mm^2)**
TP106	P216	900	17.5
TH1	P181	600	67.5
	P182	600	80.0
TH2a	P315	1200	81.5
TH2b	P316	1200	81.5
TH4	P103	600	58.0
	P222	900	78.5

6 TRANSFER SLAB STRUCTURE

The load transfer from the new column grids to the existing piles was achieved by a new transfer slab and ground beams. Figure 8 shows the foundation layout for the Belgrave House. The centre bay of columns is supported on a 1 metre deep transfer slab which is supported on the existing piles. Few 750mm new diameter friction piles were constructed to the south sector of the site to compliment the existing piles where their load capacities have been exceeded. Similarly adjacent to Eccleston place, where a new area of basement was created, new piles were installed to support the row of column on the rear façade of the building. The new columns on the Buckingham Palace Road elevation are supported on a new line of 1200mm diameter, 40 metre long friction piles.
A finite element software package was used to design the 1 metre deep basement transfer slab.

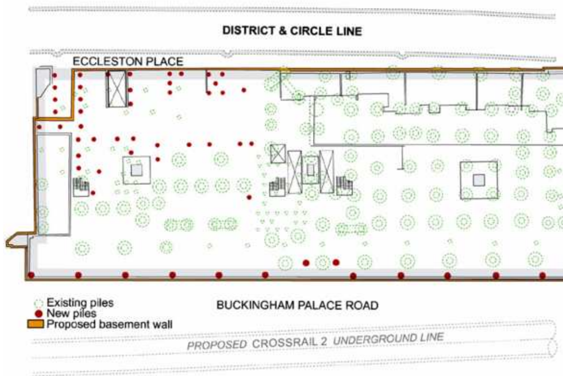

Figure 8: Proposed Foundation Layout

In the analyses the existing piles were modelled into two distinct groups, one straight shafted with a load capacity of 1170kN, the other under-reamed piles with load bearing capacities ranging from 3850-5600 kN respectively. Due to the different type of piles different stiffness properties were assigned and hence predicted how each group of piles attracted a portion of the column loads.

From the analyses pile loads were predicted and subsequently the slab reinforcement was designed. It follows from first principles that a stiffer support will attract a higher amount of load than one showing a softer response. Equally, a stiffer support will induce a larger hogging moment at that support and a lower sagging moment mid-span. Following a parametric study and systematic approach to the analysis, the worst case loads on each pile and the worst case bending moments and punching shear forces in the slab were determined.

In cases where existing piles became overloaded and settled by more than their acceptable value, new piles were introduced into the scheme. From the finite element model the location of the new piles were determined and the model was re-run to confirm that the piles are no longer over-loaded and the settlements were not exceeded.

7 CONCLUSION

The presence of existing piles at Belgrave House imposed restrictions on the design of the new foundations and provided an opportunity for their re-use in the new scheme. The principal benefits that reuse of the existing piles provided included; reduction in construction programme, less risk to future development arising from obstructions in the ground and lower foundation cost. A strategy for the reuse of piles was developed at the outset to address the design risks and successfully followed through the design scheme and demolition phase to confirm their suitability for re-use.

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