Crossrail: a Specialist Foundation Construction Lessons Learnt conference

in association with the Federation of Piling Specialists

PROGRAMME OF PROCEEDINGS

Tuesday 10th November 2015

City Hall, London

50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

FOREWORD

On 10th November 2015 the Federation of Piling Specialists and Crossrail jointly hosted a themed conference on the lessons learnt during the construction of the deep foundations and ground stabilisation works of the new Crossrail railway. The objective of the conference was to ensure a record of some of the methods, challenges and innovations encountered by specialist contractors and consultants was captured and could be made publicly available for the benefit of those working on future infrastructure programmes. At the time of the conference the majority of the underground works were complete and the success of the specific solutions understood.

The conference was organised into 4 themes:

- Construction & Geology
- Specification & Design Verification
- Monitoring & Innovation
- Equipment & Innovation

These proceedings include papers presented during the day, many of which span several of the themes, supporting knowledge sharing and stimulating debate.

Crossrail and FPS would like to thank the authors for recording their companies experiences in working on the Crossrail project and making them available as legacy papers. Thanks also goes to the chairs of the various session themes, Prof. Stephan Jefferis, Asim Gaba, Prof. Robert Mair and Tony Suckling for supporting the event and stimulating debate.

The conference was closed by the President of the European Federation of Foundation Contractors who heralded the event as a great example of a client organisation and contractors working together through a trade organisation to show leadership for the benefit of the industry.

It is hoped that you enjoy reading the papers, made available without liability, and find them as a useful reference source for years to come.

Mike Black
Head of Geotechnics, Crossrail

Martin Blower
Chair, Federation of Piling Specialists

50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

CONTENTS

CONSTRUCTION & GEOLOGY

Abstract

Bond Street Eastern Ticket Hall, located at Hanover Square to the South of Oxford Street, is one of the flagship construction sites on the Crossrail project. The Eastern Ticket Hall will provide a new entrance to the station platforms under Mayfair, at the corner of Hanover Square and Tenterden Street.

This paper describes the challenges associated with the design of the temporary piled retaining walls, detailing of the permanent works pile reinforcement cages, permanent works design of the plunge columns splices and construction of all the piled elements of the main structures. The Paper discusses the requirement for close working relationships and clear communication by all parties involved and how this was achieved to help successfully deliver the works.

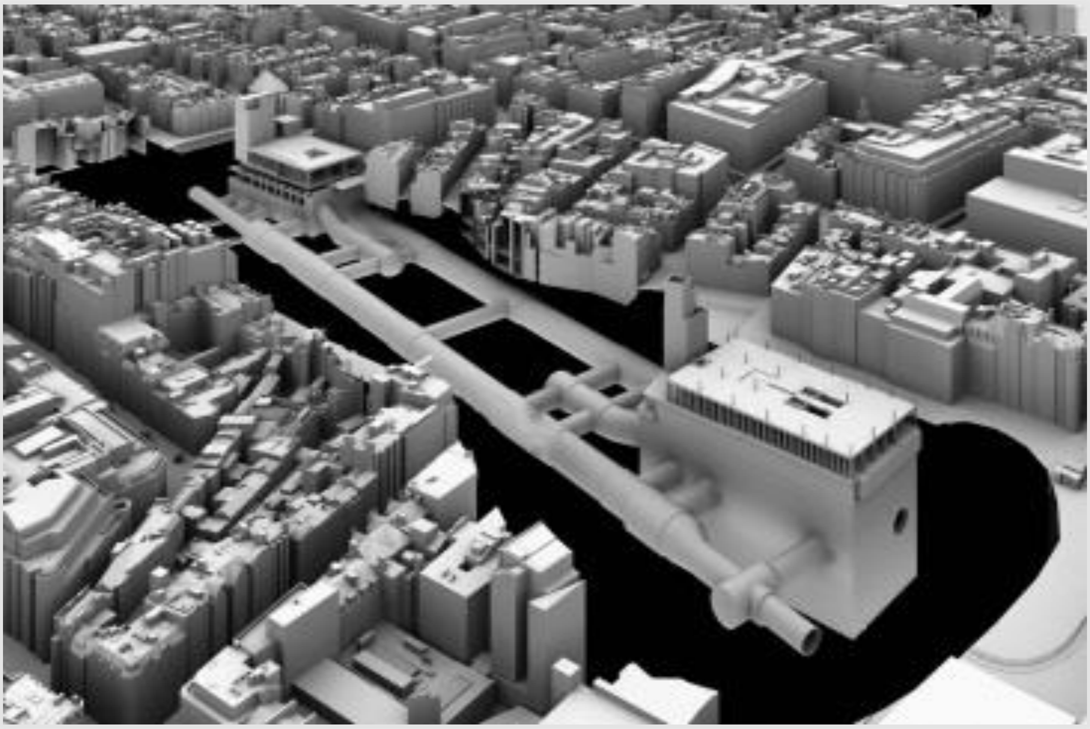


Figure 1 - Bond Street – Crossrail: Station Layout

Figure 2: Bond Street Eastern Ticket Hall – North West Shaft – by day.

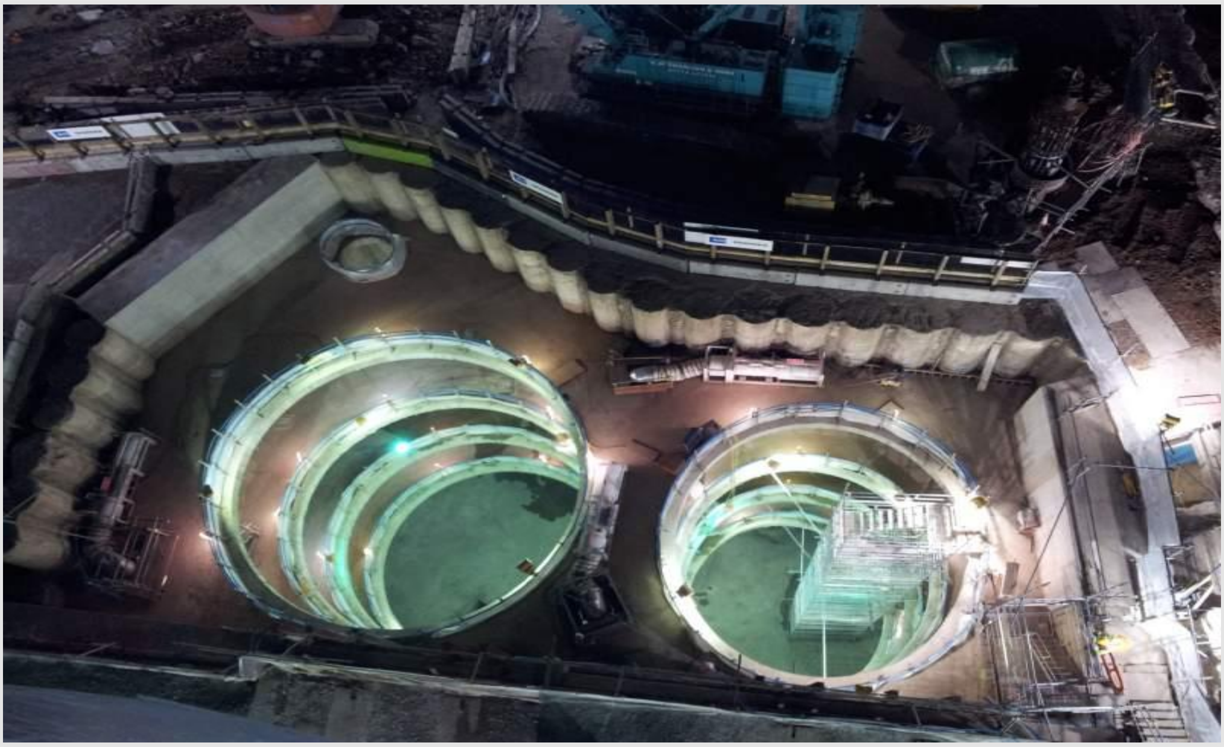


Figure 3: Bond Street Eastern Ticket Hall – North West Shaft – by night.

1. The Background

Costain-Skanska Joint Venture (JV) was awarded the £35m main contract for the construction of the Eastern and Western Ticket Halls. The scope included the construction of the Western and Eastern Ticket Halls including the installation of piling, diaphragm walling and plunge columns.

Specialist piling contractor Cementation Skanska was awarded piling packages for both Eastern and Western ticket halls, with the main construction commencing November 2011.

.

The Eastern Ticket Hall
Bond Street Eastern Ticket Hall is located on the site of 18 & 19 Hanover Square. The site is bounded to the east by Hanover Square, to the north by Tenterden Street, to the north-west by 1 Tenderton Street, to the south by 20 Hanover Square and to the west by Dering Yard. A number of buildings fronting the Square, including numbers 20 & 21 adjacent to the Eastern Ticket Hall site are listed. Numbers 20 & 21 are Grade II listed. Number 20 Hanover Square is a substantial terraced town house dating from 1710. Just your typical City of London site.

Figure 4: Up close and personal with Nr 20 Hanover Square

2. Geotechnical Design

The principal structural elements of the Eastern Ticket Hall were:

- A deep box extending from ground level down to the platform level – about 27m below ground floor, with perimeter walls propped by the internal slabs acting as props, and founded on piles and a raft slab at platform level.
- An escalator link from ground level to platform level which starts inside the deep box and then protrudes outside to the west to meet cross passages at platform level.
- A vent shaft rising to approximately 30m above ground and starting at level B-4.
- Running tunnels and platforms with associated cross linking passenger passages and service tunnels.

Ground Profile
The ground profile for the Eastern Ticket Hall can be summarised in the table below:

Stratum		Revised Design Values	
		Top of Stratum (mATD)	Thickness (m)
Made Ground		124.0	4.0
Terrace Gravel Deposits		120.0	3.0
London Clay (Division A3)		117.0	17.0
London Clay (Division A2)		100.0	11.0
Lambeth Group	Upper Mottled Beds	89.0	6.5
	Laminated Beds	82.5	1.0
	Lower Mottled Beds	81.5	7.5
	Upnor Formation	74.0	3.8
Thanet Sands		70.2	1.2
Chalk		69.0	EOH

EOH = end of hole

Given the general site and ground conditions present on the project, together with the construction requirements, large diameter rotary bored segmentally cased secant piles were considered the most appropriate in terms of minimising the environmental impact.

Temporary secant piled retaining wall design
The analysis was carried out using the computer programme WALLAP. The calculations were carried out using the strength factor method with factors on actions and resistance in accordance with BS EN 1997 – 1:2004 (Eurocode 7).
The design is based on 1300/1200mm diameter male piles at 1500mm spacing with 750mm diameter female piles.

The WALLAP analyses carried out by WSP for design of permanent secant wall piles were used to form the basis of the temporary secant wall design. The analyses were modified to reflect the surcharge loading conditions to which the temporary walls would be subjected, since the temporary wall piles are within the station box (i.e. entirely within the permanent secant piled perimeter).

The piling works for the eastern ticket hall required the installation of:

- 220 linear metres of permanent perimeter secant piled retaining wall
- 45m of temporary secant piled retaining wall
- 25m of temporary contiguous piled retaining wall
- 69 large diameter bearing piles (26 of which were constructed with plunge columns to facilitate top down construction)

Male secant piles and contiguous piles were constructed using 1300mm diameter segmental casing. Where the male piles were founding below the base of the London Clay formation, they were constructed under bentonite support fluid, otherwise they were constructed dry, ie without the use of the support fluid.

Female secant piles were constructed as 750mm diameter Continuous Flight Auger (CFA) piles to an average depth of 20m. Concrete mix was a P280 soft mix. Failure to delay the strength increase of the concrete would have increased deviations problems with the augers during the construction of the male piles. Typical strength of the P280 at 28 days was 16N/mm2.

Figure 5: Construction requirements of the male secant piles pushed the boundary

Specified Tolerances
During the construction of the works strict specification tolerances had to be adhered to as detailed in the contract specification. The secant wall tolerance required a vertical tolerance of 1 in 200 over the cased length and 1 in 75 below the cased length, together with a positional tolerance of +/- 25mm.

The maximum permitted deviation of the finished plunge column from the vertical was 1:400 with a position tolerance at commencing surface of +/- 10mm. Plunge column twist tolerance angles was +/- 2.5 degrees. These strict tolerances on the plunge columns were achieved using Cementation Skanska instrumentation rig, CEMlock. Ref to Figure 6 below.

The tolerances were achieved by a number of standard practises adapted by Cementation Skanska, a Guide wall was installed prior to the secant piles being constructed. This helped ensure that the pile tolerances could be achieved throughout the pile installation. In additional, computerised rig instrumentation allowed pile verticality to be monitored during the digging process. Furthermore, all personnel involved in the piling works were clearly briefed on the latest contract Inspection and Test Plan (ITP) so that all roles and responsibilities could be clearly defined.

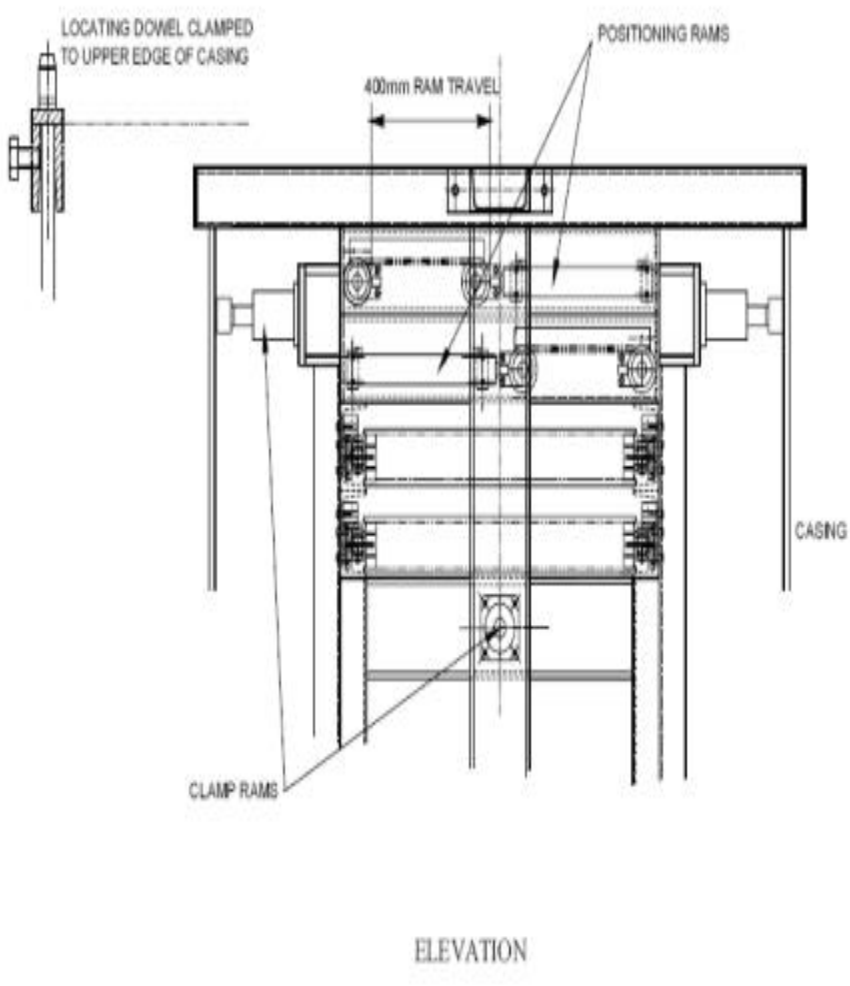


Figure 6: Elevation profile of CEMlock beam used for Plunge Column installation

3. Cycle Time Analysis

Due to specification requirements, the maximum period for the construction of the male secant piles was not to exceed 48 hours. With the site team working a double shift period of 20 hours per day in a 5 day week, detailed cycle times were compiled for the three varying male piles lengths:

- up to 47m in length,
- >47m - <52m in length
- piles >52m in length

The detailed cycle times allowed the site team to not only programme more accurately the material deliveries and required sequence of the piles it also allow the client to have a clear understanding of the works being carried out each day so progress could be monitored.

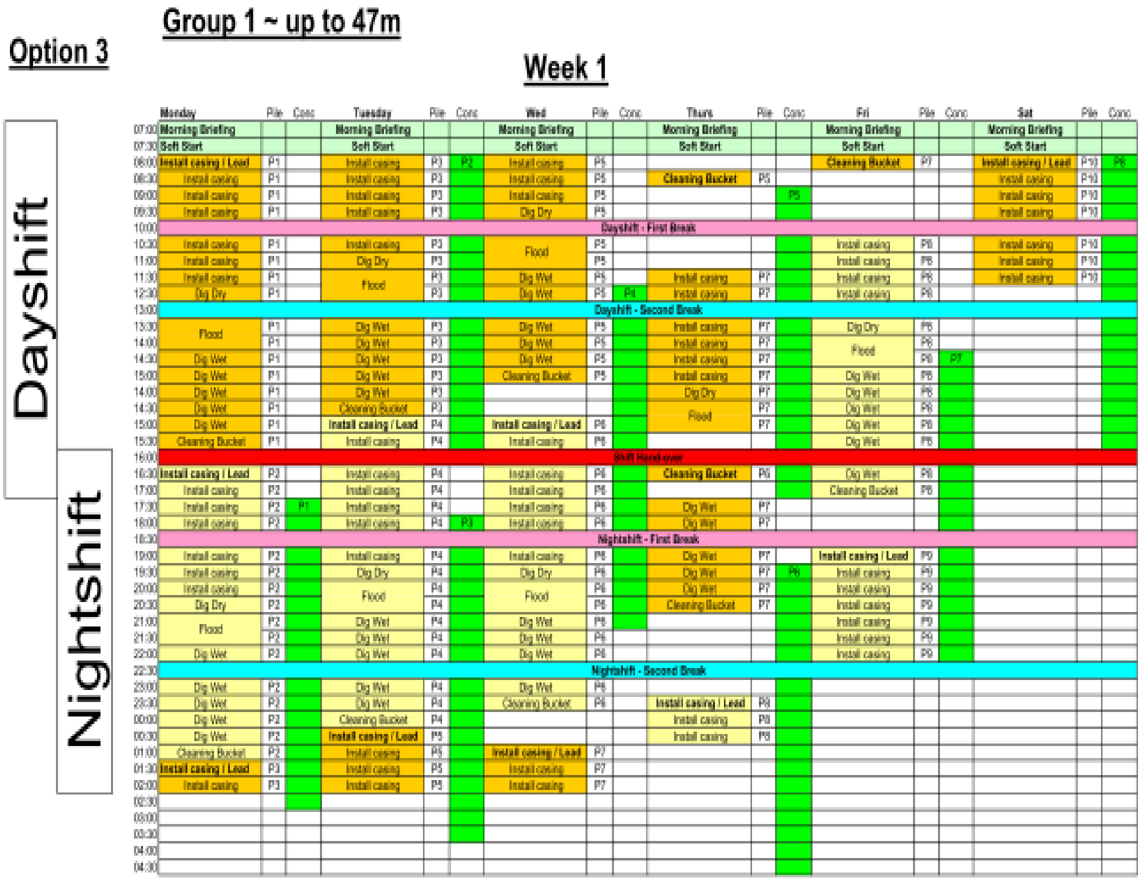


Figure 7: Example of Cycle Time Analysis

4. Reinforcement Cages

One of the greatest challenges of the project (apart from the logistics and programming) was the fabrication, handling and installation of the unusually, complex and heavy reinforcement cages required for the permanent Secant Piles. Overall reinforcement cages were up to 44m in length and weighed up to 23t.

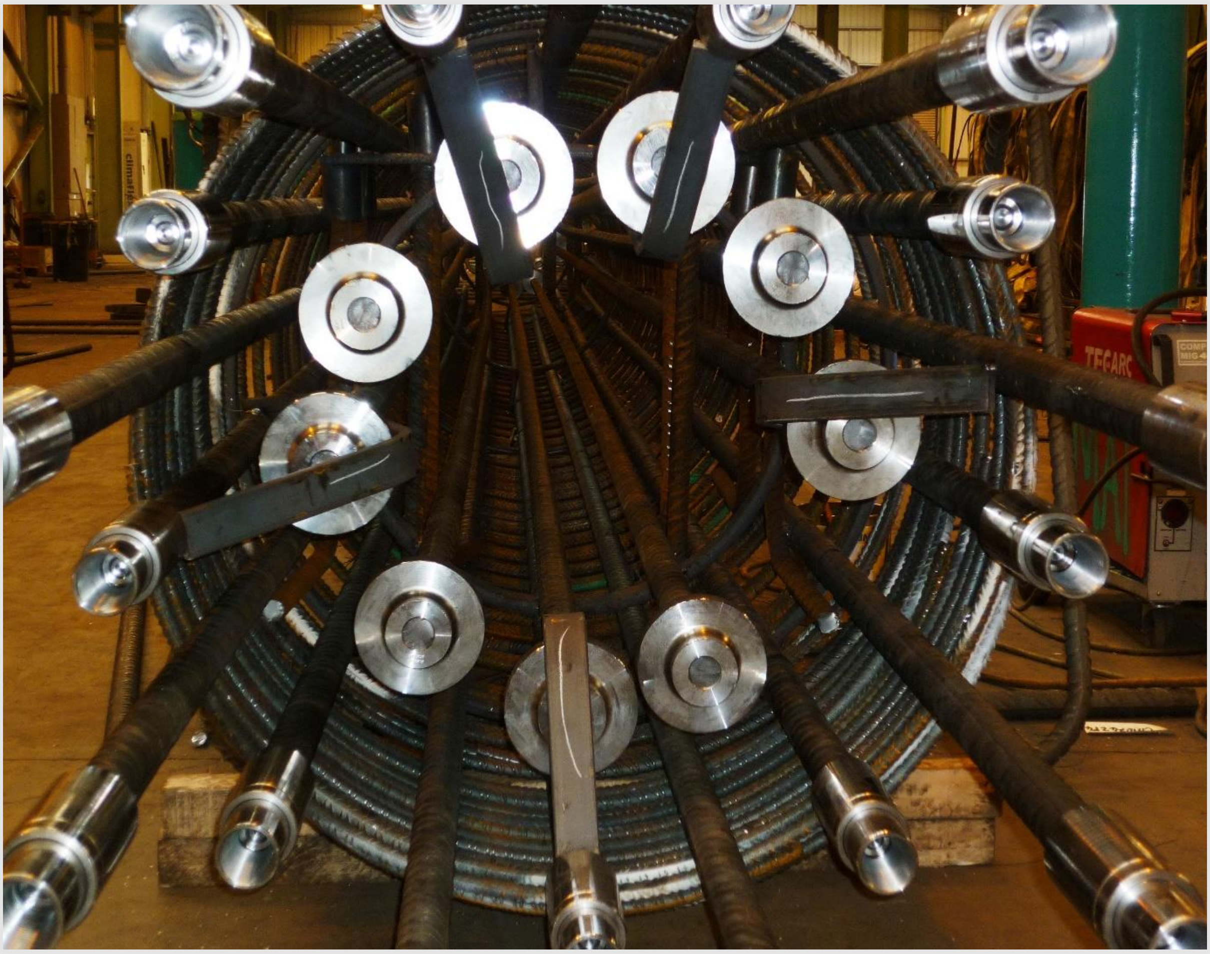


Figure 8: Typical cross section of a typical reinforcement cage for the male piles

The required couplers, terminator plates and internal cage made this one of the most challenging cages Cementation Skanska has produced over recent years. A high degree of skill was not only required to fabricate the cage, but also to assemble the cage on site. In addition to complying with both internal and external quality procedures, the cage was subjected to four levels of checking to ensure that it was constructed to the required specification and quality.

5. Plunge Columns

26Nr of the 69Nr large diameter piles required plunge columns to be installed after the construction of the piles. Plunge column lengths ranged from 9.5m to 34.5m with the universal columns (UC's) being typically 356 * 406 * 634's. The total weight of the column assemblages varied from 10t up to 25t.

Cementation Skanska were responsible for the design of the permanent splice between column sections. A typical splice connection is shown below in Figure 9

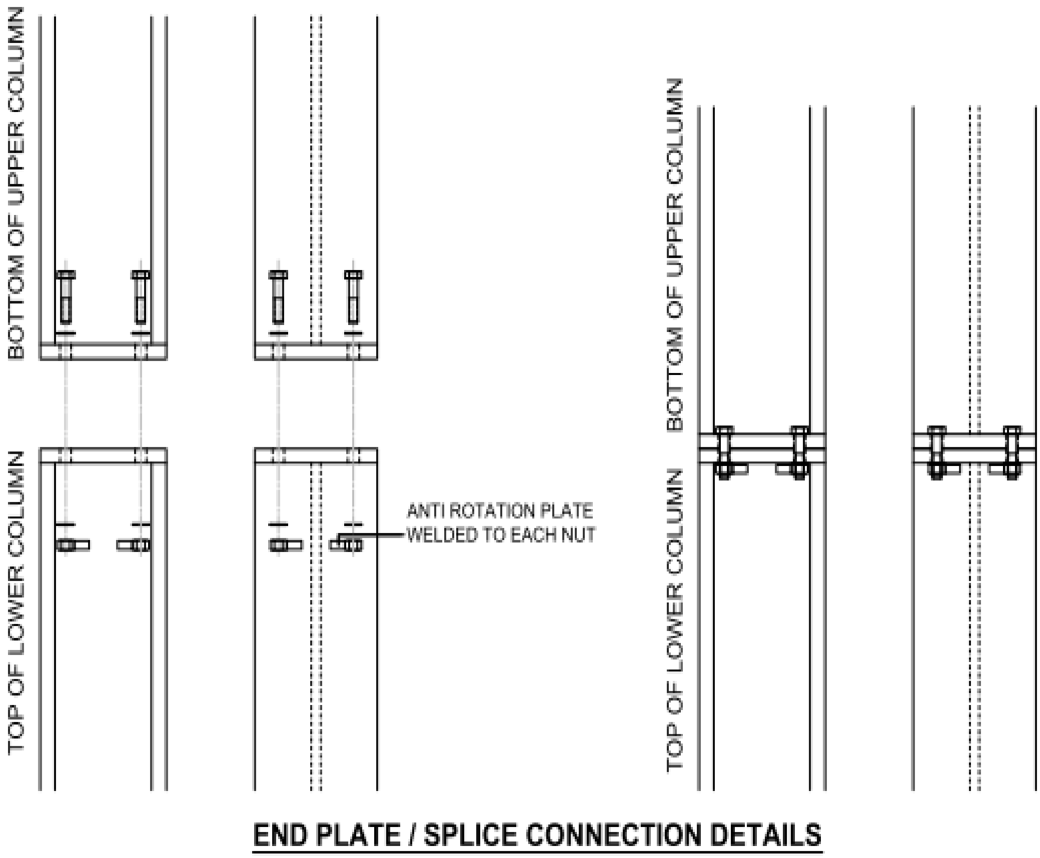


Figure 9: Typical plunge column permanent splice detail

6. Close Working Relationship

Clear lines of communication are essential on any large construction project and Bond Street was no exception. It was imperative that CSL had clear lines of discussion with both the main contractor and supply chains partners.

One key element of clear communication with CSJV came in the form of the 07:30 morning meeting. Each morning the required parties from both companies would sit down and discuss events from the previous day and forward plan the details of the coming days. This level of discussion meant that the required planning and safeguards could be put in place to ensure that the programme targets could be achieved.

Communincation between Cementation Skanska and their suppliers was another critical element of the works. A full time engineer was based at the fabrication factory of the reinforcment suppliers Romtech. This helped ensure that the latest information was being feedback to those working on the fabrication floor. It also meant that any issues could be flagged up early and dealt with before the cages were delivered to site.

The foregoing are not examples of new and immovative ways of working, merely an enhancement of previous methods learned from other contracts that ensured lines of commincation were as effective as possible.

7. Conclusion

The technical challenge of the secant male cage design, the limits of the piling equipment pushed during the installation of the segmental casing and the complexity of the piling sequence with the just in time cage deliveries, resulted in Bond Street ETH being one of the most challenging geotechnical projects that Cementation Skanska had undertaken in recent years.

Ultimately the end product of a secant wall which was delivered safely, on time and budget, and with no remedial leak sealing required during the excavation period of the shafts speaks volumes for the effort that was put in by all the personnel involved.

Diaphragm Wall Construction at Woolwich Arsenal Station Box

Mark Pennington – Balfour Beatty Ground Engineering

Abstract

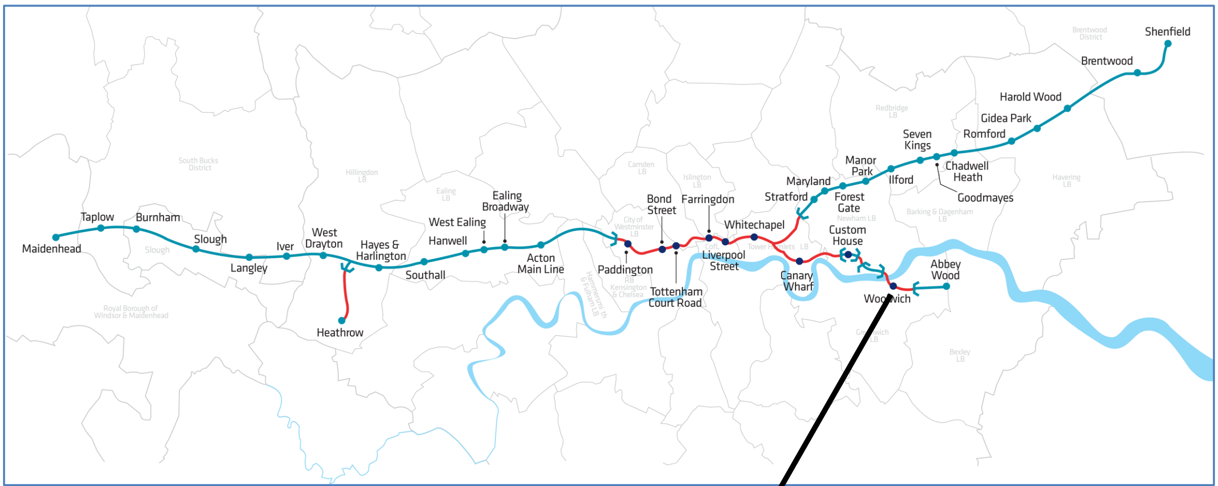
Balfour Beatty Ground Engineering were appointed by Berkeley Homes to construct the diaphragm walls and bearing piles for the Woolwich Arsenal Station Box. The works comprised 500 linear metres of diaphragm wall, 51 Large diameter bearing piles, 465 continuous flight auger piles and significant temporary works piling.

The strata on the site generally comprised Thanet Sand overlying Chalk with a thin layer of Bullhead Bed in between. Previous experience in the area had highlighted problems with diaphragm wall construction where significant quantities of support fluid had been lost. Mitigation measures of grouting the bull head bed layer were put in place prior to diaphragm wall construction. The level of grouting had to be revised and increased during the project.

This paper will highlight the benefits of previous knowledge of local ground conditions and will also highlight the need for adequate and relevant site investigation information. It will describe the measures taken to grout the Bullhead Bed and the further mitigation required.

1.0 Introduction

The Woolwich Arsenal Station box is being constructed for Crossrail by Berkeley Homes. The station will provide access to a significant new development being constructed by Berkeley Homes at the Western End of the Crossrail route. The location of the site in relation to the crossrail scheme is shown in Figure 1.



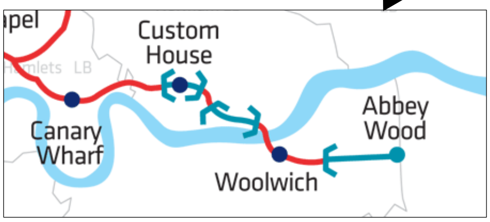


Figure 1 : Site location

This site is one of the few sections of the Crossrail structure that are south of the river Thames. It's location is at the eastern end of the London Basin meaning the ground conditions are different too much of the tunnelled sections of Crossrail where significant depths of London Clay are present.

The piling and diaphragm walling works for the Station Box was constructed by Balfour Beatty Ground Engineering directly for Berkley homes. The basic requirement was to provide a box approximately 260m long 23m to 29m wide and 12m to 13m deep to allow the tunnel boring machine for the South Western drive to pass through. For both tunnel drives, the TBM would enter the box from the Eastern end and then exit towards West. When the TBM had passed through for both of the tunnel drives the station could be constructed within the box.

2.0 Scope of the Piling Work and Method of Construction

2.1 Diaphragm Wall

The piling scope comprised the construction of 500 linear metres of 1000mm wide diaphragm wall. The wall was constructed out of 88 different panels which were up to 7.1m in length. The diaphragm wall panels were not particularly deep at 24.5m. This depth was adequate to allow the 12 to 14 metre excavation. The diaphragm wall provided the support to allow the station box to be excavated, was propped at the top and the basement slab level and did not need to provide a water cut-off in the temporary condition to allow the excavation as a temporary de-watering system was employed. This reduced the water pressure around the box to allow excavation of the box in a dry condition.

The diaphragm wall was constructed using Kelly Bar Grabs as shown in Photographs 1 and 2. These grabs excavate the panels in a standard manner (in bites), bringing the excavated material to the surface to be stock piled and then disposed off site. During excavation the panel needed to be supported by site mixed bentonite support fluid to avoid collapse.

Photograph 1 and 2 : Diaphragm Wall Construction

At the ends of the box the diaphragm wall panels were constructed with fibre glass reinforcement cages to allow the TBM to easily excavate through the panels.

2.2 Bearing and Tension Piles

In addition to the diaphragm wall there were a number of piles both inside and outside the box:

The piles inside the box acted as tension piles within the box to prevent the box from lifting due to water pressure uplift. These comprised 51 piles, 1500mm to 1800mm in diameter with a bored length of 32m to 46m. These piles were constructed from ground level with the concrete cut-off level at 12m to 14m below the platform at the mass excavation level. These piles were constructed using standard large diameter rotary piling and required the use of bentonite support fluid during construction.

The piles outside the box were installed as foundation piles to support the superstructure associated with the station and the Berkeley Homes development. These comprised 418 piles, 750mm in diameter with depths of 16.5 to 33.5m. These piles were constructed using continuous flight auger piling. This meant that the piles could be constructed without the use of support fluid.

2.3 Temporary Works Piling

In addition to the permanent works piling there was a significant amount of temporary works piling required to enable construction. This included a slurry wall which allowed the box to be excavated in two halves, lean mix walls at the end of the box to allow the TBM's to enter the box and stop for some time without water ingress or getting stuck and grouting to allow the diaphragm wall to be constructed – described in sections 4, 5 and 6.

Figure 2 shows diagrammatically the scope of works.

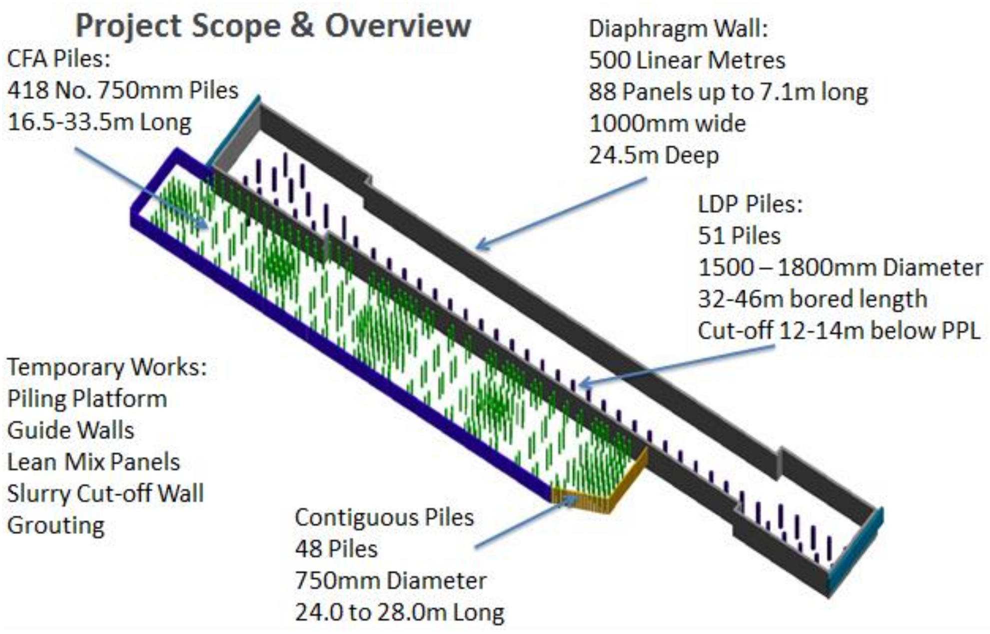


Figure 2 : Scope of Works

3.0 Ground Conditions

The ground conditions at the site are not typical of the standard London Basin strata that exists for much of the underground section of Crossrail.

The site comprises approximately 1.0m of made ground, overlying 2.5m of terrace gravel, overlying 11m of Thanet Sand, overlying a 1.0m band of Bull Head beds on top of Chalk (Figure 3). This strata did not provide any obvious water cut-off hence the requirement for temporary de-watering. All of the strata is permeable and the need for support fluid to support any open excavation during piling or diaphragm wall construction was essential.

Particular attention was given to the details of the Bullhead bed layer which was considered critical to the diaphragm wall construction for reasons discussed in Section 5.0 below. The site investigations had not particularly targeted this layer for investigation and so the information was not as complete as it should have been. In particular there was no information on the permeability of the layer. The Bullhead Bed is a distinctive unit that is commonly present at the base of the Thanet Formation and usually comprises small well-rounded flint gravel ranging in size and shape to cobbles and fine boulder of unworn nodular flint, within a dark greenish grey or black glauconitic sandy clay or clayey sand matrix. The flint nodules (the “bullheads”), which can be up to 0.3m in diameter, are characteristically green-coated. Fossils derived from the chalk occur in places. The unit is generally 0.5 to 1.0m thick.

The ground water levels from Geological information are stated to vary from 5m to 10m below ground level. This confirmed during pile construction on site. As discussed a de-watering system was employed prior to excavation of the box to locally draw the water table to below the excavation level.

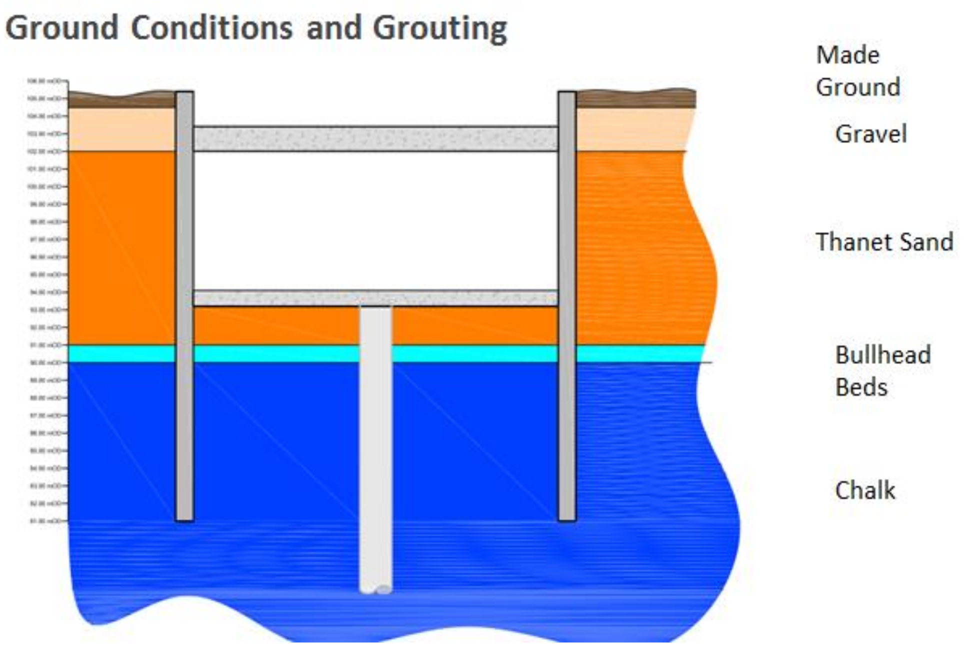


Figure 3 : Typical Ground Conditions

4.0 Existing Knowledge of the Area

The Balfour Beatty Ground Engineering team had recent experience of constructing a diaphragm wall approximately 200m to the South of this site. This was part of the TBM reception chamber for the Docklands Light Railway Extension – City Airport to Woolwich Arsenal. The relative location of the two projects is shown on Figure 4. During construction of this diaphragm wall, a significant amount of the bentonite support fluid was suddenly and unexpectedly lost when the diaphragm wall excavation reached the Bull Head beds. The support fluid apparently flowed into the Bull Head bed strata leaving an open and potentially unstable diaphragm wall trench above this level. It was not possible to recharge the excavation with more support fluid as this ran away.

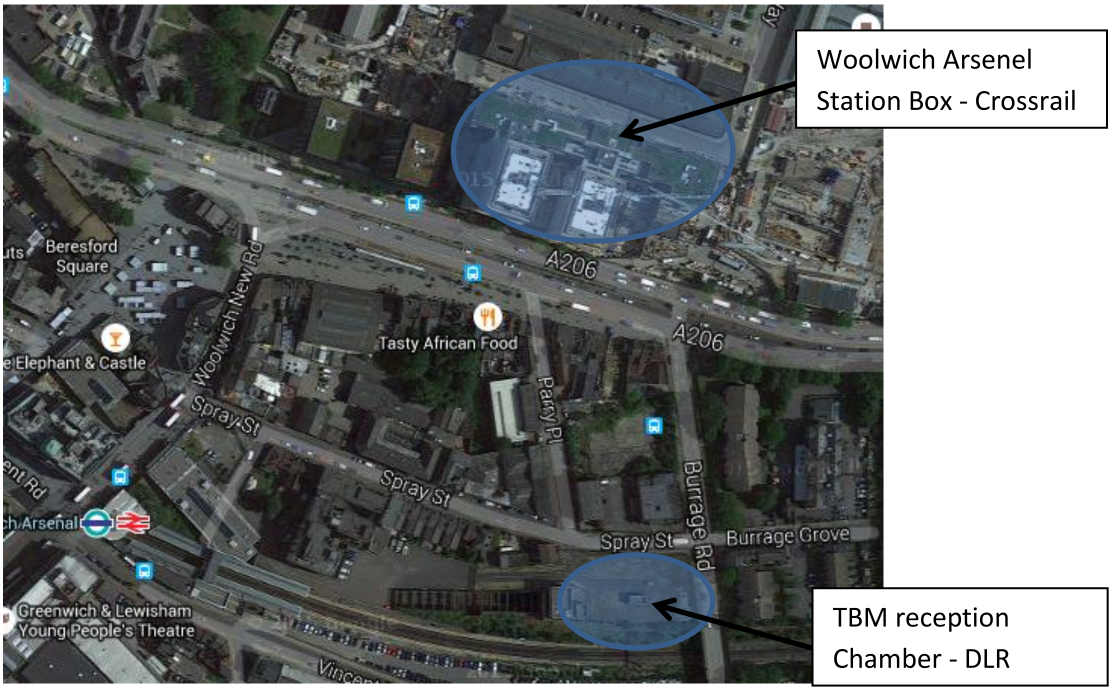


Figure 4 : Relative Position of DLR Diaphargm Wall and Crossrail Diaphragm Wall

This diaphragm wall panel was backfilled and a grouting operation quickly planned to seal the bull head beds either side of the diaphragm wall. Grouting was carried out using a continuous flight auger (CFA) piling ring casting concrete through the level of the Bull head beds and then back screwing out above this. This grouting was carried out using a 900mm diameter auger with piles at approximate 2000mm spacing c/c. This grouting operation proved successful and the diaphragm wall could be constructed.

There was no specific information from the site investigation carried out at the Woolwich Arsenal site to suggest that bentonite may be lost during construction. However, the bull head beds descriptions were very similar to those on the Docklands Light Railway Site and so it was considered prudent to carry out some grouting works of these beds.

5.0 Proposed Grouting Solution

The proposed grouting solution at the start of the project was to drill 114mm cased holes at 2.0m centres each side of the guidewall constructed for the diaphragm wall. The grout holes on each side of the wall would be staggered by 1.0m. This meant the grout holes would be spaced 2.0m in line with the diaphragm wall and 2.6m perpendicular to the wall. Grout would be pumped into the holes over the depth of the Bull Head Beds only and the holes would be backfilled above this with arisings. The pressure and the grout take would be measured and recorded during grouting. This proposal would utilise minimal grout but provide a seal. The general arrangement of the grouting proposal is set out in Figure 5.

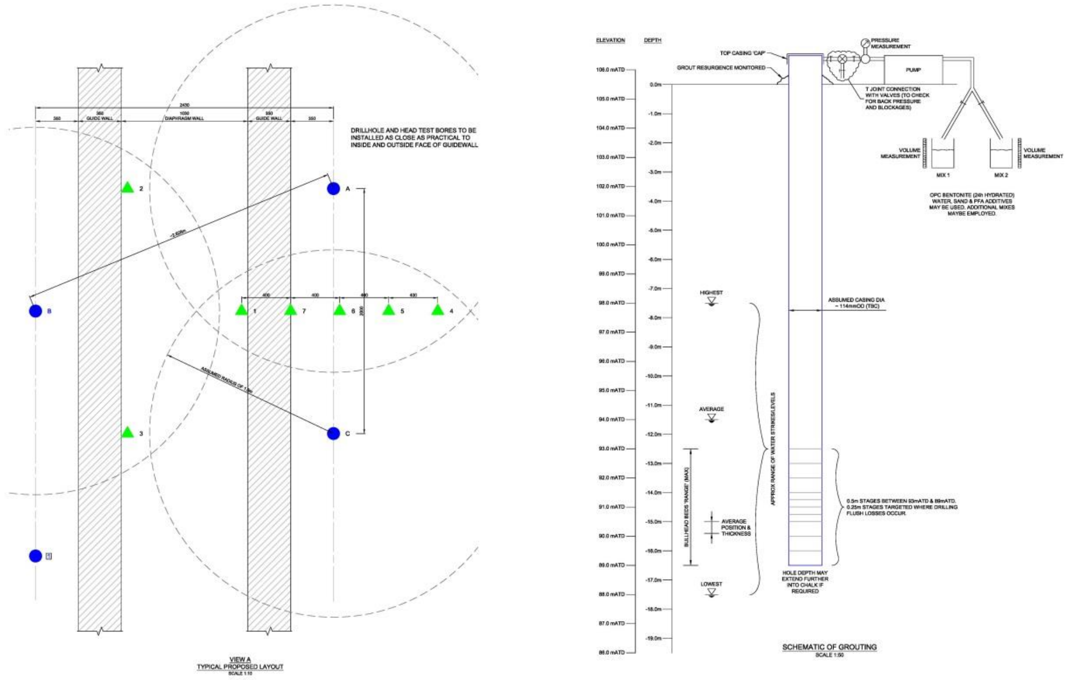


Figure 5 : Plan and Section through proposed Grouting

The assumption was that grout would extend 1.5m into the bull head beds from each grout hole location. The overlap of the grout positions would provide a barrier to prevent flow of the Bentonite support fluid into the Bull Head beds. In order to check the grouting was successful, a number of falling head permeability tests were proposed in between the grout holes.

No grouting was planned for the large diameter piles cast using support fluid within the station box.

6.0 Construction and Final Solution

During detailed planning of the diaphragm wall construction and grouting operation it was assessed that constructing the grouting operation using a 350mm diameter CFA auger would provide significant programme advantage providing cost savings in excess of the additional grout required. The 350mm diameter CFA piles were proposed at the same locations as the mini piled solution. As at DLR, grout was installed only through the layer of the bull head beads with the auger then back screwed out above this.

The CFA auger was extended through the Bull Head beds into the chalk in the usual manner. A pre-mixed grout with a 28 Day strength of 2 to 4 N/mm^2 was then pumped through the hollow stem of the auger. The auger was then lifted in the usual manner to the top level of the Bull Head beds. The auger was then stopped and up to 2.0m^3 of additional grout pumped into the holes.

Diaphragm Wall construction was programmed to commence in the South Western corner of the site. The grouting and falling head permeability testing was successfully carried out in the area and the first panel excavated under the bentonite support fluid. When the panel excavation reached the bull head bed layer, the support fluid suddenly dropped within the panel to the level of the Bull Head beds. It was clear that the grouting operation had not proved successful.

A revised grouting proposal was developed, using the same grout mix but closing up the spacings of the grouting holes. This comprised a full grout curtain around the affected panel (see Figure 6), tertiary grout holes in between the primary proposed holes and grouting in between the guide walls (see Figure 7).

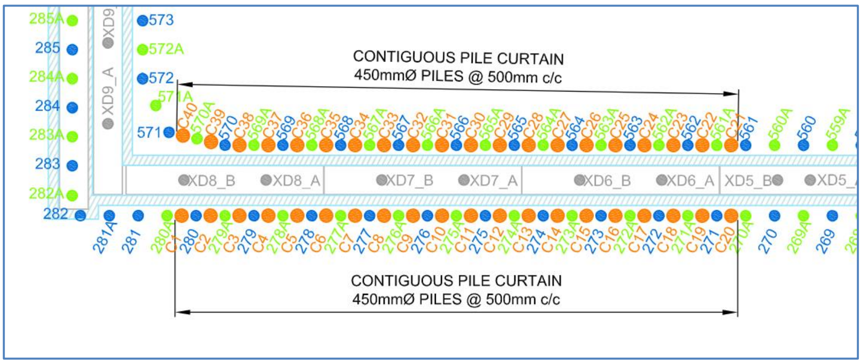


Figure 6 – Grout curtain around affected panel

The revised solution meant that primary and tertiary CFA grouting solution provided grout holes 1.0m centres along the line of the wall either side of the guidewall. The inclusion of grout holes on the line of the diaphragm wall meant that potential bentonite flow parallel to the line of the diaphragm wall also also cut-off.

Additional falling head permeability tests were carried out within each of the panels prior to construction to ensure the grouting operation had worked.

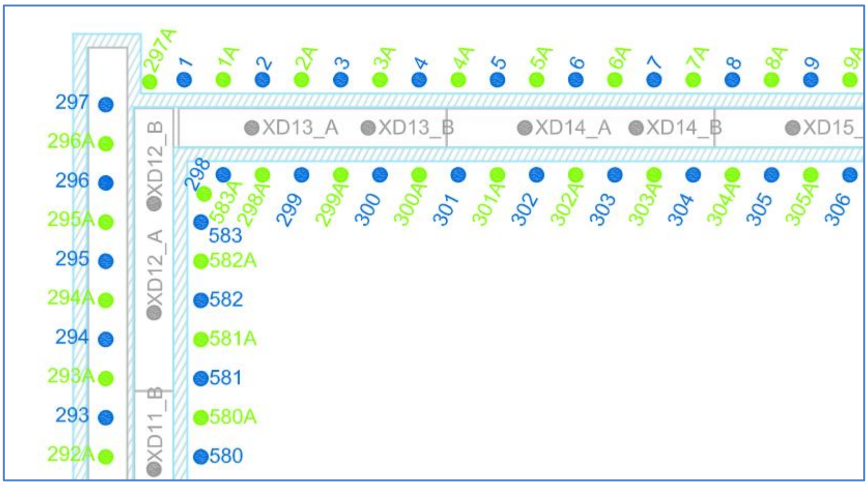


Figure 7 – Revised Grouting proposal

The revised grouting operation proved successful and allowed the diaphragm wall construction to proceed. As the diaphragm wall construction progressed from the western side of the box to the eastern side the grout take from the CFA piling was noted to reduce significantly. Maintaining the grouting in the line of the wall and carrying out falling head permeability tests it was possible to reduce the grouting along the line of the wall back to the primary locations only at a 2.0m spacing. The primary holes were used on their own where the grout take on these holes was less than 2.0m^3.

No grouting was planned or used for the piles constructed under bentonite. These piles were constructed in the usual manner. The issue of bentonite loss during construction was for the diaphragm wall construction only. This must have meant is was either a function of the construction technique or the exposed area of bull head beds – or possibly a combination of the two.

7.0 Conclusion

It was clear from construction at the Woolwich Arsenal site that previous knowledge of the ground and in particular how the use of bentonite support fluid was influenced be the presence of the Bullhead Bed, was key in developing a constructible solution. Even with this previous knowledge problems were encountered during the construction phase. The ground and how this affected the construction varied across the length of the site.

This highlights a number of key points:

- Any previous experience of similar construction in similar ground is key.
- Good initial site investigation providing the correct information for the proposed construction can save significant time and cost during construction.
- In difficult ground conditions time and cost of temporary works to allow construction to proceed may be considerable.
- Particular care and thought needs to be taken when relying on support fluid for pile or wall construction.

Crossrail Lessons Learnt Conference

C350 Pudding Mill Lane

Alistair Briffett, Bachy Soletanche

Abstract:

The contract works consisted of piling the foundations for the new DLR station and viaduct in 2011, followed by the extensive piling of the raft transfer slab to support the load of the DLR embankment in 2012.
As additional work in 2012 a diaphragm wall was constructed to infill the gaps left from the previous contract. Bentonite piles were also constructed for the transfer slabs as a remedy to potential ground stability concerns that ruled out CFA piles.

A secant wall formed the northern wall of the cut and cover trench that formed the tunnel portal as it transitioned up to the Network Rail embankment.
The secant wall was designed as hard /hard piles and were constructed using CSP techniques in 2014. These are the largest piles constructed using this method in the UK and required specialist heavy duty plant and equipment. Specific techniques and materials were developed to ensure the success of this work.

Further CFA piles were also installed as part of the structures that assisted the transition to cross over Marsh Gate lane and Pudding Mill lane

Pudding Mill lane summary of geotechnical works carried out:

282no 1050mm dia CSP secant wall piles to a depth of 20.5m hard hard reinforced
1,162no CFA bearing piles ranging in diameter from 600mm to 1050mm and depth from 20 to 31m deep into Thanet sands
95lm of 800mm wide diaphragm wall to a depth of 18m
41no 1000mm diameter bentonite piles to a depth of 33-35m.

Introduction:

Contract C350 – Pudding Mill Lane Portal involved the construction the north eastern cut & cover tunnel portal that brought the new line up above ground and tied it into the existing Network Rail corridor between Stratford and Liverpool Street (refer to figure 1). The proposed alignment of the tunnel portal clashed with the existing DLR station at Pudding Mill Lane. This necessitated a new station and viaduct to be built south of the pre-existing station and a reinforced earth embankment created to the west which would allow the DLR line to be slewed and the old station demolished. The cut & cover portal could then be progressed along with the associated structures required to bring the line up into the Network Rail corridor. The site had previously seen heavy industrial use in its time including having incarnations as a gasworks, chemical works and notably the Bow Power Station. This industrial use has seen significant levels of hydrocarbon contamination of the soil. This contamination permeated throughout the River Terrace Deposits.

The general site geological profile was of 2m of made ground followed by 4-5m of River Terrace Deposits. This was followed by a thin layer of the London Clay formation before the Lambeth Group geology was encountered. This ranged in thickness from 8-12m. The Thanet Sands were struck at varying depths across the site but generally were encountered at a depth of 28-30m below commencement level.

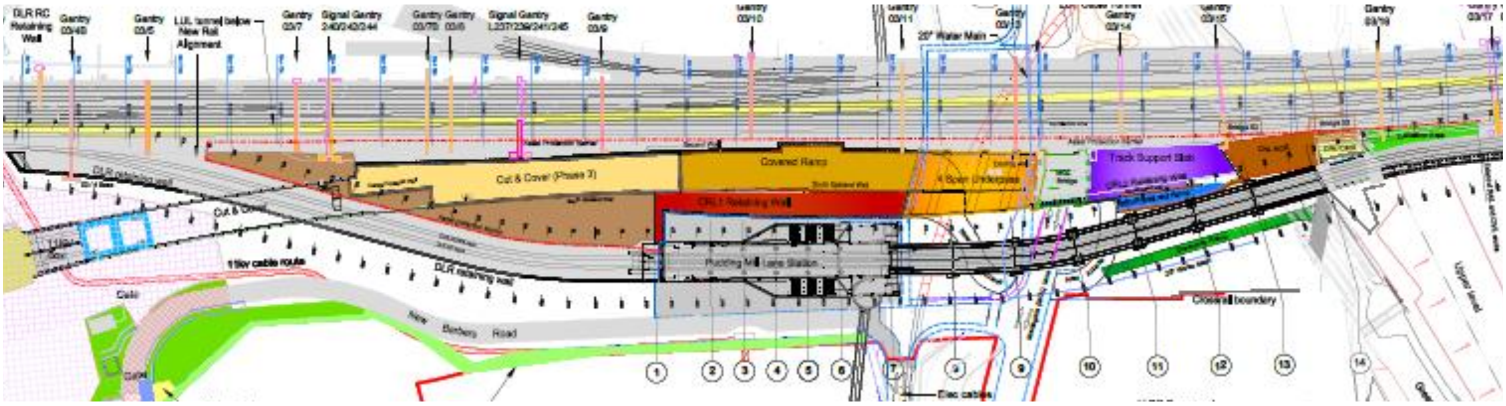


Figure 1 Site overview

The utilisation of CFA piling:

The presence of the sand channels within the Lambeth Group and also the fact that the piles would be founded in the Thanet Sands meant that stability would be an issue were they to be constructed as open rotary bore. The design depth and diameter of many of the project's piles meant they could be constructed via CFA technique. Given the sites proximity to rail assets, CFA was ideally suited as it required far less lifting operations to construct a pile. It also judged to be the most programme effective and cost efficient way to construct the piles and as such was favoured where possible.

CFA piling was utilised on the following structures:

The new DLR station, CRL retaining walls and viaduct required the installation of 457no CFA piles ranging diameter from 600mm to 900mm and in depth from 21.5m to 29.5m.

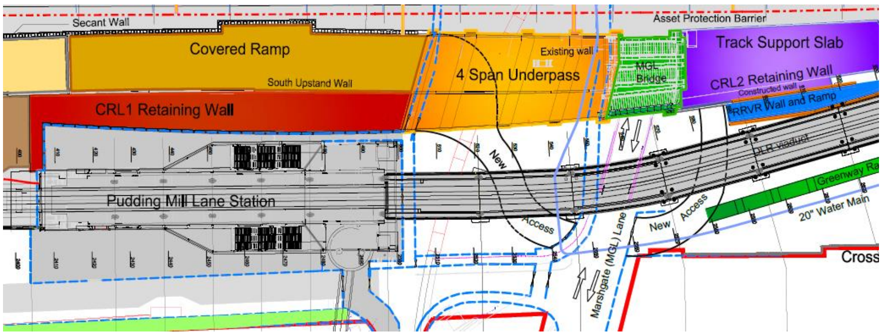


Figure 2 DLR station and associated structures

The transfer slabs, upon which the earth embankment would be constructed, were originally detailed to be stone columns but this was revised due to settlement criteria to consist of a raft of 425no 750mm diameter piles. These were constructed to a depth of 25.5m.

The associated structures required to bridge the Crossrail line across Pudding Mill Lane, Marshgate Lane, and City Mill River, required 280 piles. They ranged in diameter from 750mm to 1050mm and depth from 22.5 to 30.5m

The presence of the Thanet Sands posed a significant risk to the operation of CFA piling as it had a tendency to grip as it settled in and around the auger string as it was progressed down. Multiple contractors have been forced to abandon auger strings once the sands had taken grip of them, with the rigs unable to generate enough torque to release them. To mitigate this risk, Bachy Soletanche employed the large Soilmec SF120 and CM120 CFA rigs with their increased torque. In addition to this care was taken not to commence a pile prior to sufficient concrete being onsite to complete the pile being drilled. This ensured that the augers were left stationary for the minimum amount of time, thus reducing the time the sands had to settle in and around the stationary augers.

The design of the reinforcement often required increasing bar diameters to allow them sufficient weight and rigidity to be plunged in one section. With cages requiring to be plunged up to 17m, any design utilising less than 25mm bar would have had to be increased to allow it to be constructed via CFA technique.

On occasions, pockets of soft ground were encountered. These pockets often resulted in the concrete slumping under the hydraulic head of concrete. It was not uncommon for the concrete level within the piles to drop by up to 500mm as the concrete pushed out into the soft river terrace deposits. Fortunately the cut-off level was significantly below commencement level so the integrity of the concrete below this level was never compromised. It did however have the effect of slumping the cage beyond the permitted tolerance. The site team over came this by hanging the cages off the lifting band which was relocated to the top of the cage rather than at cut off level.

Cut & Cover Tunnel – Diaphragm wall:

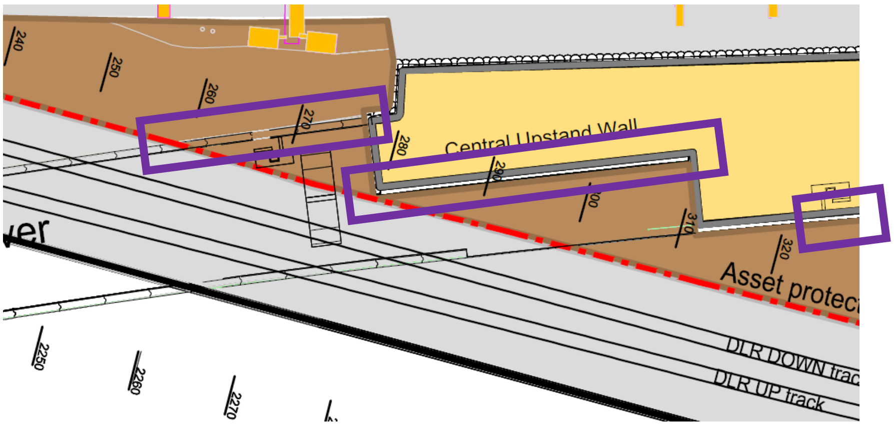


Figure 3 Bachy elements of diaphragm wall

The diaphragm wall that formed the southern wall and the initial section of the north wall of the cut and cover portal had been installed under contract C248. Due to a programme delay in diverting an 11kV cable that obstructed the last elements of the wall, Crossrail took the decision to assign the last remaining sections to the C350 contract rather than have two principal contractors onsite. As a result Bachy Soletanche installed the remaining 95lm of 800mm thick wall to a depth of 18m.

The footprint of the wall was in very close proximity to the DLR and Network Rail assets. This created significant logistical problems with the orientation of the grab and support cranes. While plotting of plant and cranes per panel on CAD was implemented and used to demonstrate to DLR and Network Rail that the panels could be constructed without the requirement for possessions, getting the cranes into these plotted positions without breaching the restrictions was testing. The site team in conjunction with the site representatives from DLR and NR utilised laser distance measures to physically check each movement onsite prior to it being undertaken.

The design of the wall utilised short panel sizes to minimise the risk of causing differential settlement to the adjacent rail lines. However, a lack of understanding of how panels are excavated meant that many of the panel lengths opted for were not suitable for the readily available grab jaw dimensions. A traditional D-wall panel's length is such that it either requires a single bite, or two bites at either end followed by a clearing bite down the middle. By selecting a panel length greater than the width of the grab jaws but less than the width of two, it forces the operator to take one bite down one side and then attempt to clear the remaining section with the grab constantly wanting to kick off back towards the void. While it can be done by utilising clamps on the grab to allow it to run down the stop end, it has a significant impact on the production rate and can affect quality of the wall joints. The detailing of these panel lengths would have greatly benefitted from direct input from a diaphragm walling contractor who could have assisted with selecting panel sizes to best suit current methods.

Figure 4 - Dwall operations

Transfer slab – Rotary bore piling under bentonite:

On the Northern transfer slab Crossrail perceived there to be a risk that constructing the piles within 3m of the DLR retaining wall would result in movement of the wall exceeding its trigger levels as a result of possible flighting that is potentially associated with CFA. It was deemed too great of a risk by CRL who opted instead to have the piles constructed rotary bore under bentonite. The thought process was that by casing down through the unstable River Terrace Gravels to the clay and Lambeth Groups below, the risk of potential collapse would be negated. The pile excavation could then continue under bentonite from there down to toe level.

The 900mm diameter design of 41no bentonite piles to be installed to a depth of 33-35m was developed in response to this concern. The depth to the clay and residual concern regarding the method of installing temporary thin wall casings meant that segmental casing had to be employed. The design diameter however did not lend itself to suit the readily available segmental casing sizes. It was fortunate that Bachy Soletanche had some rare 1m casings in stock, otherwise the pile would have had to be constructed to the larger 1180mm diameter which would have had potential implications for a redesign due to reworked pile spacings. Had the designers involved a specialist contractor at an earlier point in the design phase, they could have together developed a solution that meet the design requirement but also took advantage of those tools readily available (see table below) and techniques that would have been the most cost effective. While designers can optimise their designs to minimise the material costs, without insight into the construction processes, they can be inadvertently increasing the overall build cost of a project.

Technical Data - double-walled casings

	Nutzlänge / *effective length* L (m)									
D1/D2 (mm)	1m	2m	3m	4m	5m	6m	a1	a2	t2	Schrauben *bolts*
	Gewicht / *weight* (kg)						mm	mm	mm	(Anz. / *No.*)
620/540	403	739	1074	1411	1747	2081	12	8	40	8
750/670	492	902	1311	1722	2131	2540	12	8	40	10
880/800	585	1069	1552	2036	2520	3005	12	8	40	10
1000/920	669	1221	1773	2326	2877	3429	12	8	40	10
1180/1100	844	1580	2316	3052	3787	4522	16	8	40	12
1200/1120	872	1620	2370	3120	3870	4620	16	8	40	12
1300/1220	933	1746	2558	3372	4184	4995	16	8	40	12
1500/1400	1433	2625	3817	5009	6201	7393	20	10	50	12
1800/1700	1730	3166	4602	6038	7474	8910	20	10	50	16
2000/1880	2450	4280	6110	7940	9770	11600	20	15	60	12
2200/2080	2700	4720	6740	8760	10780	12800	20	15	60	12
2500/2380	2960	5240	7520	9800	12080	14360	20	15	60	16

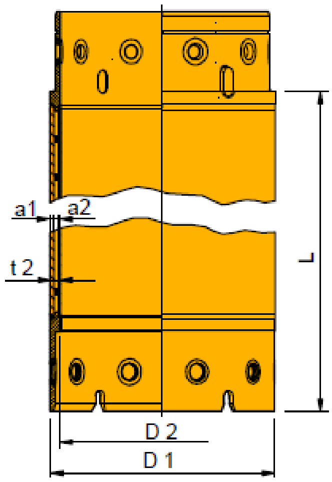


Figure 5- Stock sizes of segmental casing

Cut & Cover Tunnel – Cased CFA (CSP) Secant piling:

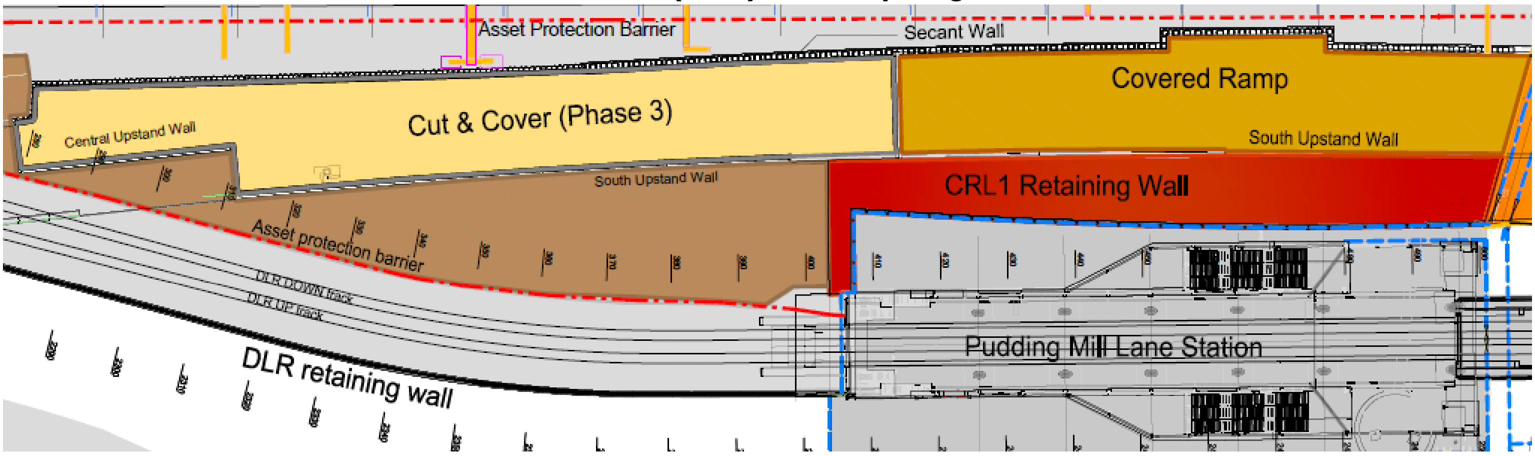


Figure 6-Secant wall layout

Where the northern wall of the cut and cover tunnel drew within 10m of the Network Rail corridor there was considerable concern that the construction of this wall may lead to differential settlement of the adjacent rail lines. The client's design team opted for a secant wall solution.

The wall design required involved the installation of 1050mm diameter hard-hard wall with piles extending down 20.5m. Again the design utilised a diameter that did not match the readily available stocks of segmental casing. To construct the wall by traditional rotary bore piling utilising specially built segmental casing would have had significant cost and programme implications to the project. The proximity of the piles to the Network Rail boundary (in some places as little as 2m from hoarding and 7m from the running rail) would have also made it difficult to construct the piles during normal working hours. The width of the site and angle at which a traditional crane would have needed to be orientated, to avoid it's collapse radius crossing over rail assets, necessitated that the works be constructed during engineering possession hours. The possession hours were only available on weekends and not always on consecutive weekends. This when combined with the restrictions of secant wall sequencing would have resulted in Bachy attempting to cut some of the female piles up to 28 days after they were first constructed. It would have greatly increased the risk of cracking the female piles and would also have drastically increased the drilling times thus exacerbating the problem of what could be achieved during the limited hours of the possession. The number of lifting operations associated with traditional rotary secant piling would been difficult to manage with the limits that were imposed by Network Rail.

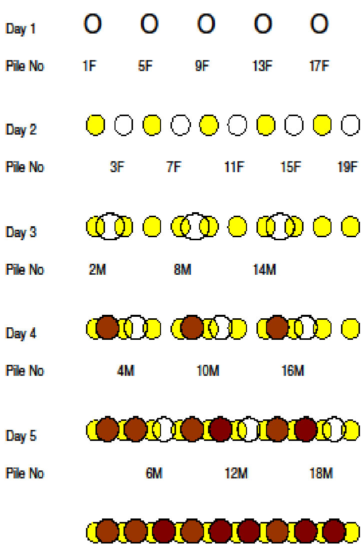


Figure 7 – Typical secant wall piling sequence

Bachy Soletanche decided that Cased CFA (CSP) piling would be best suited given the operating environment stated above. At time of tender the piles were outside the capabilities of the available CSP rigs in the country and thus required a larger Bauer BG46 rig to be sourced from Germany. The rig had sufficient torque to cut the female piles to the required depth and enough ballast to take the weight of the full drill string and rotary table.

To overcome the issue of the bespoke pile diameter, Bachy commissioned a custom drill string and full length casing to be fabricated. The full length casing was of thin wall construction with a cutting shoe connected to the bottom. The cost of a full length casing was significantly cheaper than that of bespoke segmental lengths but had the disadvantage of being cumbersome to fix to the rig and was not as robust.

The design also called for the female piles to be reinforced to the full depth of the pile. The original detail for these piles in the construction drawings called for a square cage, however this increased the risk of striking steel during the excavation of the male piles as they cut through the females. The tolerance on placement of the female cage to ensure that the clearance between the outside edge of the cage and the cut was considered too tight. Together with

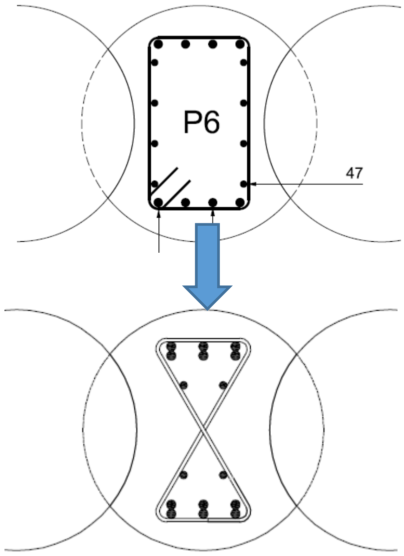


Figure 8- Butterfly detail

the designers from C152, a butterfly shaped caged detail was agreed upon during pre-construction planning. It was fortunate that it was a detail that the designer had seen before on a previous contract Bachy had conducted for them as it dramatically reduced the time it took to get approved. Other potential cost saving initiatives were tabled but declined due to the amount of time it would have taken to get these other proposals approved by a CAT 3 checker. As it was the CAT3 checking of the steel detailing caused significant delays to the construction programme. It had a significant impact on the projects ability to implement cost engineered solutions as the time taken to approve any such solution was such that the window of opportunity within which these solutions could be applied, and be effective, was often missed.

Constructing the 282 wall piles via CSA meant that 282 pile cages had to be plunged to a depth of 20.3m into piles that had been constructed through the Lambeth Group with its sandy layers. The sands had the effect of drying out the concrete reducing its workability and potentially impeding cage plunging. It made for challenging conditions through which to plunge reinforcing cages. To give the best chance of ensuring that the cages were successfully plunged to depth, Bachy spent extensive time with their selected concrete supplier developing a high flow concrete supply with a 5hr workable life that met the requirements of the specification. Furthermore, the team looked to cut down the time it was taking to concrete the piles to reduce the time between when the concrete at these sandy layers was placed and when the cage was plunged.

The restrictions placed on crane's slew radius for works taking place near rail assets, when combined with the narrow site and close proximity to the Network rail line, required a unique solution to allow for the cages to be installed without impeding pile installation.

Bachy opted to utilise a leader rig to lift and install the cages. The leader rig due to its classification as a piling rig, was able to move and orientate itself towards the railway lines unlike a traditional crane. This allowed for the piles to be constructed without impeding the CSP rig and negated the potential significant impact to the construction programme.

Figure 10- Leader rig with cage suspended

Figure 9- BG46 in close proximity to rail assets

Conclusion of lessons learnt:

Pudding Mill Lane is significant in the fact that it utilised multiple different techniques in a very challenging environment, allowing for parallels to be drawn between the viability of one technique over that of another. The project demonstrates that heavy piling can be effectively conducted adjacent to live rail assets but requires significant levels of forward planning to mitigate the risk posed to these assets.

With respect to what could be done better, there is one consistent factor that has lead to issues along the way with this project. It is the apparent lack of understanding by the design community of the piling techniques and processes and their limitations. The impact a foundation design can have on the cost and programme by not being suited to construction by methods readily available cannot be understated.

Large schemes such as Crossrail in future would greatly benefit from having pile designs reviewed for practical implications by a working group consisting of FPS members. Such a working group could identify at an early stage potential issues with designs and also cost saving solutions. This early identification would allow for sufficient time to have the changes implemented and approved before they have a chance to impact construction programmes.

Examples of Specialist Grouting on Contract C310 Thames Tunnels

R.A. Smith CEng MICE

Keller Ltd

ABSTRACT

The paper focuses on specialist grouting works undertaken by Keller Ltd on the Contract 310 Thames Tunnel, providing a review of three completed pieces of works. The first is a compensation grouting scheme used to protect a Network Rail Substation. Due to the shallow depth between the TBM and the foundation of the building, the space available for the compensation scheme was less than 5.0m and one of the key risks which needed to be controlled was the potential of grout ingress into the TBM. The works were completed within the specified movement criteria of +5/-10mm and no grout loss was recorded within the TBM face.

For the C310 project, the cross passages are located within the highly permeable chalk layer and therefore, to allow the construction of the cross passages to be completed, the permeability of the chalk needed to be reduced to an acceptable level. At cross passage 19, the Crown of the Cross Passage was very near the interface between the chalk and the Thanet Sand. A normal fissure grouting scheme would not have been able to treat the Thanet sands, therefore a combined jet grout and chalk fissure grouting solution was designed and implemented by Keller.

Keller were employed to provide ~~a~~ design and construct reception shaft for a pipe jack machine. The space available was very limited. Due to this, a secant wall shaft using a restricted access piling rig was proposed. The ground conditions showed a very high ground water level and a jet grout base slab was proposed by Keller.

Introduction

Keller has been employed by the Hochtief Murphy Joint Venture (HMJV) to undertake a combination of ground treatment and settlement mitigation measures on the Crossrail C310 Contract. The works have been completed to a very high standard and due to a good collaborative working relationship which has developed between HMJV and Keller, a number of additional design and construct works packages have been completed by HMJV and Keller.

Cathedral Substation Compensation Grouting Scheme (Value £2.0million)

A Network Rail substation which provides the power supply to the North Kent Line – a busy commuter route into London, lay directly above the line of the proposed Crossrail tunnels. The substation is within 100m of the launch of the Tunnel Boring Machine (TBM). As shown in the section drawing below the TBM was still at a shallow depth when it advanced past the Cathedral Substation (less than 5.0m between the crown of the tunnel and the base of the building’s footings). Based on this, a settlement calculation completed by HMJV calculated that during tunnelling a maximum settlement of 20mm was predicted.

The building housed a number of sensitive power supply equipment and therefore a settlement control criteria of +5/-10mm during tunnelling was stipulated.

The photo below shows the sub-station building. It is a Victorian built structure with a 300mm thick floor slab with structural edge thickenings to support the main structure of the building.

Photo 1 – External View of Cathedral Substation

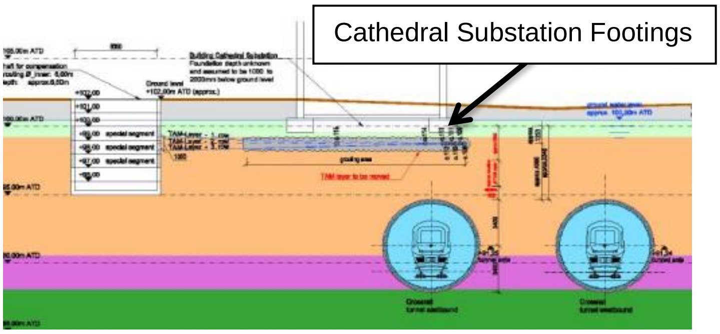


Fig 1 - Cross Section showing depth of TBM below the Cathedral Substation footings. Approximately 5.0m space between the base of the footing and the crown of the tunnel.

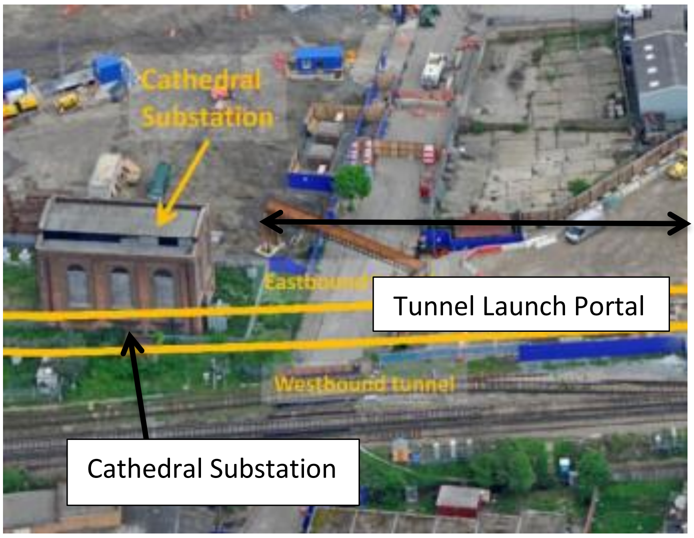


Photo 2 - showing the Cathedral Substation position in relation to Launch Portal for the Tunnel Boring Machine

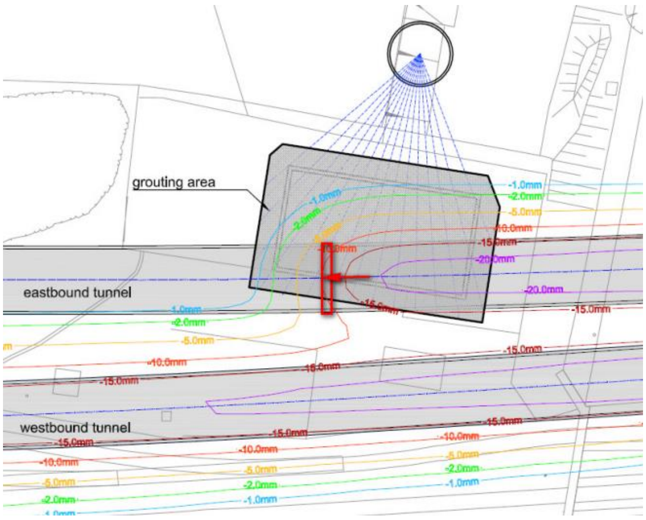


Fig 2 Layout Drawing showing settlement predictions during TBM Passage passed Cathedral Substation

HMJV were employed by Crossrail to develop a settlement mitigation measure, to control the settlement during the TBM drive to the stipulated levels. Through early sub-contractor involvement with Keller, a compensation grouting scheme was developed. Compensation grouting is a relative new and innovative geotechnical technique, first used in the UK on the Jubilee Extension Line. (Mair 1998)

Compensation Grouting is a three phase drilling and grouting process which uses a surveying system installed around the building to accurately record movement in real time.

The photo below shows the Hydrostatic Levelling Cells (HLC) which were installed around the Cathedral Substation. The information from these cells was transferred via an internet connection to a database and this information was shown on a drawing using the Grout Control Software. Below is screen shot showing the HLC results in real time.

Photo 3 - showing the HLC installed around the outside of the Cathedral Substation. Monitoring system was installed by Keller Getec.

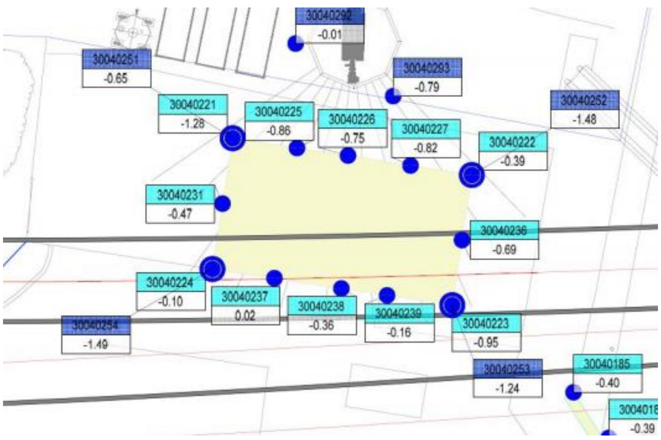


Fig 3 Grout Control Screen Shot– HLC recording being shown,

The first phase of the compensation grouting scheme is the installation of a horizontal array of Tube a Manchette (TaM) pipes. An access shaft is required to allow access for the rig to install the TaM pipes. Using a specially modified restricted access drilling rig, temporary casing was installed to the required depth. TaM pipes in 2.0m lengths are installed to the required length and connected using a thread connection. Please see photograph below which shows the 'shaft rig' installing the temporary casing required to install the TaM pipes.

A modified Klemm Rig was used for the shaft rig with a duplex drilling head. This allowed the advancement of both the augers and casings as required. The drilling system used were drilling rods with drag bit using a water flush. The casing diameter was 114mm OD.

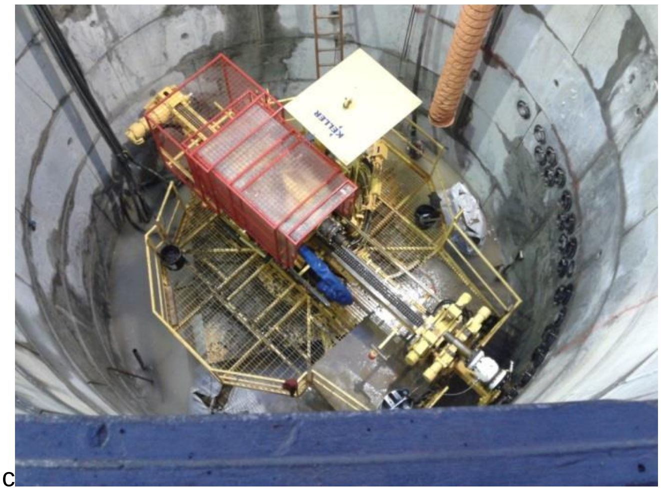
c 

Photo 4 – Shaft Rig installing TaM pipes

Construction Risk Overcome

Due to the shallow depth between the TBM and the foundation of the building, the space available for the compensation scheme was less than 5.0m and one of the key risks which needed to be controlled was the potential of grout ingress into the TBM face during concurrent Grouting. .

This risk was managed by using a three array layout for the scheme as shown in the section drawing below. The top and bottom array acted as a protective grout slab when concurrent grouting was being completed as the TBM advanced. In addition using a traffic light system, a detailed drawing was produced which showed which TaM pipes could be used as the TBM progressed beneath the building. This system was successfully implemented during the works.

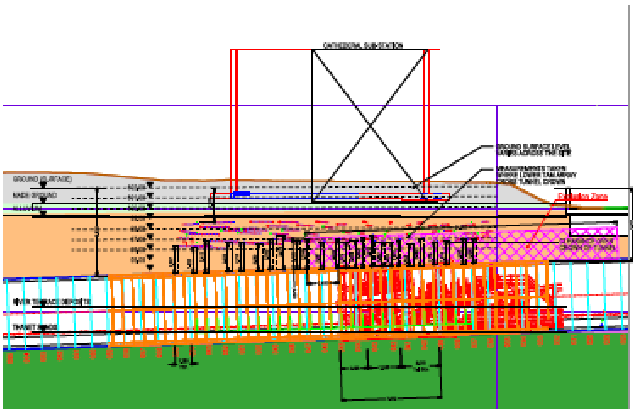


Fig 4 - Section drawing showing the three levels of TaM installed in relation to the TBM drive and the Substation footings.

The drilling works were completed from within a shaft, approximately 5.0m below ground water level – which equated to 0.5Bar of back pressure. When drilling works were being completed, there was an increased risk of settlement fine loss into the shaft. To overcome this risk, a Stuffing Box was used to drill through, this controlled the water ingress into the shaft. To actually install the TaM to the required length proved difficult, as they were being pushed passed a 0.5Bar backed pressure and therefore a flushing mechanism for the TaM pipes was developed which allowed the TaM pipes to be pushed to full depth.

The second phase is a pre-treatment phase. Using the TaM pipes grout is injected using controlled pressures and to a set system. An innovative approach used on this scheme was the combination of the pre-treatment phase with the drilling. This allowed far better control of settlement induced during the drilling works This meant that grout was injected in a controlled fashion as soon as TaM pipes were installed ensuring no significant settlement was observed during the drilling phase.

A software system called Grout Control was successfully used on this project. The software allows grouting to be implemented in real time to allow quick reaction to settlements observed. Provided overleaf is a typical drawing provided by the software. The drawing shows a proposed grout pass shown in green based on the settlement recorded by the HLC installed on the building.

Using a Maxibor surveying tool, 3D Modelling of the as-built position of the TaMs was complete to ensure none were in the line of the TBM. This also allowed the 3D model to be incorporated into the BIM model for the scheme.

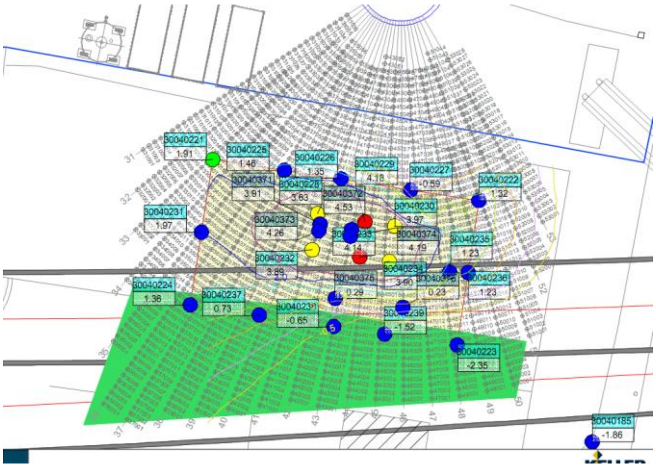


Fig 5 - Drawing produced by grout control software showing the position of the proposed grout pass.

The works were completed to a very tight programme working 24/7, the pre-treatment had to be completed to the required contractual dates else the Tunnel Boring Machine would not have been permitted to commence. This was successfully achieved.

Concurrent Grouting

The graph shows the control available from the grouting system. As soon as a settlement trend was noted, a grouting pass was initiated which jacked the building back to level. The works were completed within the specified movement criteria of +5/-10mm and even better settlement control was achieved with no movement greater than +5mm/-5.0mm being recorded.

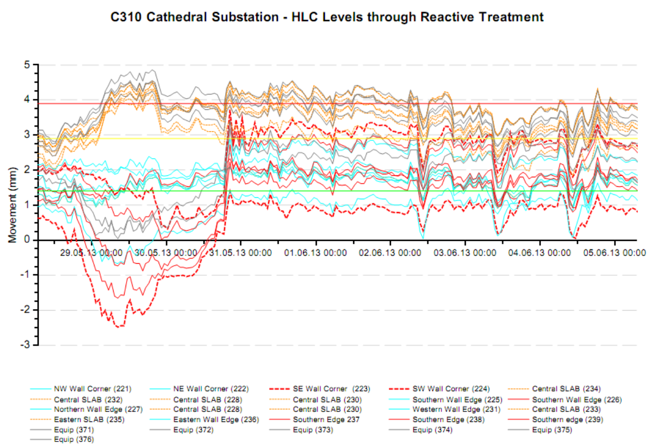


Fig 6 - Graph showing the movement of each of the HLC installed at the Cathedral Substation during the concurrent grouting phase as the TBM passed beneath the building

Cross Passage 19 Jet Grouting and Chalk Fissure Grouting

Value - £1.0million

Between the two Crossrail running tunnels, emergency cross passages are to be constructed by HMJV. The cross passage are located within the highly permeable chalk layer and therefore to allow the construction of the cross passages to be completed, the permeability of the chalk needed to be reduced to an acceptable level. At one of the cross passage locations, the Crown of the Cross Passage was very near the interface between the chalk and the Thanet Sand. Therefore a normal fissure grouting scheme would not have been able to treat the Thanet sands, therefore a combined jet grout and chalk fissure grouting solution was designed and implemented by Keller.

A 4.0m jet grouted slab was constructed between the interface between the Thanet sand and the weathered chalk layer. The slab extended the full length of the cross passage (approximately 20.m) and 3.0m beyond either side of the cross passage. Interlocking jet grout columns were installed to 1.75m diameter. The jet grout was installed between 12m and 16m below ground level.

To confirm the column diameters achievable in the ground conditions Keller completed a number of trial columns. To assess the diameter of the columns Keller used the innovative technique of temperature measurement. The temperature increase in the centre of the column is monitored and compared with the result of a thermos-chemical analysis of the hydration process in the column. (Meinard, Adam & Lackner). When back analyses of the grout mix of 1:1:0.01 CEM II/Bentonite was completed it demonstrated a column 1.75m in diameter was achievable.

To comply with the Crossrail specification, a Falling Head Tests and laboratory testing on cored samples were completed. These demonstrated the jet grout slab achieved a permeability of better than 1 x 10^{-8}ms^{-1}.This demonstrated the works were in compliance with specification.

To ensure a complete slab was provided at Cross Passage 19, Keller used a SAA surveying tool to provide the 3D model. The photo overleaf shows Keller's Site Engineer installing the SAA through the jet grouting drilling equipment.

Photo 5 – SAA tool being installed into jet grouting drilling tools.

Using the results of the SAA survey, a 3D model of the completed column slab was completed. Using this model, an assessment of any gaps between the columns could be completed and if required additional columns were installed. Below is the 3D model completed at Cross Passage 19.

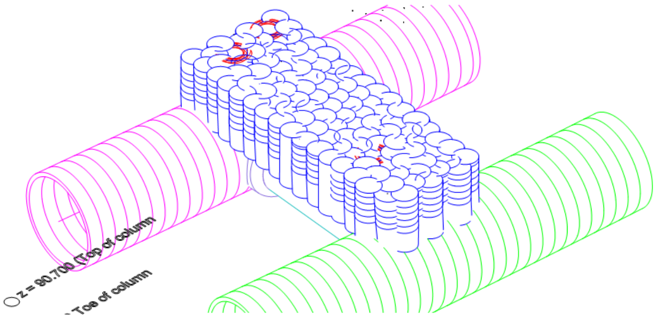


Fig 7 – Figure showing the completed 3D model of the jet grout columns installed at Cross Passage 19.

A secondary chalk fissure grouting was then implemented below the jet grout slab to a maximum depth of 25.0m. This was completed using a descending stage technique. Lugeon water testing was used to verify the woks complied with specification. 95% of tests were less than 2 Lugeons and 100% were less than 5 Lugeons. Photo 6 shows HMJV beginning the excavation of the Cross Passage.

The table below shows the grout take used at XP19 when compared to chalk fissure grouting on the CTRL cross passages: the table demonstrates that poorer quality chalk can be treated using a combination of jet grouting and chalk fissure grouting although the grout take is significantly higher.

Cross Passage	TBM Face Geology	Ground Treated (%)
CTRL XP2	B5	7.7
CTRL XP3 and Nadir	A2/A3	1.2 and 0.8
CTRL XP4	C3	4.4
Crossrail XP19	C4/5 – C2/3	9.0

Table 1 – Grout take used on chalk fissure schemes on CTRL and Crossrail schemes (Warren 2002).

Photo 6 - Exposed face at XP19.

3 Marmadon Road Reception Shaft

Value - £600k

Background to the Project – Keller were employed by HMJV to provide a design and construction of reception shaft for a pipe jack machine. The pipe jack was required to replace an existing sewer which clashed with the proposed tunnel portal structure.

Solution Provided – The position and access of the reception shaft was very constricted. The position was within a very small back garden of a terraced house which backed onto a railway line. Due to this, a secant wall shaft using a restricted access piling rig was proposed. The ground conditions showed a very high ground water level and due this a jet grout base slab was proposed by Keller. This was completed using a restricted access jet grout rig.

The secant wall was installed to using 508/450mm diameter hard pile and a 450mm soft pile. Both the hard and soft piles were installed to a depth of 7.0m. The excavation depth for the shaft was 5.0m. The jet grout slab was created using interlocking 1.75m diameter jet grout columns, varying in length 2.0m – 6.2m.

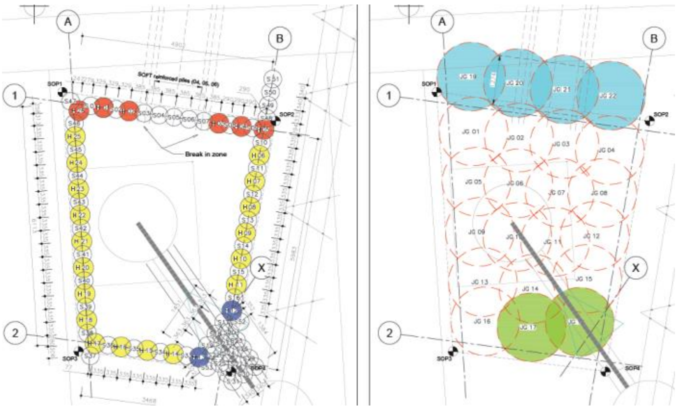


Fig 8 - Layout of the secant wall and the jet grout slab.

Photo 7 – Mini-piling installing 450mm diameter piles for the secant wall. Very limited space provided on site.

Photo 8 – Shaft being excavated during the works

Photo 9 – Shaft excavated to top of the jet grout slab. The surface water shown is from heavy rainfall prior to taking this photo.

All this work was completed remotely from the main site as there was no space on the site except for the rig and an attending excavator. All concrete had to be delivered to the main site and then pumped beneath the railway line to be used on site. This required control of the concrete and grout mix used to minimise the risk of blockage.

This works is provided as another example of early sub-contractor involvement within a scheme and therefore the risks of working in a confined site, close to a railway line were adequately controlled. All works have been completed and were completed within the required programme duration with no incidents recorded.

A key sustainability benefit completed from these works was that the spoil generated from the jet grout production was used to backfill the shafts used for the compensation grouting works.

References

Meinhard, Adam & Lackner Temperature Measurements to Determine the Diameter of Jet Grouted Columns,

Warren, Hughes, Swift, Mielenz (2002) Cross Passages Ground Treatment for the CTRL Thames Tunnel

Mair (1998) Recent experiences of tunnelling and deep excavations in London

Crossrail Lessons Learnt

Compensation Grouting

O.J. Francis[*1] and R. Hayman-Joyce[2]

[1] *BAM Ritchies, Glasgow, United Kingdom*
[2] Keller Geotechnique, Watford, United Kingdom

[*] Corresponding Author

ABSTRACT

Compensation grouting was selected by Crossrail as a settlement mitigation measure at a number of key locations along the route. This paper describes the works on the western section, as part of contracts C300 and C410 where the compensation grouting was carried out from 13 Nº shafts 4.5m diameter and up to 20m deep. Some 50km of Tube à Manchette was installed, to enable grout to be injected at the required locations. Discussion points include the information provided at tender stage, the updated information received on contract award and the conditions encountered on site. The design of the system is discussed, along with the division of risk and responsibilities between the parties. Details are included on the drilling, installation and commissioning of the compensation grouting system. This includes the methodology we selected and the factors affecting our decisions. During the concurrent grouting phase of the works, the circular process is discussed, from the initial notification of excavation, through the design of the grout pass and the execution of the works to the feedback of the results to improve the process for subsequent excavation advances. During this cycle, the constant interaction with other teams is essential to the success of the operation. The monitoring of buildings, the ground and excavations was outside of our direct scope of works, but we worked in close partnership with the monitoring sub-contractor throughout the project. The method of controlling each grout injection is described, including the technologies employed to monitor the effect of the grouting in real time on site. Through good teamwork and by using the latest technology, we were able to carry out accurate compensation grouting so that the buildings on the surface experience as little movement as possible during the construction of the tunnels below.

1 Introduction

Compensation grouting was specified as one of the settlement mitigation measures to be used on Crossrail. The last time this technique was used to this scale was during the extension of the Jubilee Line in the 1990s.

The Crossrail scheme is split into a number of specific contracts. The focus of this paper is contract C300 – The Western Running Tunnels from Paddington to Farringdon and C410 – The station caverns at Bond Street and Tottenham Court Road. The Main Contractor for both of these contracts was the BAM Ferrovial Kier (BFK) joint venture. The compensation grouting sub-contractor was a second joint venture of Keller and BAM Ritchies (KBR). A key element in any compensation grouting project is the information gathered by the instrumentation and monitoring. In this case, this formed a separate, significant sub-contract in itself and was not part of our scope of works.

Although BFK had split the compensation grouting and monitoring into two separate packages, we worked together as one team. Had the two packages been awarded to competing organisations, this relationship may not have been so seamless.

2 Site layout

On this contract, the compensation grouting was carried out from shafts specifically constructed for the purpose. The grout shafts had an internal diameter of 4.5m at the working height, but due to the congested nature of services in central London, some of the shafts had a reduced opening toward the surface. There were a total of 13№ grout shafts over the project, 5№ at Bond Street, 7№ at Tottenham Court Road and a single shaft at Fisher Street where a ventilation shaft is located. Finding suitable locations for the grout shafts was not an easy task to begin with, as each location had to satisfy all of the following criteria:

- Enable full coverage of the required area for compensation grouting treatment.
- Allow any services to be suitably diverted.
- Include a working area for the compensation grouting operations.

On completion of the works, the grout shafts will be backfilled and the ground reinstated but they need to remain operational for a number of years. During this time, the re-development of nearby buildings also needed to be considered. Every effort was made to minimise the disruption to the public, including the locating of one of the grout shafts within an existing disused tramway tunnel.

The tender information included detailed information on the available space around each grout shaft. Although this did prove to be accurate, it still had a significant impact on our works. Given the limited space available at each shaft, a compound area at each station would have been beneficial.

During the works, we were made aware of agreements between CRL and other stakeholders, such as granting access over a working grout shaft and interactions with other contractors. The relatively short notice of these changes caused delay and disruption to our works.

3 Drilling

The compensation grouting system consisted of a number of arrays of Tube à Manchette (TàM) pipes which were drilled radially from each of the grout shafts. TàM pipes consist of tubes with ports at regular (in this case 0.5m) intervals, covered by a rubber sleeve which acts as a one way valve. Combining this with a double packer (a pair of inflatable seals) allows grout to be injected at very specific locations in the ground.

Some of the TàM pipes installed extended over 100m from the grout shafts, although there was a significant reduction in accuracy and production rate when compared to lengths of up to 60m. The layout of the TàM arrays was governed by a number of factors:

- The outer boundary of the treatment area had to meet with the client's requirements.
- The length of individual TàM was kept to a minimum.
- The plan spacing at the distal end of the TàM was limited to a maximum of 3m (4m including deviation). This proved to be a suitable limiting criterion in the ground conditions and tunnel depths of this project.
- The plan spacing at the proximal end of the TàM, on the inner face of the grout shaft, had a minimum limit in order to maintain the structural integrity of the shaft.
- The maximum elevation of each TàM had to be such that there was a minimum of 2m cover of London Clay over the working length of the TàM.
- The minimum elevation of each TàM had to be such that it was outside of the grouting exclusion zone of the tunnel.
- There were limitations on the vertical spacing of the holes through the shaft lining.
- Practical matters such as positioning boundaries between shaft arrays along road centrelines or outside building footprints.
- Where possible, distributing the total area equally between each of the grout shafts.
- Programme considerations such as assigning a particular section of tunnel to a particular shaft.

Three dimensional modelling was used to design all of the TàM arrays, ensuring that all of the above criteria were met. Drilling and installation schedules were produced directly from the three dimensional model. A view of Tottenham Court Road station can be seen in Figure 1.

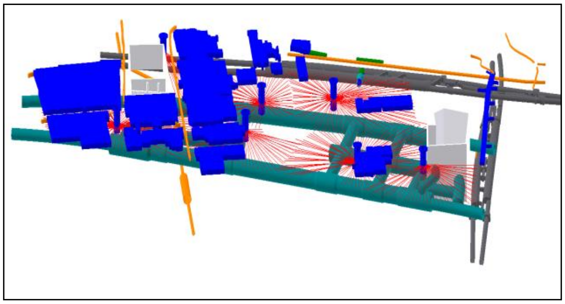


Figure 1. Three dimensional view of Tottenham Court Road Station

The detailed design of the TàM arrays was required relatively early in the contract. At this time, the information provided "for construction" was only 2D drawings. We found it difficult to obtain 3D models and these were only issued "for information" as they often contained inconsistencies. Important information such as the location of obstructions and the profile of the London Clay surface were detailed on other drawings, but not included in the 3D models we were provided with.

At the beginning of the project, two different methods of drilling were trialled:

- Water Flush
- Auger

Having completed one grout shaft with each method, it was decided to progress with the auger method. Although this did generate some settlement, it required less equipment at the surface and involved more pleasant working conditions for the operatives in the shaft.

Table 1. Summary of TàM arrays installed

Location	Holes [Nr]	Total Length [m]
Bond Street Station	398	20,035
Tottenham Court Road	480	22,870
Fisher Street Shaft	65	2,333
Total	943	45,238

Once installed, every TàM pipe was surveyed along its full length, giving an accurate three dimensional as-built path of each individual installation.

The tender information included an outline design for the TàM arrays. This proved to be generally similar to our final detailed design. We were required to include an allowance of 20% for re-drilling out of tolerance TàM installations. In reality, we found 10% to be sufficient.

The drill rigs were customised to work within the 4.5m diameter grout shafts. The space required for break out jaws and the drilling head reduced the length of the drill string segments to 1.0m. Increasing the diameter of the shaft would increase the production rate by reducing the frequency of rod changes.

4 Pre-treatment grouting

This phase of grouting was essentially the commissioning of the system. The process was deemed to be complete when heave of between 3 and 5mm had been generated over the whole area of the grouting arrays. The activity of installing the TàM pipes did in itself cause some settlement, but this was more than compensated for during the pre-treatment grouting phase.

As this was the first activity which intended to cause changes to the ground level, it was done gradually over a number of passes. Each grout pass generated approximately 0.5mm of heave. This gradual approach provided the client and asset owners with confidence in our ability to control the ground profile to a high degree of accuracy.

During this phase of the works, the Grout Efficiency Factor (GEF) was measured. This is a measure of the heave generated per unit volume of grout injected and the loss of grout into the surrounding ground (without generating heave).

The completion of the pre-treatment is often linked to Key Dates under the terms of the contract as the area is then ready to react to any settlements that may occur as a result of the planned excavations.

A low strength grout was used to facilitate repeat injections. The grout mix had a water:cement ratio of 2.0 and a nominal quantity of Bentonite in order to limit the bleed. Whilst we appreciate the requirement for quality control by viscosity and density testing, the benefit of expensive Modulus of Elasticity tests was not understood.

Although the GEF varied around the site and changed with time, a value of 0.4 was generally observed.

5 Concurrent grouting

As the name suggests, this is where grouting is carried out concurrently with excavation activities and forms the bulk of our works. The excavation activities fell into three categories:

- Station Box Excavation
- Tunnel Boring Machine
- Spray Concrete Lined

The settlement generated by each of these techniques was dealt with in a different way.

5.1 Station Box Excavation

The layouts of the Crossrail stations generally include a large box at either end of the station. The perimeters of the boxes were formed using secant piles or diaphragm walls. The inside of the boxes were then excavated form the top down with props installed as the excavation progressed. Although settlement was observed in the surrounding area and deflections were monitored in the walls and props of the excavation, they were not significant enough to warrant compensation grouting to be carried out continuously. We were able to monitor the settlement and prepare reactive grouting proposals – more akin to grout jacking than concurrent grouting.

5.2 Tunnel Boring Machine

Two earth pressure balanced Tunnel Boring Machines (TBMs), Phyllis and Ada were used to form the 6.9km long tunnels from Westbourne Park to Farringdon. Compensation grouting facilities were not required along the entire length of the tunnel as the passing of a TBM alone does not generally cause sufficient settlement to require compensation grouting as a mitigation measure. At the location of the stations, however, the cumulative settlement caused by all the excavation activities warranted the installation of the grouting scheme and hence it was prudent to use the installed system to manage the settlement, regardless of cause, for the duration of the contract.

By the time the TBMs arrived at the first compensation grouting array, it had already tunnelled a considerable distance. This gave ample opportunity to measure the actual settlement caused by the passage of the TBM without the use of compensation grouting. The results of the monitoring demonstrated that the magnitude of the settlement was well within the allowable limits.

When planning the resource levels on site, one of the scenarios with the potential to create a peak demand was the passage of the TBM at its maximum rate of excavation and on the maximum allowable limit of volume loss (leading to settlement). The low volume loss observed from the TBM passage and the grout efficiency factor observed during the pre-treatment, allowed for the preparation of compensation grouting passes which maintained the ground level within acceptable limits, did not hinder the progress of the TBM and could be carried out from the existing grout shafts and surrounding compounds.

5.3 Spray Concrete Lined

The station caverns and a number of connecting passages were all formed using Spray Concrete Lined (SCL) excavation techniques. The TBM excavated running tunnels were enlarged to form platform tunnels using SCL techniques. The high concentration of excavation in the vicinity of the new stations made this the largest contributor to surface settlement and thus the bulk of the compensation grouting was focused on the SCL excavation.

In order to maintain the best possible control of the surface levels, each small advance of SCL excavation was accompanied by a concurrent grouting pass. The grout passes were each designed individually to compensate for the predicted settlement caused by the accompanying excavation advance. The site teams responsible for each of these activities were in constant communication with each other, to ensure that the two activities remained concurrent. This was important as it affected not only the settlement profile but the safety of the underground operations.

6 Grout jacking

Whilst the grout passes for concurrent grouting are proactive and designed on the basis of predicted settlements, grout jacking passes are reactive and are designed in response to observed settlements. The results from an assortment of monitoring systems were used to produce surface contour plots. These were commonly used to define the limits and intensity of a grout jacking pass. The mechanism is similar to the pre-treatment phase in that heave is generated in increments of approximately 0.5mm.

If a significant amount of grout jacking was required following a concurrent grouting pass, this information was fed back so that the intensity of the concurrent grouting passes could be increased, leading to a more effective balance of settlement and heave.

In some cases, it was not clear which specific activity had caused the settlement. There were activities being carried out by other contractors on nearby sites which had an influence on the behaviour of the ground. Having installed the compensation grouting system, our role was to respond to the settlement as it occurred, without necessarily investigating the cause of each incident.

On completion of any excavation activity in an area, grout jacking was required for up to three months as the ground continued to respond to the excavation activities.

7 Information exchange

Our aim as a grouting contractor was to carry out grouting that mitigated the settlement caused by the excavation as accurately as possible. In order to do this, we relied on a continual flow of information between ourselves, the excavation team and the monitoring team. The primary route for this information exchange was during the Shift Review Group meetings and also during the weekly Contract Technical Committee meetings where asset owners were also invited to discuss the current activities.

This system of meeting proved to be an excellent forum for exchanging information.

7.1 Excavation details

The excavation team provided full details of each planned advance. This included the co-ordinates of the start and end point as well as the volume to be excavated, the anticipated volume loss and the programmed time for each advance to take place.

At tender stage, the volume of each tunnel section was not readily available and had to be calculated from a number of 2D sections.

7.2 Grout pass design

From the excavation data supplied, the as-built location of the TàM arrays were added at this location and the specified exclusion zone calculated to ensure that the grouting did not cause increased stresses on unsupported sections of excavation.

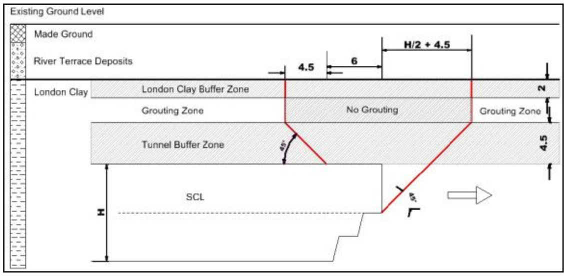


Figure 2. Grout exclusion zone for SCL tunnels

The exclusion zone shown in Figure 2 shows a minimum clearance above the crown of the tunnel, below the upper surface of the London Clay, in advance of and behind the excavation.

An assessment of the anticipated zone of influence was made, which defined the outer limits of the specific grouting ports which may be used in a particular grout pass. Figure 3 shows an example excavation advance with the inner exclusion zone and the outer zone of influence along with some proposed injection locations.

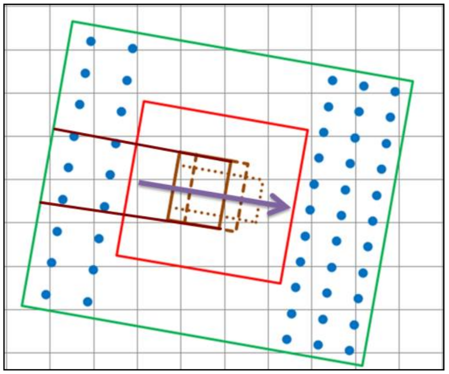


Figure 3. Example plan view of ideal injection locations in relation to a tunnel advance

From the volume of ground to be excavated, the anticipated volume loss and grout efficiency factor, a total volume of grout to be injected was calculated, to compensate for the predicted settlement.

Having established the limits of the grouting and the exclusion zone, a distribution of grout was designed. This took into account the distribution between the injections carried out ahead of the excavation and those which could take place after the excavation has passed. The cross sectional distribution of grout used a higher intensity of grouting along the excavation axis, where the settlements were predicted to be at their greatest and tapered towards the outer edges to minimise the gradient at the outskirts of the treatment area.

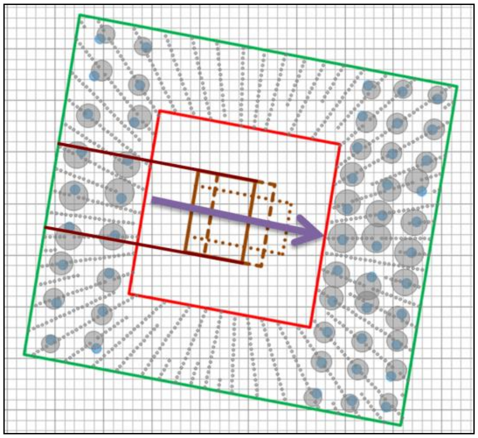


Figure 4. Example plan view of actual injection locations in relation to a tunnel advance

The final step in the process was to select actual grout ports for the injections. Figure 4 shows the same example grout pass as Figure 3, but with the as-built grout ports highlighted. Efficiencies could be gained by selecting grout ports within the same TàM pipe as this reduced the time required to place the packer at each of the required ports.

Each designed grout pass was presented in tabular and graphical format, along with the accompanying excavation information. The proposed grout passes were circulated to both technical and non-technical members of the team so the information was presented accordingly.

7.3 Injection control

The high level of control required for these grout injections required the use of a computer controlled bank of pumps known as a grout module. Once a grout pass had been approved, it was then passed on to the grout module operator for implementation. The data behind the grout pass was used to generate a paper pro-forma for the module operator as well as a data file for the module computer. Once ready, the file was loaded and each grout injection was then able to terminate automatically at the required volume, without exceeding the pressure limits set.

In addition to controlling the grout injections, the computer in the grout module also displayed the surface level monitoring in real-time. As injections were being carried out, the module operator was able to observe the effect of the injection on the ground and, if necessary, he would terminate the injection manually if he observed that an unexpected amount of heave was being generated.

7.4 Monitoring feedback

On completion of each grout pass, the actual heave generated was monitored and assessed against the settlement which occurred. From these results, an assessment was made of the effectiveness of the concurrent grouting pass and any changes made to subsequent pass designs as required. This could be a global change, a change in grout distribution or a localised factor unique to a particular part of the site.

The system which collated all the monitoring data also included the as-built TàM positions, street maps and the location of the proposed tunnels and excavations. Having all the data in a single system made it easy to correlate settlement and heave with specific grout ports. An area could even be selected on screen, which generated a list of all the grout ports within a drawn area.

8 Conclusion

The process of generating heave by grouting, to balance the settlement caused by excavation requires a continual flow of information. By working together in a collaborative way and sharing information, the process can be refined to an efficient and accurate means of mitigating settlement.

By using current technologies, we have been able to analyse, produce and share vast quantities of data. As the construction of Europe's largest construction project continues, much of the general public remain blissfully unaware of the works going on under the feet.

SPECIFICATION & DESIGN VERIFICATION

Cementation
SKANSKA

Crossrail Conference – 10th November 2015

Specification & standard implementation across Crossrail contracts

Chris Robinson BEng MSc CEng FICE - Technical Manager, Cementation Skanska Limited, Doncaster UK

Cementation Skanska Limited (CSL) were employed on a large number of contracts across the Crossrail scheme, including four tunnel portals, three station boxes and one launch shaft. The involvement that Cementation Skanska had on the Crossrail project, when combined with a degree of hindsight, gave CSL a comprehensive overview of areas where there was a lack of consistency in specification interpretation / implementation and also a lack of standardisation of certain details.

The most significant areas which CSL believe would have benefitted from greater standardisation are:-

- Reinforcement cage detailing (e.g. standard diaphragm wall cage fabrication methodology)
- Implementation of BS EN 1992-1-1 (Eurocode 2) e.g. use of couplers within reinforcement cages and anchorage / lap lengths
- Embedded retaining wall integrity testing (to test or not to test?)
- Construction details at portal headwalls and shaft / station box TBM breakthroughs

There will be a balance between prescriptive implementation of standard details (to give consistency of delivery across the project and to maximise the extents of the supply chain) and stimulating innovation across the supply chain (sub-contractors and material suppliers for example).

This paper will describe the areas where greater consistency and communication across the entire project may have enabled more effective sharing of best (and emerging) practice across all parties involved in the project, from the Client to material suppliers, and also afforded the project enhanced quality of delivery and greater economies. Where lessons can be learnt these areas will be discussed to provide food for thought from which future large infrastructure projects may benefit.

Cementation
SKANSKA

Introduction

Through their involvement on a significant number of Crossrail contracts involving construction of embedded retaining walls, Cementation Skanska Limited (CSL) have had the opportunity to experience a variety of approaches adopted across these works, and compare good experiences with those which were, perhaps, rather more challenging.

This paper will describe the principal areas where greater consistency and communication across the entire project may have enabled more effective sharing of best (and emerging) practice across all parties involved in the project, from the Client to material suppliers, and afforded the project enhanced quality of delivery and greater economies.

Reinforcement cage detailing

General considerations

Details for reinforcement cages such as anchorage projection length, lap lengths, location of lapped bars & couplers (staggered or otherwise), and bond condition, was one area where considerable discussions were had with the Design Engineer on most of the contracts on which CSL worked (although the Crossrail Civil Engineering Design Standard is explicit on some of these points). When CSL undertook their first Crossrail contract works at Royal Oak Portal, the majority of the UK construction industry had relatively little implementation of Eurocode requirements (BS EN 1992-1-1: 2004[1]) for these reinforcement details.

One of the more contentious issues was that of the definition of “good” bond conditions (although the Crossrail Civil Engineering Design Standard is explicit on this point). On the early contract works undertaken by CSL there was an assumption that for any element constructed under bentonite support fluid then “good” bond conditions did not apply and therefore a greater lap / anchorage length was required. This is contrary to previously published guidance e.g. codes of practice BS8110[2] and BS5400[3], and also guidance within published papers, e.g. Jones & Holt 2004[4], although there was also existing published information agreeing with the adoption of “poor” bond conditions when designing to EC 2 (BS EN 1997-1-1: 2004[1]) since EC2 explicitly considers the potentially beneficial effect of enhanced cover to the lapped reinforcement bars. The situation was therefore somewhat confused. An additional detail to consider is the magnitude of bar stress at the level of the lap (or anchorage length) which could potentially significantly reduce the lap length if this were to be calculated rather than a generic “worst case” multiplier being adopted (although this is arguably significantly less burdensome to the designer / detailer). EC2 (and relevant execution standards) does state that laps should not be located in zones of high moment (Cl 8.2.2), lap lengths derived assuming fully stressed bars together with partial factors such as a material factor, γc, for concrete of 1.65 (for foundations) could be adding considerable conservatism into the reinforcement detailing solution.

Below is a typical extract from a Crossrail “General notes” drawing. This states the required number of bar multiples depending on the vertical location within the element (pile / panel) and also differentiates between bar sizes of 32mm or smaller, or 40mm. This is at variance to the guidance contained within the UK national annex (NA to BS EN 1992-1-1: 2004[5]) Table NA.1 subclause 8.8(1) which states large bars are those bars which are greater than 40mm diameter. The value of $\eta 2$ adopted will affect the design values of ultimate bond stress and therefore the lap or anchorage length required.

7. Anchorage lengths shall be in accordance with the following table (UNO) -

Concrete type	Vertical Location	anchorage length=n x diameter ≤32mm bar dia	anchorage length=n x diameter 40mm bar dia
C32/40	Within the top 300mm	49	53
C32/40	Below the top 300mm	34	37
C40/50	Within the top 300mm	42	46
C40/50	Below the top 300mm	30	32

Lap lengths for all bars in diaphragm walls and piles formed under bentonite slurry shall be determined from the above anchorage lengths increased by a factor of 1.4.

Fig 1, Extract from typical Crossrail "General notes" drawing

Where 50mm diameter reinforcing bars were used these were required to be coupled rather than being spliced, with the coupler levels being staggered, although it should be noted that EC2 (cl 8.8) does make provision for lapping of "large bars" (i.e. >40mm as defined by NA to BS EN 1992-1-1: 2004[5]) where the section has a minimum dimension of 1m or the reinforcement stress is less than 80% of the design ultimate strength.

Diaphragm wall reinforcement cages

The reinforcement cage detailing for diaphragm wall panels needs to consider many aspects including the main (longitudinal) reinforcement requirement, shear links, inclusion of couplers or other slab connection details (e.g. Kwikastrip / pull-out bars), bar congestion (to facilitate flow of fluid concrete / "clean" bentonite), cage assembly methodology, couplers / splices for connecting sections of reinforcement cages within individual panel). Many of these factors can be in conflict to each other and all must be considered to derive the optimal solution to achieve the design requirements together with the required level of quality.

On the diaphragm wall works undertaken by CSL there were differences in how the Design Engineer communicated reinforcement requirements. On the one hand fully detailed reinforcement cage drawings were provided which, in principal, could have been sent for fabrication with no further input from the specialist contractor. The contrasting scenario was where design intent reinforcement drawings were provided with the detailing to be undertaken by the specialist contractor.

Whilst the former situation sounds more attractive for the specialist contractor, CSL's experience is that there was generally insufficient consideration of the fabrication process (in terms of the approach preferred by CSL) for this to represent a sufficiently safe proposition. Consequently, in the cases where CSL were provided fully detailed reinforcement drawings, these were re-detailed principally to suit the fabrication processes being adopted, although there were situations where cage congestion could be alleviated through factors such as re-locating laps /splices.

The detailed design of diaphragm wall reinforcement cages should consider how the cages are to be fabricated including consideration such as whether closed or open shear links are more appropriate (Ref. ICE Temp works: Principles of design and construction, Chapter 13[6]).


Fig. 2 Historic image showing closed link diaphragm wall cage fabrication

As is illustrated in Fig 2 above, adoption of closed links makes the fabrication process significantly more onerous for the operatives involved.

An additional consideration is whether the reinforcement cages are to be fabricated off site and transported to site (which will significantly constrain the maximum length and width of the diaphragm wall cages), or if on-site fabrication is possible, which will facilitate construction of diaphragm wall panels up to 7m long, and significantly longer reinforcement cages (further reducing risks associated with splicing cages, although there may be increased lifting risks associated with tandem lifts). Both situations were encountered across the works undertaken by CSL, typically the portal structures were located on larger sites which could accommodate on-site fabrication, whereas the station boxes and access shafts were located on sites with a much smaller footprint.

BS EN 1538: 2010[7] does state minimum clearances between adjacent reinforcement bars in the vertical and horizontal planes. One area which can often appear to be overlooked however, is the effect on cage congestion through the introduction of additional reinforcing elements such as lacer bars for couplers at the location of slabs / skin (liner) walls. These can locally significantly increase the reinforcement cage congestion which potentially can have a significantly detrimental effect on the quality of the finished product, e.g. mattressing of concrete due to inadequate flow through the reinforcement (typically manifesting at the near face of the diaphragm wall panel) or inclusions within the body of the panel due to inadequate displacement of the bentonite support fluid.

<u>Rotary bored piles –“super-reinforced” cages</u>

The Bond Street Eastern ticket hall required male secant piles to be reinforced over part of their length with a double layer of longitudinal reinforcement. The detail was referred to as the “super-reinforced” section. This detail pushed the limits of CSL’s reinforcement fabricators and required careful consideration of how to effectively enable the sections of pile cage (overall pile cage lengths were up to around 40m long) to be spliced together at / over the super-reinforced section. The detail that was developed involved the use of terminator couplers at the top and bottom of the internal reinforcement cage (See Fig. 3 below). This detail negated the requirement for complicated double staggered laps at the top and bottom of these cage sections which would have been exceedingly difficult to connect and would have introduced significant additional risks to the operatives involved in the splicing process.

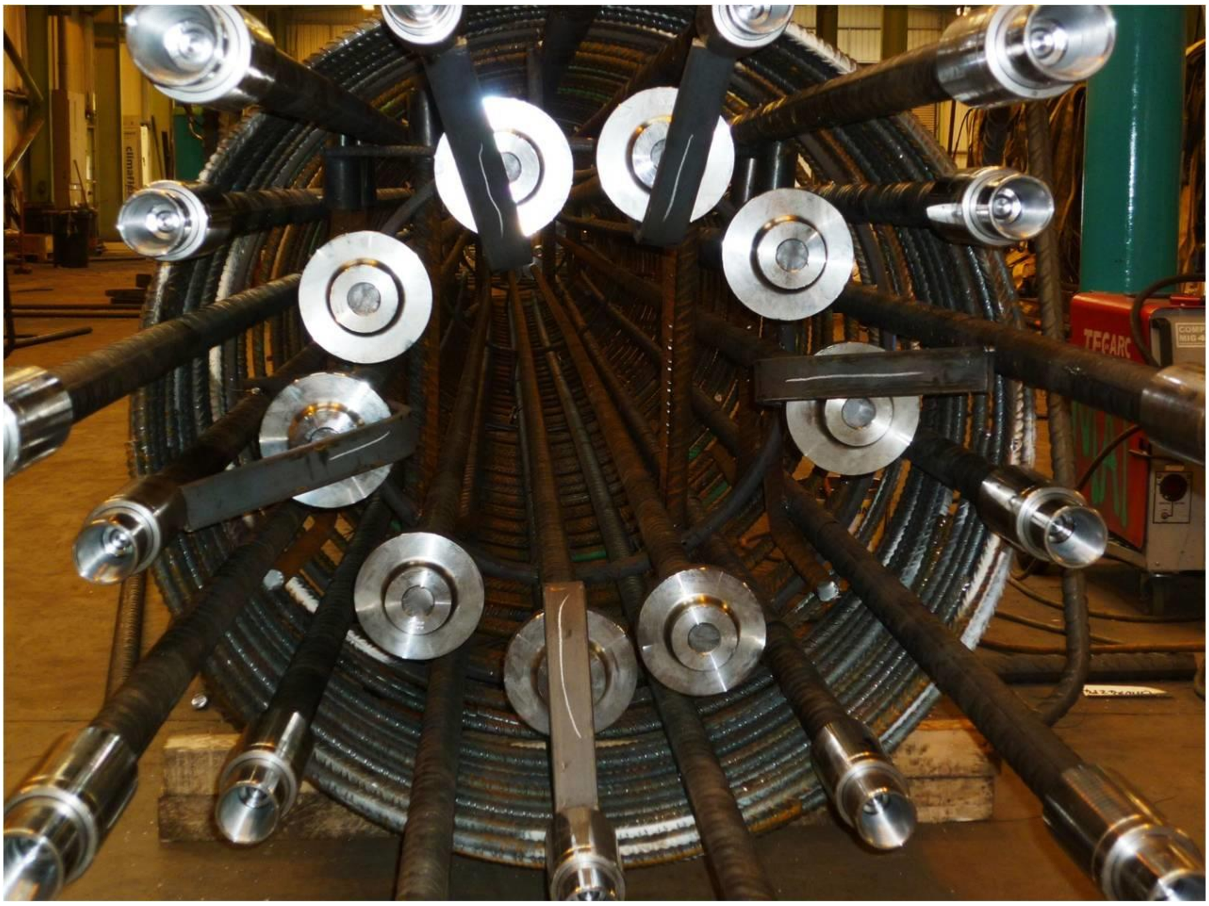


Fig 3, “Super-reinforced” secant pile reinforcement cage

An additional consideration when detailing the “super-reinforced” section of the secant pile reinforcement cage was to ensure that the risk of snagging the tremmie pipes during the concreting process was minimised. The use of terminator couplers, whilst negating the requirement for the double staggered laps, introduces a greater risk of snagging the tremmie pipes since by their very nature of providing a mechanical anchorage for the internal cage creates an effective lip which could catch on the joint of the tremmie pipes. A simple solution was adopted whereby a shaped flat steel “bar” was installed over sufficient terminator couplers to effectively guide to tremmie pipe around the terminator couplers when they were installed or extracted during concreting. This simple solution proved to be highly effective.

<u>Cased CFA piles</u>

The reinforcement cage lengths required for a cased CFA secant wall at Plumstead portal (part of the C310 contract) pushed the limits of what would be considered within the normal range. At Plumstead 18.5m were detailed which needed to be plunged full length into the fluid concrete forming the male piles immediately following the concreting process. This requirement within bearing piles would have presented a much lower risk profile for two principle reasons; 1) the 300mm cut into the female piles would create significant heat which could cause an increased rate of concrete hydration, and 2) bearing piles typically give more scope for remedial measure either through design review or, at worst case, introduction of replacement / additional piles.

There were two additional factors which further increased the risks associated with constructing the cased CFA secant piled wall, 1) slab couplers were specified to tie the base slab into the secant wall, and 2) the reinforced pile length was 18.5m. At 18.5m length the reinforcement cages would have needed to be spliced since the maximum stock length of reinforcing bar is 18.0m which when combined with the requirement for slab couplers would have increased the resistance to plunging the cages to the required level significantly.

CSL therefore requested a change to the design requirement to limit the reinforced pile length to 18.0m and to replace the cast in slab couplers with a drill and fix solution following excavation of the portal. These changes were ultimately adopted. Out of around 300 No. male cased CFA secant piles installed at the Plumstead Portal, all except two successfully had the 18m long reinforcement cages installed. The two pile which experienced issues with plunging the cages were the first two male piles to be constructed, and modifications to the concreting process, essentially to control the age and therefore stiffness of the “fresh” concrete, lead to the successful construction of all remaining male piles.

At 300mm the size of the cut into female piles required for the cased CFA secant wall at Plumstead portal was beyond the conventional norm for rotary piling, and was certainly greater than had been previously undertaken by CSL using cased CFA techniques. Using a Bauer BG40 rig fitted with Bauer’s DKS cased CFA equipment the cut was successfully achieved with a lower rate of wear on the segmental casing than had been anticipated from experience.

Integrity testing of embedded retaining wall elements

Across the contracts on which Cementation Skanska worked, there was a lack of consistency in the approach taken for integrity testing of embedded retaining walls elements, in particular diaphragm wall panels. This very much depended on the Design Engineer for each contract, rather than the particular characteristics of the contract (e.g. ground conditions or construction methodology). Extreme examples would be a diaphragm wall box to the West of London constructed almost entirely within London Clay which was required to have sonic logging tubes installed in every panel compared to a diaphragm wall portal to the East of London constructed through soft alluvial deposits, peat, river terrace gravels and chalk for which no sonic logging tubes were required (indeed no integrity testing at all was specified or required to be undertaken).

The inclusion of sonic logging tubes within reinforcement cages is potentially problematic, particularly from a safety perspective where sections of reinforcement cage need to be connected together, since operatives have to place their hands within the reinforcement cages, the upper section of which is of necessity suspended from a crane and therefore potentially subject to unplanned vertical movements. Where this risk can be eliminated it is incumbent upon all those involved in such works to ensure that this is achieved.

Alternatives to sonic logging are increasingly being adopted due to their inherent safety advantages as well as there being greater industry confidence in these alternatives. The principal alternative is a technique known generically as thermal integrity testing. This is a method whereby the heat of hydration is measured at various points within the cross section of the element (e.g. pile or diaphragm wall panel). The quality of the constructed element is assessed from the temperature across the section, with greater temperature consistency being indicative of a high quality defect free (or low defect) product. Areas of lower temperature would tend to indicate inclusions or a reduction in cross sectional geometry local to the monitoring location, whereas areas of higher temperature would indicate areas of increased cross section (e.g. significant overbreak).

Due to the method relying on the heat of hydration, there is clearly a greater time constraint when adopting this method than with sonic logging or low strain techniques, but this constraint should be easily managed though a well developed and implemented planning process. This technique also enable early results to be obtained, typically within 48 hours of pile concreting being completed.

Headwall details

The tunnel portals constructed as part of the C310 contract (Plumstead and North Woolwich) required soft panels to be constructed behind the main structural headwalls to assist the TBM launch processes, in particular to improve “watertightness” to reduce the potential for water / slurry ingress through the headwall as the TBM’s launched through the headwalls.

Cementation
SKANSKA

Fig 4, C310 Plumstead Portal / Headwall

At the Plumstead portal “soft” panels behind the headwall were constructed prior to the main structural headwall. These “soft” panels were located such that there was around 50mm overbreak into structural panel zone (ostensibly to allow the structural panel to be in intimate contact with the “soft” panel to reduce the risk of groundwater flow between the two panels). However the “soft” panel exhibited a much greater strength gain than planned (as validated by early trial mixes), which when combined with the deliberate over break / encroachment into the main structural headwall zone, caused some difficulties with maintaining correct panel (grab) alignment.

At the North Woolwich portal the “soft” panel behind the headwall were constructed after main structural headwall panels. CSL used a brush arrangement to clean the back of the main structural headwall panel to ensure there was intimate contact between the two panels. This approach worked well and with less issues than experienced at Plumstead.

The requirement or otherwise, for the construction of soft panels immediately outwith the main structural headwall will depend on a number of factors, not least, the prevailing ground conditions, the tunnelling method being adopted, and the risk management strategy being employed by the tunnelling contractor.

Soft eye details

Generally the soft eye details adopted were simply a replacement of the conventional steel reinforcement with glass fibre reinforced polymer (GFRP) with temporary works steel to facilitate lifting and placing. GFRP was adopted over the soft eye zone since this can be readily cut through by the TBM when exiting or entering the portals, boxes or shafts. This is a relatively standard process common to these types of infrastructure schemes.

However, on the Paddington station box contract the requirements were quite different. At Paddington the TBM drives were to be undertaken prior to the box being excavated and fully formed (a function of the scheme TBM drive programme in relation to the station box construction). The original concept adopted by the Design Engineer was to install a “soft” secant wall at each of the four entry / exit locations within end walls of the station box (to provide ground support to allow the tunnel eye structure to be constructed when the station box was excavated) with diaphragm walls being constructed to form the main structural box.

The “soft” secant wall was proposed to be located part over the plan position of the diaphragm wall (see Fig 5 below for the general configuration), with the secant wall being constructed prior to the diaphragm wall. The

Cementation
SKANSKA

diaphragm wall at the secant wall positions was to be of limited depth, around 16m, the box excavation being around 25m deep) such that the TBM would pass beneath the toe of the diaphragm wall panels and pass relatively unimpeded through secant wall which was to have similar strength characteristics to the in-situ London Clay. In essence the "soft" secant wall was to allow the TBM's progression to be unaffected but to provide a greater consistency of material than the in-situ London Clay may have had.

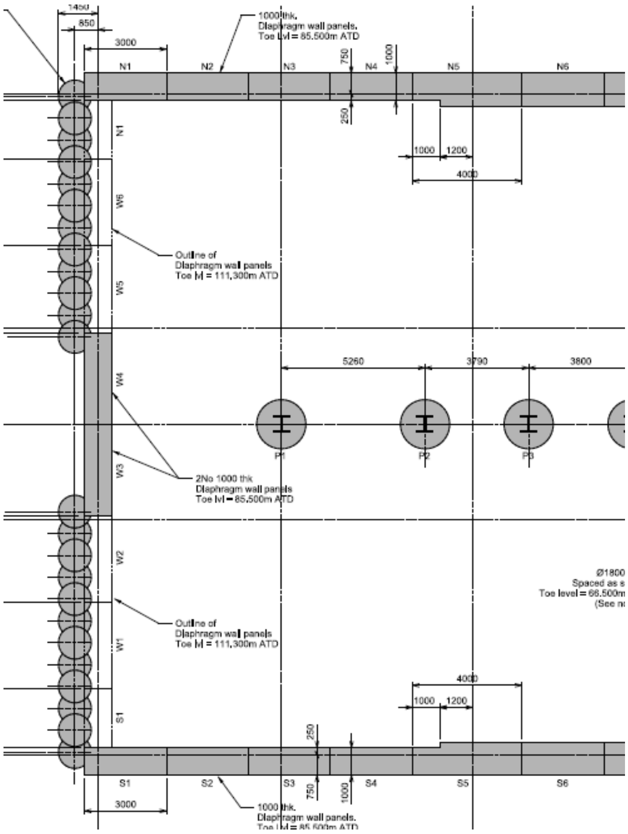


Fig 5 "Soft" secant wall / diaphragm wall arrangement

The "soft" secant wall's position relative to the immediately adjacent diaphragm wall panels was of concern to CSL since there was considered to be a high risk of the diaphragm wall panel kicking off the secant wall into the station box. An alternative solution was therefore proposed, whereby "soft" diaphragm wall panel would be installed at the four TBM entry / exit locations at the exact location where a structural diaphragm wall would subsequently be installed (again to a limited depth of around 16m). The general configuration of the "soft" panel and structural panel are indicated in Fig. 6 below with the "soft" panel section shown in orange and the structural panel shown in grey.

This alternative proposal was adopted, however only one such configuration was constructed before a significant re-sequencing of the station box construction was required. As one method of reducing the critical path elements of the station box construction, CSL proposed that the remaining three soft panel construction zones be omitted in entirety, with only the relatively shallow structural diaphragm wall being formed. Since the "soft" panels were designed to give similar properties to the in-situ London Clay, there seemed to be little benefit from installing these if a sufficiently robust risk management plan could be introduced (e.g. use of the observational approach with sufficient mitigation measures such as temporary shoring, grouting etc. being put in place). This proposal was ultimately adopted and enabled the diaphragm wall construction works to be reduced by around one - two weeks.

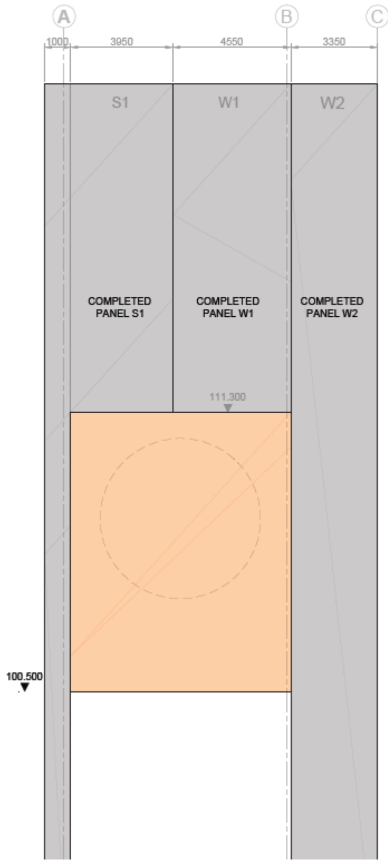


Fig. 6, “Soft” panel / structural panel headwall configuration

Where similar construction details and ground conditions apply, this methodology should be considered from the outset rather than introducing complicated and relatively onerous construction details.

Conclusions

Whilst early contractor engagement was undertaken on the Crossrail project, one area which could be improved would be to ensure that the information received through this process does not get “muddled” or misinterpreted. In CSL’s experience there are some factors which can relatively easily be taken slightly out of context and be presumed to co-exist with others, rather than being mutually exclusive. An example would be the maximum length of reinforcement which has been / can be plunged within a CFA pile in one set of ground conditions say for bearing pile, and translated to equal that achievable for cased CFA secant piles with slab couplers and inclinometer reservation tubes in a different set of ground conditions.

Additionally each specialist contractor is likely to have slightly different experience, which will inform their attitude towards risk, and also have slightly different plant capabilities. When pushing the boundaries of standard practice, these factors, together with the contractual framework, will determine what differing organisations view as being practically achievable, and possibly desirable to strive to achieve.

In terms of improving health and safety practices a significant yet simple change would be to specify alternative integrity testing techniques to sonic logging, e.g. thermal integrity profiling. For reinforcement cages in excess of 15 – 18m length this would then negate the requirement to splice reservation tubes within reinforcement cages with the associated risk of operative entrapment.

For further information on lessons learnt with regard to reinforcement cage fabrication on the Crossrail scheme, the reader is referred to other papers within the conference proceedings.

References

1 BS EN 1992-1-1: 2004; Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings. BSI London, 2014
2 BS 8110-1: 1997 Structural use of concrete – Part 1: Code of practice for design and construction. BSI London, 2002
3 BS 5400 Part 4:1990 Steel, concrete and composite bridges – Part 4: Code of practice for design of concrete bridges. BSI London, 1990
4 Jones, A.E.K and Holt, D.A; Design of laps for deformed bars in concrete under bentonite and polymer drilling fluids. The Structural Engineer pp32- 38, 21 September 2004
5 NA to BS EN 1992-1-1: 2004; UK National Annex to Eurocode 2: Design of concrete structures –Part 1-1: General rules and rules for buildings. BSI London, 2014
6 Temporary works; Principles of design and construction: ICE Publishing London, 2012
7 BS EN 1538: 2010; Execution of special geotechnical works – Diaphragm walls. BSI London, 2010

Cementation
SKANSKA

Crossrail Conference – 10th November 2015

Steel Yourself

Graeme Mitchell – Director – Southern Projects, Cementation Skanska Limited, Bristol UK

Abstract

Paddington Station, like many open cut Crossrail structures in central London, demanded stiff retaining walls to mitigate settlement of adjacent listed buildings. The challenges that this presents to designers is almost in direct conflict with the challenges faced by construction teams in relation to the ability to achieve buildable reinforcement cages and ensure concrete flow around high density reinforcement.

The designer's key requirements for footprint, geometry, cut off and toe levels combined with typical reinforcement and connection details can be summarised into a few relatively simple drawings. However, when these have to be translated into the contractor's requirements to take into account aspects including where cages can be fabricated, transport, equipment sizes, panel type sizes, tremie placement etc, this can result in vast numbers of drawings that have to be detailed, managed and constructed.

With experience of both on-site and off-site cage fabrication, this paper examines these issues and how Cementation Skanska's close work with their supply chain overcame the challenges of quality and production control for steel reinforcement cages for diaphragm walls at Paddington and other sites across the Crossrail Project. Consideration is also given to how such complex individual cage requirements can limit production flexibility and the ability to respond to any potential programme challenges or acceleration.

A significant safety risk associated with large and complex diaphragm wall reinforcement cages is the presence of loose or temporary bars (and tools) that can remain undetected and fall from the cage during lifting operations. Details are explained of how the specialist contractor and supply chain worked continuously together to try and eliminate these risks at source and effectively manage the residual risks.

Engineer’s Requirements

The Engineer’s requirements for a structure are primarily driven by those of their Client and the operational demands for whatever is being built. The Engineer has also to take into account location, environment, ground conditions and proximity to other buildings and structures. Additionally, the Engineer clearly has to take into account the overall principals of how a structure has to be built and hence loadings in the temporary condition during construction.

On the Crossrail project these structures were most obviously associated with an operational railway and comprised access/ventilation shafts, tunnels and stations. The Engineer has to consider fundamentals of the operational requirements such as the length of trains and platforms, the number and location of access points combined with the anticipated volume of passenger traffic from which the number of turnstiles can be calculated. These factors are considered to develop the overall structure dimensions and geometry required. This overall footprint has then to be considered against that of the available land-take at surface to determine likely construction techniques and structural solutions.

Design sections are developed based on ground conditions, geometry and the loadings of adjacent buildings. The sensitivity of adjacent structures to settlement is also considered and these lead to the overall selection of toe levels and cut-off levels for embedded elements. The requirements for longitudinal and shear reinforcement is developed and specified along with slab connections and any other considerations.

The sensitivity of adjacent structures and ground conditions are considered and may influence specification of panel sizes. Additionally testing and future monitoring requirements have to be taken into account. Traditionally this will require sonic logging tubes, inclinometer reservation tubes and potentially grouting tubes. There may even be requirements for geothermal pipes for ground source heating and cooling.

Another key area for consideration for the Engineer will be provision for connection for other structural elements into the vertical wall: slabs, walls, lining walls etc.

The pressures and demands on the Engineer always tend to lead towards maximising bar diameters and minimising bar spacing and therefore increasing overall reinforcement densities. This inevitably leads to complex reinforcement cages which are not conducive for the flow of concrete and efficient displacement of bentonite.

Contractor’s Considerations

Before progressing to the detailing of reinforcement cages a contractor has a different set of constraints and considerations to take into account. Key is the available site area which will fundamentally dictate whether the cages are fabricated on site or off-site and delivered to the work site. On-site fabrication requires a large working area for storage of steel, fabrication of the cages and potentially storage of completed cage sections.

Panel construction sequence also has to be considered to accommodate starter, intermediate and closer panels, and also any overall construction sequence requirements that dictate sections of wall being completed ahead of others.

Ground conditions and equipment availability can dictate choices between rope or hydraulic grabs or indeed mill selection. These will have notional excavation sizes that will impact on the individual panel sizes. This has to be balanced with any overall dimensional constraints required by the Engineer to try and rationalise and minimise the number of different panel sizes and hence cages.

The panel layout, sizes and bite sequences have to be considered against the Engineer’s requirements for corner panels, direction changes or differences in toe level. Consideration also needs to be given to the numbers and placements of tremie pipes and this is of particular importance where significant densities of steel across the thickness of the panel are required for slab connections.

Cage Detailing

The reinforcement cage detailing process has to take into account the main reinforcement diameters and spacing with possible multiple levels on each face of the panel. Along with the diameter and spacing of links, depending upon the method of fabrication, decisions have to be taken on whether to use open or closed link sets.

If panels are deep, there are restricted headroom requirements or transport requirements for off-site fabrication dictate multiple cage sections within a panel then the method of cage connection/continuity has to be determined. This can either be spliced or coupled joints or indeed a combination of both. The difficulties of joining cage sections by couplers should not be underestimated – it is both time consuming, frustrating for site teams and more significantly presents a significant safety risk as operatives have to place their hands amongst bars within a suspended load. (See Figure 1) Quick splice systems are available that reduce these risks significantly and are easier and quicker to install and join.

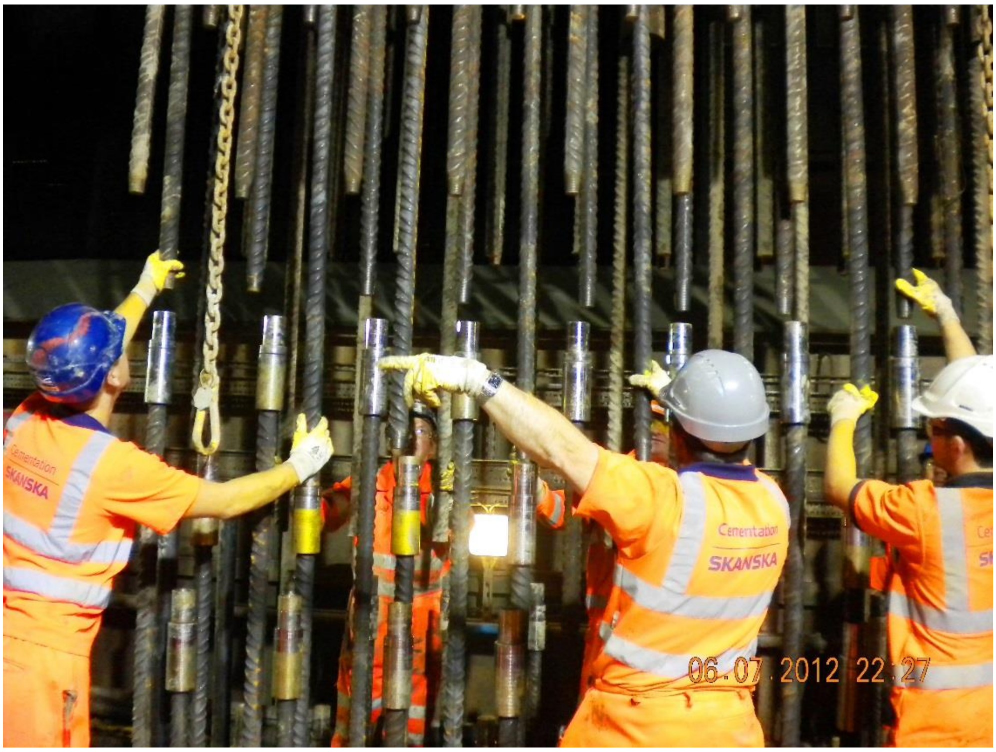


Figure 1. – Site operatives aligning couple cage section.

Particular attention needs to be paid with cage joints when there are multiple layers of main reinforcement. The choice of joint location needs to be selected carefully with respect to bar curtailment positions and also methods of allowing the inner layer of reinforcement to be joined separately from the outer developed.

The requirements for cost effective, stiff structures with minimal section thickness often leads to very high reinforcement densities with large bar diameters in multiple layers being detailed at absolute minimum spacing. It has been well documented for many years (Puller 1996) that this is a significant factor that can have a detrimental effect on the flow of concrete within the panels and hence panel integrity, waterproofness and finish.

There are also limits to what bending radii can be achieved for given bar diameters and even what former sizes are available to create these bends. Pushing these to the limits and the use of certain shapes can lead to inherent potential inaccuracies on bar lengths and positions. This is particularly relevant when considering slab connections and the positioning of couplers within box-outs. The options and benefits of U or L shaped bars with respect to concrete flow should be considered.

Box-outs for slab connections are common place but it should always be remembered that overall they remain a compromise. Their overall benefit to a project to form a structural connections between the walls and slabs without the need for post-drilling bars is clear but as outlined in the Institution of Civil Engineer's Specification for Piling and Embedded Retaining Walls 2nd Edition (ICE SPERW) they will compromise the flow of concrete and lead to potential inclusions.

As mentioned previously in relation to couplers the provision and joining of sonic logging tubes and inclinometer reservations carries a significant safety risk for site operatives. At Paddington a number of inclinometer installations within the walls were replaced with strain gauge arrays. Looking to the future technology such as thermal integrity profiling should be exploited to obviate the need for handling and joining sonic tubes on site.

Project specifications tend to apply potentially tighter construction tolerances than dictated within the ICE SPERW often in relation to vertical positioning of reinforcement. With deep panels and multiple cage sections it should be considered whether this is realistic as increasing depth and each cage joint will introduce further challenges with regard to achieving ever tighter tolerances.

Particular advantage was gained from using 3D modelling to produce cage detail drawings. This allowed close examination of cage splices and box-outs to check for clashes and buildability in the fixing and cage assembly process. See Figure 2.

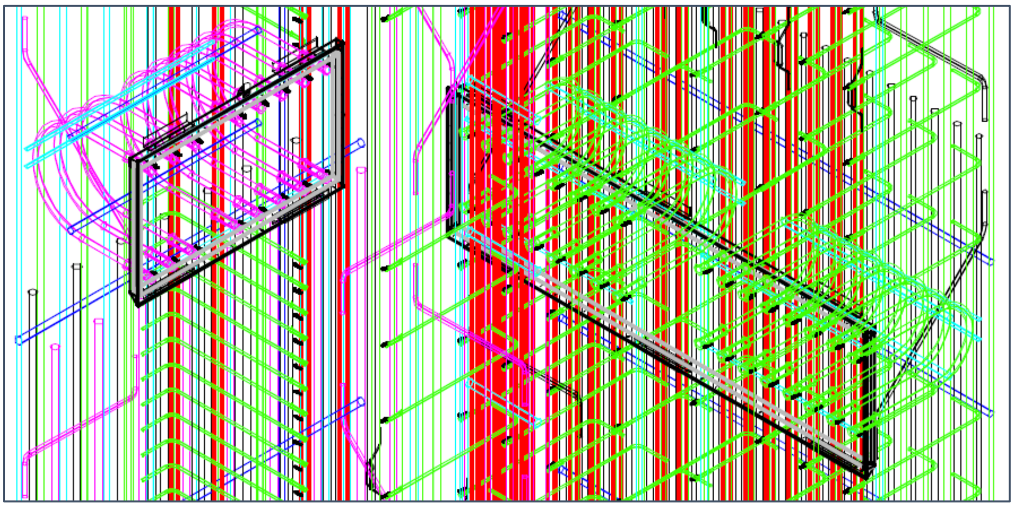


Figure 2. - Sample from 3D CAD model of cages at Paddington showing density of reinforcement and how U-bars for slab connections create a horizontal restriction to the flow of concrete within the panel.

Temporary Works

Various temporary works elements need to be introduced to the reinforcement cages by the contractor. These include longitudinal cage stiffeners to ensure unnecessary cage deflection or damage does not occur during lifting cages in the horizontal position or transitioning them to vertical prior to installation within the panel.

Lifting bands need to be designed and positioned for cage lifting and handling. Trapping bands are required to allow temporary holding of cages over the excavation whilst additional cage sections are added. Finally the top cage section will require a hanging system to allow the cages final position and level to be fixed prior to concreting.

On-Site or Off -Site Fabrication?

As on most infrastructure projects availability of space in city centre sites is sparse, hence opportunities for site fixing of cages is reduced and Crossrail was no different in this respect. Away from the city centre opportunities do exist and Cementation Skanska has experience of both off-site and remote fabrication on the project. Diaphragm walls at Paddington Station, Bond Street Station, Royal Oak Portal and Limmo Peninsular shaft had the reinforcement cages fabricated off site due to space constraints but farther east the portals at Pudding Mill Lane, Plumstead and North Woolwich all had their cages fabricated on-site.

Off-site cage fabrication often becomes a default choice due to the space constraints available at the construction site but, if undertaken by a specialist supplier, also has the advantage of cages being constructed and welded in more controlled factory like environments. To balance these advantages detailing can be more difficult and fabrication has to be much more accurate to allow cages to be joined on site. Even when cages are made in complete coupled lengths and then split the differences in forces between the horizontal factory environment and vertical installation on site means that sections rarely re-align perfectly.

Transport constraints generally limit cage section dimensions to about 18m long and 3.5m wide and there are the obvious additional handling and logistical problems with the potential for damage during transit.

On-site fabrication requires large amounts of space and can require additional lifting details and stiffening requirements but by using lifting/pitching beds increased length cage sections and indeed single piece cages can be constructed. Staggered laps can be used more readily to reduce the reliance on couplers to join cage sections. Cages are generally easier to fabricate and to install within panels.

As well as providing a suitable environment for site welding of reinforcement, an initial challenge to overcome is to gain confidence in site welding and to ensure that adequate supervision and testing is in place along with a stable workforce of qualified and tested welders.

Finally, a key advantage of being able to site fix reinforcement cages is that lines of communication from the construction team to fabrication team are much shorter and therefore improved. Both elements of the operation feel more connected and interdependent and therefore tend to work as a single team helping in regards to safety, quality, production and being able to respond to change.

Rationalisation and Programme Flexibility?

A construction project and programme are rarely completed without the potential for unforeseen problems. Re-sequencing to mitigate programme delay is a frequent possibility. Paddington Station was particularly complex with 165 panels requiring a total of approximately 350 individual cage section drawings that were combined into approximately 60 different arrangements for individual panel types. This degree of individuality to cage types makes for a complex detailing process which then follows for much less flexibility in the construction process on site once committed to a sequence of cage fabrication.

Much thought should be given to minimising the loading cases and minimising and rationalising slab thickness, level changes and voids. Simplification at the design stage can reduce the chances of errors in construction but also liberate flexibility for change as the project progresses. Due thought should also be given to whether additional storage capacity is required to accommodate fabricated cage sections to again give greater flexibility, remove transport delays and ensure that the fabrication facility does not become log jammed.

Health and Safety Considerations

As the Paddington project got underway there were a number of near misses relating to loose bars being found within cages. The presence of tools or temporary bars accidentally being left in cages is not new and can be difficult to manage – particularly with respect to cages pre-fabricated off-site. The risk that these present is that they lie concealed and then fall from the cage when it is pitched from horizontal to the vertical prior to installation in the ground.

Working closely with our specialist supply chain, Cementation Skanska introduced a series of measures that significantly reduced the incidences of loose bars in the cages. (Figure 3)

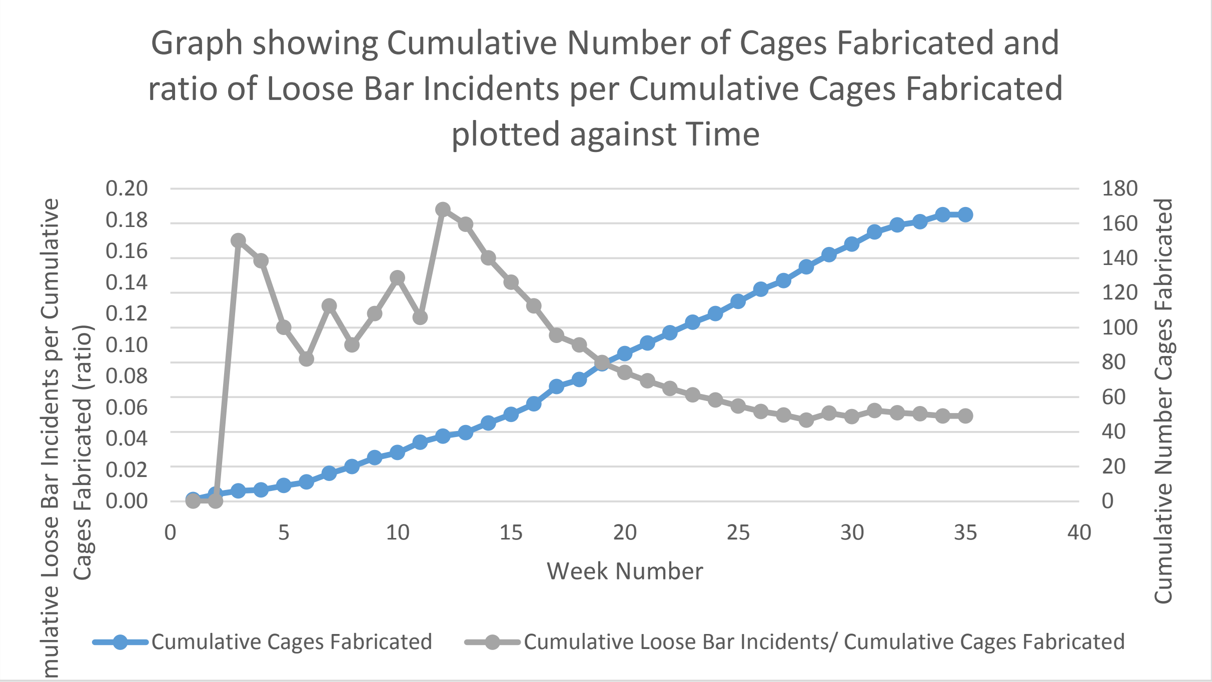


Figure 3. – Graph showing Cumulative Number of Cages Fabricated and Ratio of Loose Bar Incidents per Cumulative Number of Cages Fabricated.

Tools and temporary bars were coloured to make them stand-out from the cages, floors were painted white and lighting improved within the facility to aid checking (Figure 4). The checking regime was improved both in the factory and at site with defined individuals being responsible for signing off cages prior to dispatch and lifting on site.

Figure 4. - Coloured fabrication tools to aid detection during exit checks.

One of the keys to success was improving communications with the fabrication facilities and giving them a better understanding of what happens with the cages once they leave their facilities. Videos of lifting operations were prepared and members of our site team travelled to the fabrication facilities to show how cages were tandem lifted and pitched from a horizontal to vertical orientation. This proved to be a surprise to many of the steel fixers. Site visits were also arranged for key members of the fabrication team.

Conclusions

Extensive use of the diaphragm walling technique in the UK tends to follow with large infrastructure projects (Jubilee Line Extension, Channel Tunnel Rail Link, Crossrail). Shortly after the Jubilee line in the mid-nineties Malcolm Puller wrote about risks of poor concrete as a result of high reinforcement densities. This risk remains and by specifying large bars at minimum spacing these risks should be understood from the outset. Box outs will rarely be perfectly formed due to the restrictions to concrete flow. Indeed, the concrete properties required to maximise flow around bars lead to a higher risk of concrete mix instability and bleed.

Where space is available site fixing reinforcement has many advantages over off-site fabrication. Consideration should be given to rationalising designs to minimise the different numbers of cage types and arrangements. This reduces the chances of mistakes during construction and creates more flexibility for resequencing works.

Safety of the site teams remains paramount. Highly reinforced and coupled cage sections increase the risks of trapping injuries to the site team. Additional measures and checks need to be introduced to minimise the risk of undetected loose bars falling from cages during lifting.

For further details relating to detailing of reinforcement cages for embedded retaining walls the reader is referred to the 'Specification & standard implementation across Crossrail contracts' paper also published in these proceedings.

References

Puller M, Deep Excavations: a practical manual, Thomas Telford First Edition 1996.

Institution of Civil Engineers, Specification for Piling and Embedded Retaining Walls, Thomas Telford, 2nd Edition 2007.

Application of the Observational Method on Crossrail projects

Ying Chen, Duncan Nicholson, Peter Ingram, Stuart Hardy, Hock Liong Liew, Imran Farooq, Giovanna Biscontin

Abstract

This paper describes the use of the Observational Method (OM) on three Crossrail station excavations. Firstly, at Tottenham Court Road, Western Ticket Hall excavation where a code compliant design was started. By the third excavation stage, it was realised that movements were less than predicted and an OM was introduced. This enabled the lowest level of strutting to be eliminated, leading to savings in both programme and costs. Secondly, at Canary Wharf Crossrail Station, an OM approach was envisaged before the start of construction, but was only designed during the final excavation stage when time was available for back-analysis. A saving in costs and programme was achieved. Thirdly, at Moorgate shaft (part of Liverpool St Station) where construction commenced with a standard code-compliant design. A client-led redesign was carried out after installation of the diaphragm wall with an innovative verification process using a proactive OM approach to enable programme savings. When unusual deflection was observed from the shaft east wall due to structural uncertainties and construction activities, the OM and verification process provided a flexible framework within which to reassess ongoing movements and effects on an adjacent tunnel to ensure construction could proceed safely.

The case histories described demonstrate the benefit of adopting the OM for excavations in order to achieve savings in time and cost, and to react to unexpected movements during construction. These case histories fit well within a new framework for OM applications developed as part of the CIRIA C760.

1. Introduction

The Observational Method (OM) was first successfully applied in geotechnical engineering by Terzaghi and had been formulated and developed as the 'learn-as-you-go' method (Terzaghi & Peck 1967). In the 9th Rakine lecture, a more formalised methodology and the name 'Observational Method' were introduced by Peck, who provided a distinct and novel design approach to manage risk (Peck 1969). The potential for savings of time and cost by applying the OM on engineering projects without compromising safety has been widely reported, such as the Channel Tunnel Rail Link (Young & Ho, 1994). The OM has been adopted and used in several countries, for example the UK with CIRIA report 185 (Nicholson et al. 1999), France with the Irex-RGCU guidelines by Allagnat (2005) 'La Methode Observationnelle pour le dimensionnement interactif des ouvrages' (2005), German Standard DIN 1054:2005-I Verification of the safety of earthworks and foundations and Dutch CUR document 166: sheet pile construction. In Eurocode EC7 (EN1997-2004), the OM is listed as one of the acceptable design methods.

The research on which this paper is based has been carried out as part of 'Crossrail lessons learnt' for the Crossrail project. The purpose of the paper is to use the Crossrail case histories to demonstrate a newly developed framework for the OM approach applied to deep excavations. The new framework is part of the CIRIA C760 (revision to CIRIA C580) due for completion in 2016 (Gaba et al.).

2. A new framework for the OM approach application to deep excavation

Historically, Peck, (1969) defined two situations where the use of the OM is appropriate:

- *"Ab initio"* approach – OM adopted from inception of the project. Peck recommended adopting most probable conditions and then introducing contingencies when required. This is an "optimistic" application of OM.
- *"Best-way-out"* approach – OM adopted after the project has commenced and some unexpected event has occurred or whenever a failure or accident threatens or has already taken place, the OM may offer the only satisfactory way out of the difficulties. This is a "reactive" use of the OM to an adverse event.

Since Peck's Rakine lecture, further study on the implementation of the OM has been carried out. The OM was defined in detail in CIRIA 185 (1999) by Nicholson et al. This concentrated on the *Ab initio* approach. It recommended that a characteristic initial design, consistent with current codes should be carried out initially. A design would then be made using most probable parameters. If the monitoring data showed the characteristic design to be conservative then the design would be progressively modified to the most probable design. This is the "cautious" Ab initio approach.

At Crossrail, the OM was introduced into excavations after construction was started using conventional designs. The initial review of construction records showed that a saving in time and costs could be made. In this case the term "Best way out" is not appropriate because a "pro-active" rather than "reactive" use of the OM is being proposed. A more general term "Ipso tempore" – at the moment is suggested by the CIRIA C760.

Based on the discussions and case histories of the OM, a framework for the OM approach has been developed for application of the OM to deep excavations as shown in Table 1. A detailed discussion on this framework will be presented in a future paper which is under preparation (Chen et al. 2016).

Approach of OM		**Ab initio (from the start)**		**Ipso tempore (in the moment)**	
		A (Peck's 69) Most Probable	**B (Ciria C185) Characteristic**	**C (Not defined) Modification**	**D (Peck's 69) Contingency**
OM design work		Before construction		During construction	
Design	Existing back analysis	Needed	Not needed	Using initial construction stages	
	Start OM design with parameters [a]	Most Probable	Characteristic	Back-analysis of existing predefined design/construction [b]	
	Design objectives	Reduce wall size and depth	Reduces support	Reduces support	Increase support
	Alternative design [c]	Use Contingency plan	Use Modification plan	Develop Contingency plan	Develop Modification plan
	During construction [d]	Progressive modification of design			
Construction programme		Planned for OM		Planned when OM is applied	
Risk Management	Red trigger based on	Most Probable design [e]	Characteristic design (SLS)	Recalibrated Characteristic design (SLS)	
	Amber trigger based on	Proportion to RED	Most Probable design	Propotion to recalibrated design	
	Triggers modificaiton [f]	Possible		Recalibrated	
	> Amber	Contingency option			Contingency option [g]
	< Amber	Modification option			NA
	Risk level [h]	Medium	Low	Medium	High
I & M	Starting plan	Extensive	Standard	Standard	
	Starting frequency [i]	High	Medium	Medium	
	Additional Instruments	Not needed	Needed	Needed when OM is applied	
	Increased frequency	Not needed	Needed	Needed when OM is applied	
Value engineering	Saving in Material	Maximum	Potential	Possible	Assurance of acceptable safety
	Saving in Programme	Possible	Maximum	Potential	
	requirement	Time and cost to use of OM ≥ the benefits achieved			

Notes:
- *a) Characteristic design using characteristic values as defined in EC7.*
- *b) For Ipso tempore approaches, the start of OM design using parameters calibrated from back-analysis of predefined design (using characteristic parameters) in initial excavation stages.*
- *c) Contingency plan is a design adopting characteristic parameters, modification plan is a design adopting most probable parameters.*
- *d) Progressive modification is applied when more data are available to back-analyse and feedback to the design.*
- *e) In Ab initio-A approach, the red trigger is used to decide if the alternative design will be adopted for the next construction stage. After switch to alternative design, the red trigger values should be redefined by characteristic design (SLS).*
- *f) Trigger values will depend on the 'discovery-recovery' contingency plans being used and not simply taken from the calculated values.*
- *g) For Ipso tempore-D approach, to prevent a SLS or ULS failure occurring, contingency or emergency plan will be required.*
- *h) Risk level for each OM approach is indicative based on a comparison between OM approaches.*
- *i) Frequency recommendation: high=hourly to daily; medium = daily to weekly*
- *j) Contractual and communication concerns are excluded in this OM approach frame table.*

Table 1 Summary of the Ab initio and Ipso tempore approaches for applications of the Observational Method

3. Crossrail case histories

The OM has been successfully applied to a number of Crossrail deep excavations and the project is therefore an ideal example to demonstrate the application of the OM approaches defined in Table 1. Examples have been listed which demonstrate the four methods of each of the OM approaches as described in the C760. Of these methods, two are demonstrated using case histories from previous projects, Batheaston-Swainswick Bypass (Nicholson et al. 1998) and a deep basement excavation with top-down method (Ikuta et al., 1999). One example is from an under construction project, the Victoria & Albert museum excavation. Three case histories from the Crossrail project are demonstrated using the new defined Ipso tempore-C (modification) OM approach. The following sections of this paper describe the relevant case Crossrail histories. These are summarised briefly below:

- Ab initio-A (most probable): Batheaston Swainswick Bypass excavation (Not discussed in this paper)
- Ab initio-B(characteristic): Top-down deep basement excavation (Not discussed in this paper)
- Ipso tempore-C (modification):
  Tottenham Court Road (TCR) station excavation, Western Ticket Hall (WTH)
  Canary Wharf Crossrail Station (CWCS) excavation, head walls
  Liverpool Street station Moorgate Shaft (LIS-MS) excavation
- Ipso tempore-D (contingency): Victoria & Albert museum excavation (Not discussed in this paper)

Back-analyses of the case histories in this paper have been carried out by using *Oasys* FREW version 19.2 (2D), unless otherwise stated. The maximum predicted wall deflection and the measured wall deflection were compared at key excavation stages, demonstrating the benefits of the OM approach for each case history.

3.1 Tottenham Court Road Western Ticket Hall

Introduction

The excavation of the Western Ticket Hall (WTH) box at Tottenham Court Road (TCR) station of the Crossrail railway project in London was undertaken using bottom-up construction to minimise the duration of excavation, see Figure 1. This decision was taken to facilitate the earliest possible construction of the base slab and preparatory sprayed concrete lining (SCL) tunnelling works in advance of the arrival of the twin tunnel boring machines (TBMs).

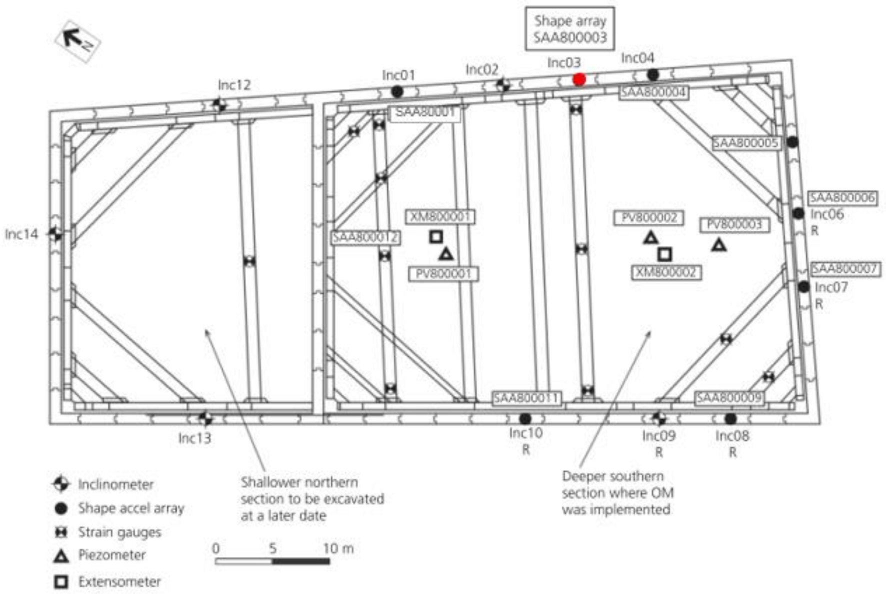


Figure 1 TCR-WTH instrument layout plan (Yeow et al. 2014)

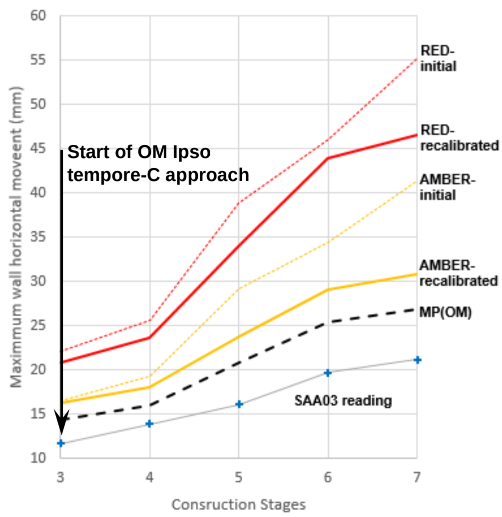


Figure 2 Triggers for TCR-WTH diaphragm wall

OM application

Before construction started the Contractor carried out a Value Engineering study which identified an opportunity for the application of the OM. This led to increased instrumentation to provide more comprehensive real time monitoring. The possibility of modifying the conforming design at early stages in the construction sequence was identified by using the OM approach. Collaboration was required between the client, contractor and designers.

The OM was used to eliminate the lowest level of temporary propping originally designed to support the retaining walls. This was after the excavation work started and was aimed to reduce the support, therefore this case can be categorised as an Ipso tempore-C (modification) OM approach - a “proactive” approach as listed in Table 1.

Stage	**Description**	**Start***	**End***
1	Excavate to +121.6mTD	09/05/2012	16/05/2012
2	Installation P1 prop & excavate to +116.4mTD	20/06/2012	27/06/2012
3	Installation P2 prop & excavate to +111.1mTD	18/07/2012	25/07/2012
4	Installation P3 prop & excavate to +108.3mTD	08/08/2012	15/08/2012
5	Installation P4 prop & excavate to +101.2mTD	05/09/2012	-
6	*Installation P5 prop** & excavate to +96.8mTD*	*21/09/2012*	-
	(Local trench excavation to +95.4mTD)	26/09/2012	26/09/2012
7	Construct the base slab	29/09/2012	-

**start and end dates are indicative dates for the excavation only*
***prop P5 had been eliminated in the actual construction sequence*

Table 2 Summary of construction sequence to casting of base slab (after Yeow et al. 2014)

Evidence to support the implementation of the OM became available at the end of stage 3 of construction, where the measured maximum wall deflection was 13mm compared with the predefined design deflection of 23mm, (see Figure 3). The stage 3 wall movements were back-analysed using most probable parameters assessed from a review of the site investigation data and the as-built construction details. The most probable predicted deflections for stage 3 excavation are shown in Figure 3, and it can be seen that they compare well with the measured deflections. The model was then run for the remaining excavation stages and the final predicted movements with no lowest level of temporary propping as shown in Figure 4, together with the predefined design predicted movements with the temporary props.

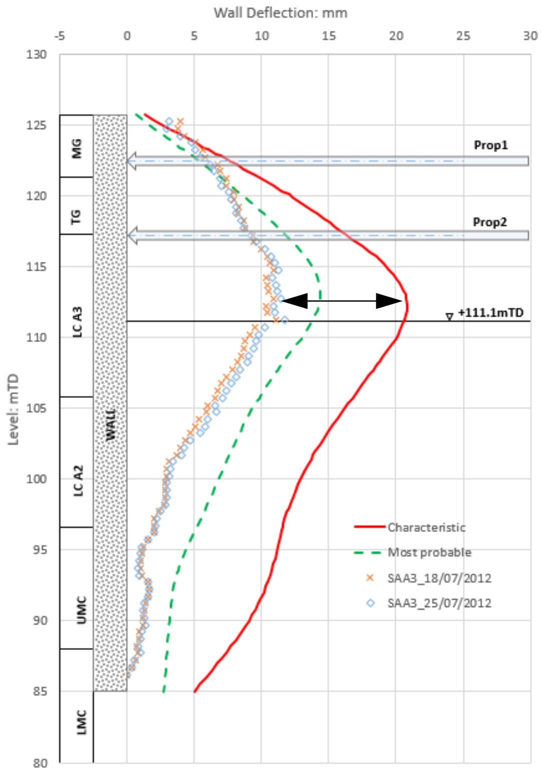


Figure 3 Greatest deflection profile at end of stage 3

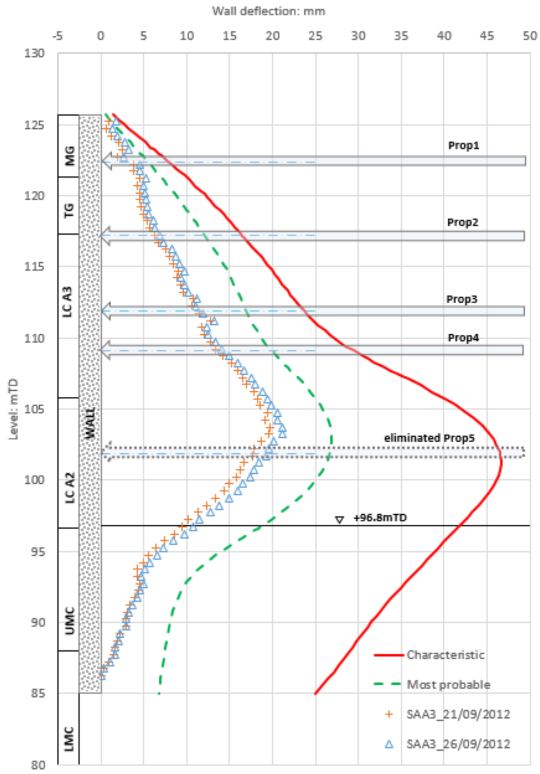


Figure 4 Revised construction sequence eliminating prop P5, wall deflection profile at end of excavation stage

Contingency measures were designed and an OM implementation strategy was set up to closely monitor and verify performance of the works. The recalibrated trigger values for subsequent stages after the back-analysis at stage 3 are shown in Figure 2 together with the measured data.

Benefits of the OM approach

The Ipso tempore-C (modification) OM approach was implemented during construction of Crossrail TCR station WTH box. This enabled the elimination of the P5 level temporary prop, leading to a critical path programme saving of 4 weeks and a £715,000 reduction in cost. (Yeow, et al. 2014)

3.2 Canary Wharf Crossrail station head walls

Introduction

Canary Wharf Crossrail station (CWCS) was constructed within the West India North dock covering an area of approximately 260m × 25m to 30m. The conforming design adopted a complex arrangement of Giken tubular piles with RC infill piles tied back to anchor piles behind the retaining wall forming a temporary cofferdam. In areas where installation of anchor piles was not possible, a cantilever retaining wall and berm were adopted. Details of the cofferdam wall design can be found in Yeow et al. (2012) and Travers & Yeow (2013).

OM application

During construction, a comprehensive array of monitoring instruments was used to capture deflections of the walls, piles, movements of the capping beams and tie forces. Those instruments were carefully read to establish baselines. At the same time, the excavation was carefully recorded. An Ipso tempore-C (modification) OM approach was applied: the design parameters were back-analysed using the monitoring data from early stages of excavation. Prior to excavating to the final formation level – excavation to -4 slab, a set of most probable parameters was derived to review the subsequent construction sequence for headwalls. This led to the removal of the lower-level berm and inclined props for the excavation to the final base-slab level. The estimated associated cost saving was approximately £500,000 in addition to substantial programme savings.

Benefits of the OM approach

This Ipso-tempore-C (modification) OM approach in the case of the CWCS head walls is a good example of using the new framework for OM approach in excavations. Based on the initial OM approach by Peck (1969), it is arguable whether this case history could be an "*Ab initio*" or "Best-way-out" method application. The concept of OM was considered at post-tender design development prior to commencing the construction work. However, a fast construction design for CWCS did not allow a full scale OM design to be developed prior to construction. The interactive involvement of client, designer and contractor, together with well-established baseline readings for the monitoring instruments and a clear excavation record provided an opportunity for OM implementation at later excavation stages. This proactive use of the OM has proven to be a success.

3.3 Liverpool Street station - Moorgate Shaft

Introduction

The irregular shaped Moorgate Shaft (MS) has been constructed at Liverpool Street Station (LIS). It is about 35m×35m in plan and extends approximately 42m below street level, see Figure 5 & Figure 6. Shaft construction was undertaken using the top-down construction technique by a 1.2m thick diaphragm wall with 7 levels of reinforced concrete ring beams and 2 levels of temporary steel props in the initial design. The north-south section differs from the east-west section which is supported by two additional temporary cross walls (1.2m thick unreinforced concrete panels) and a pair of slab strips (3.0m wide by 1.5m deep) in the initial design, to minimise wall deformation and protect nearby assets rather than for a ULS design requirement.

The removal of existing piles at the MS site had delayed the shaft construction programme by 11 months. To mitigate the delay, a reanalysis of the shaft has been carried out using FLAC 3D based on non-linear soil parameters and utilising a granted concession against Crossrail Engineering Design Standards (CEDS) to allow use of undrained conditions on the active side for fine grained soils. The reanalysis confirmed the lowest temporary prop at +76.0mTD could be omitted. Further value engineering review based on the reanalysis, identified further potential programme-saving measures: i) omission of temporary propping at +80.5mTD; ii) combining excavation stages for ring beams 4 and 5; iii) combing excavation stages for ring beams 6 and 7; iv) omission of the slab strips. The reanalysis of the shaft was discussed in detail by Farooq et al., (2015).

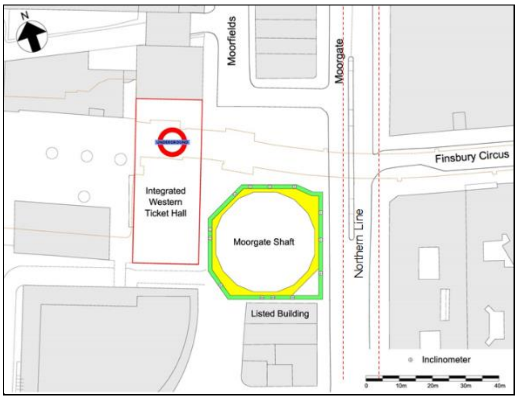


Figure 5 LIS-MS site plan (Farooq et al., 2015)

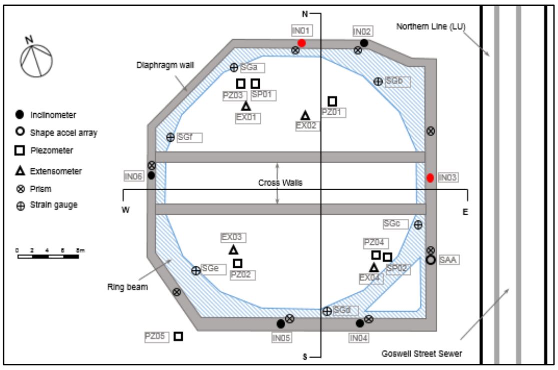


Figure 6 Indicative instrument layout plan for LIS-MS

OM application

The OM was used as part of a defined verification process which was a variation of progressive modification methodology (as defined in Powderham, 1998). The OM was agreed to be used at three selected Verification Points (VP) during excavation, to provide explicit safety assurance and reassure external stakeholders that project risks would be minimised. The innovative verification process (Farooq et al., 2015) in the MS confirmed those potential programme-saving measures at the VPs .

The redesign was carried out after the diaphragm walling and the first two excavation stages were completed, therefore, the MS case history is classified as Ipso tempore. Given the bespoke application of the OM approach detailed in Farooq et al., (2015), the approach here was not strictly

Ipso tempore-C or D approach. However, as the overall proposed scheme was a proactive progressive modification where changes were only positive to the construction cost and programme, it can therefore be classified as Ipso tempore-C (modification). It could be argued that this application of the OM could also be considered as the Ab initio-B (characteristic) OM approach, as a full scale of preliminary 'most probable' redesign was completed prior to the potential measurements commencing in the field (although most probable rather than characteristic parameters were used in the initial re-design). This example serves to demonstrate the fluid nature of the boundaries between the different methods.

In the verification process, the pro-actively and progressively modified analyses used the state-of-the-art A* soil model as outlined by O'Brien et al., (2011 & 2013) in response to observed behaviour. This was a fluid real time assessment and the designer had to consider not only uncertainties in ground behaviour, but also the anticipated structural behaviour (such as the irregular shape of the shaft and ring beams as well as the efficiency of the cross walls) together with the uncertainties associated with adjacent concurrent construction activities (for example permeation grouting and SCL tunnelling work). The implementation of the process was discussed in detail by Farooq et al., (2015) and therefore will not be discussed further in this paper.

Stage	**Description**	**Start**[1]	**End**[1]
1	Installation capping beam & Excavate to +106.5mTD	24/10/2013	25/11/2013
2	Installation RB1 & excavate to +100.3mTD	16/12/2013	11/01/2014
3	Installation RB2 & excavate to +94.4mTD	17/03/2014	09/04/2014
4	Installation RB3 & excavate to +89.7mTD		
5	Installation RB4 & excavate to +87.6mTD	-	-
5* (Alter.)[2]	*Installation RB3 & excavate to +87.6mTD*	*07/05/2014*	*17/05/2014*
6	Installation RB5 & excavate to +85.4mTD	-	-
7	Installation RB6 & excavate to +83.5mTD	-	-
7* (Alter.)[2]	*Installation RB4, RB5 & excavate to +83.5mTD*	*28/06/2014*	*02/07/2014*
8	Installation RB7, slab strips[3] & excavate to +79.1mTD	-	-
8* (Alter.)[2]	*Installation RB6, RB7*	*-*	*-*
9	Installation T2[3] & excavate to +71.3mTD	-	-
9* (Alter.)[2]	*Excavate to +71.25mTD*	*22/08/2014*	*02/09/2014*
10	Construct the base slab		

1 start and end dates are indicative dates for the excavation stage only in line with the actual construction sequence.
2 the alternative construction sequence (italic) confirmed by the OM during verification process.
3 the slab strips and the temporary prop T2 at +80.5mTD were eliminated in the actual construction sequence.
4 o to 600mm further dig have been included in the actual excavation to cast blinding layer and construction purpose.
5 the detailed construction sequence refers to Farooq et al., 2015.

Table 3 Summary of construction sequence for the Moorgate Shaft – up to casting base slab stage

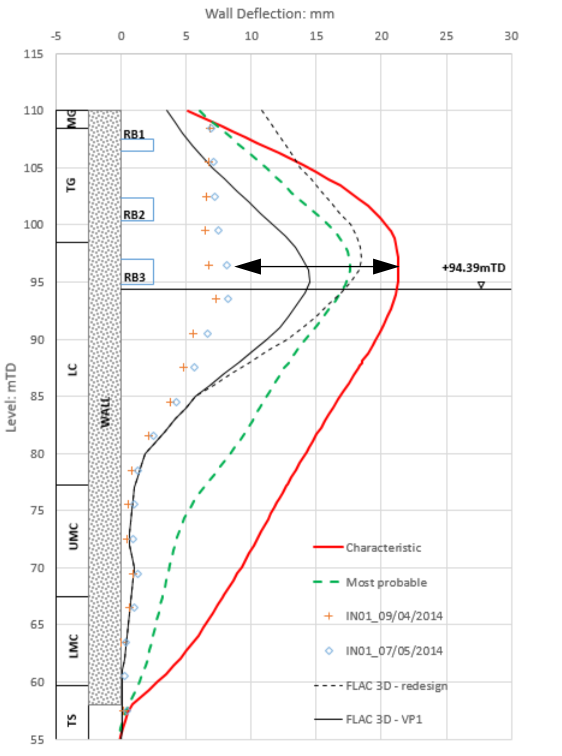


Figure 7 North wall deflection profile at excavation level of +94.4mTD

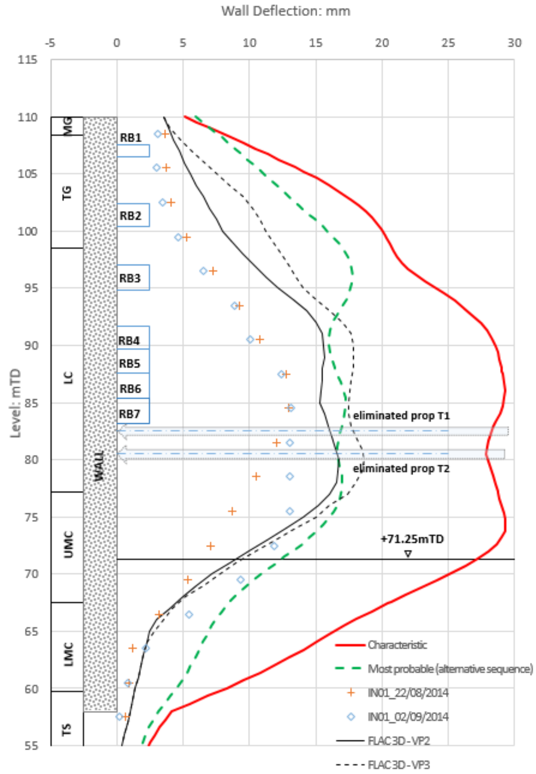


Figure 8 North wall deflection profile at final excavation level of +71.3mTD

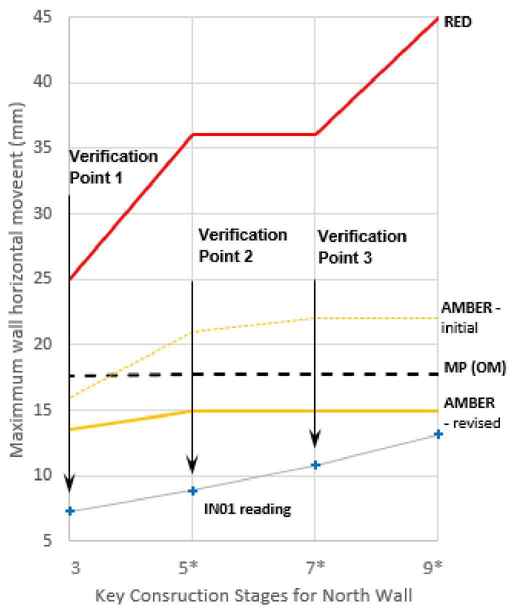


Figure 9 Triggers for LIS-MS north wall

The north wall as an example was selected to carry out a 2D back-analysis for this paper using FREW representing the north-south section and demonstrating the Ipso tempore-C (modification) OM application, which has considered the site constraints to the other walls as shown in Figure 5: the Northern Line tunnel less than 5m from the east wall, listed buildings immediate to the south wall and the existing Moorgate station ticket hall to the west. It should be noted that the verification process was based on using FLAC 3D model which had covered the above site constraints. The FREW analysis is included in this paper to demonstrate a 2D set of results for comparison purposes. A summary of key excavation stages and the alternative construction suggested by the redesign and confirmed by the OM at VPs is presented in Table 3. The 2D FREW back-analysis model has referenced to Table 3 for construction sequence.

Monitoring data was reviewed at three VPs corresponding to three excavation levels (at 94.19mTD, 87.35mTD and 82.9mTD) and wall movement projections produced using FLAC 3D and assessed to the bottom of the excavation at each VP stage to ensure the alternative construction sequence was applicable. Figure 8 shows the wall horizontal deflections at final excavation (stage 9) with a formation level of +71.3mTD.

A good match is seen between FLAC and measured results and also in the lower part of the wall between the measurements and the predictions using most probable parameters in FREW. However, this simplified 2D FREW back-analysis model is not able to capture the 3D effects from the MS geometry modelled by the FLAC 3D model referred to Farooq, et al., (2015). The north wall deflection profiles from the FLAC 3D model were shown in the figures, which were the actual prediction used to enact the Ipso tempore-C (modification) OM approach in the MS case history.

The triggers against the measured movement from key excavation stages are shown in Figure 9.

East Wall

For the MS east wall, the wall movements were initially minimised by installing two cross walls, see Figure 6, to control the movements of the adjacent Northern Line tunnels within the trigger limits. During the verification process at LIS-MS, it was noted that the east wall projected movements could lead to a potential amber trigger level breach when the excavation reached the bottom at 71.3mTD. This projection was due to uncertainty in modelling the east wall: the irregular shape of the shaft, inefficient ring beam geometry, concurrent adjacent grouting works on the east side, and difficulty in accurately predicting east-west cross wall stiffness/behaviour. To capture the uncertainties listed above a bounded approach for both soil and structural elements (such as ring beams, cross walls and diaphragm wall) was adopted for the analysis. The potential breach of the east wall amber trigger (due to lower bound analysis) and its consequent effects on the northern line tunnel were reassessed and accepted by the LUL (although eventually the original trigger levels were never breached during construction).

As the verification process progressed the analysis was refined until all completed construction stages could be replicated. Although the observed structural behaviour was at the lower end of the bounded parameters (i.e. pessimistic) in the model, the soil behaviour was at the upper end of the bounded parameters. The three phase verification process allowed the OM to be applied to modify the 3D model and each stage was fully assured by Cat III checker, with a best way out option available at each stage if unfavourable results were generated.

Given the unusual structural behaviour, particularly on the east wall, further contingency measures or best way out options were discussed and considered with the client during the verification process, such as suspension of prop omission, or less onerous propping. However, those measures were not needed, as by the end of the verification process there was a high level of confidence that the projected movements would stay within accepted trigger limits. Within the LIS-MS excavation, due to the speciality of the east wall, it is considered that the Ipso tempore-D (contingency) OM approach classification could have been applied for the MScase history if the best way out options/contingency had been implemented. The approach used at MS ensured that the safety requirements of nearby structures during the shaft excavation and the construction were understood at all stages of excavation.

Benefits of the OM approach

The verification process with the OM design approach was successfully implemented for construction of Crossrail LIS - MS. This enabled the construction programme to be accelerated by the elimination of the temporary prop at +80.5mTD and slab strips as well as the combination of excavation stages and overcoming the potential 11month delay. The OM application had not only made the construction faster but also safer by omitting the temporary propping, and reducing the number of construction stages. An additional benefit was that all parties had to work collaboratively throughout the process which facilitated communications and improved understanding of risks and opportunities.

4. Summary

The successful use of the OM in the Crossrail excavation case histories presented, shows that the OM can be used in the design of excavations to achieve savings in both time and costs. The OM approach was adopted at TCR-WTH after major excavation, leading to 4 weeks saving in programme and £715,000 in cost. It is estimated that if the OM approach had been adopted as “Ab initio”, the potential additional saving in material costs would have been between £350,000 and £500,000 under an associated design fee of approximately £50,000. The implementation of the OM requires full confidence in the design, construction approach and the active collaboration of all involved parties. The speed of the OM design is another key concern in the OM application. The OM case history of the CWCS head walls gives an example of the flexibility of the OM in adapting to the situation. The OM implementation at LIS-MS construction managed to achieve a construction programme saving. It also serves to demonstrate the fluid nature of the boundaries between the different OM methods. At the same time, it enabled the reassessment of unusual wall deflections of the east wall to ensure the safety requirements of a third party asset were satisfied so that the construction suspension period was minimised.

These Crossrail excavation case histories show that the revised framework for the OM approach from the C760 is applicable to excavation works. Potentially, this framework can also be applied to extended geotechnical engineering design: tunnel excavation, deep foundations and embankments, which will need additional support from the OM case histories for these applications.

Acknowledgements

The authors would like to thank Tony O'Brien from Mott MacDonald, and Mike Black and Peter Cracknell from Crossrail Ltd for their kind help on case histories and permission to publish the data presented. The authors would also like to thank their colleagues at Arup and Professor Kenichi Soga from the University of Cambridge for their contribution during the case histories study and compilation of this paper.

References

Terzaghi K. & Peck RB (1967) *Soil mechanics in geotechnical engineering Practice, 2nd Edition* John Wiley & Sons, New York

Peck RB (1969) *Advantages and limitations of the observational method in applied soil mechanics* 9th Rankine Lecture, Geotechnique 19 No.2 pp171-187 ICE, London

Ikuta Y. et. al. (1994) *Application of the observational method to a deep basement excavated using the top-down method*, Geotechnique 44 No.4. pp655-679 ICE, London

Powderham AJ & Nicholson DP (1996) *The observational method in geotechnical engineering* ICE, Thomas Telford, London

Nicholson DP et al. (1998) *Value achieved using the observational method on the retained cutting at the Batheaston/Swainwick bypass* The value of Geotechnics in Construction, pp123-146, Proceedings from the seminar ICE, London

Powderham AJ (1998) *The observational method – application through progressive modification Proceedings of Journal ASCE/BSCE Vol.13 No. 2 pp87-110*

Nicholson DP, Tse C & Penny C (1999) *The observational method in ground engineering: principles and applications* Construction Industry Research and Information Association (CIRIA) Report 185, London

Gaba et al. (2003) *Embedded Retaining Walls Guidance for Economic Design*, Construction Industry Research and Information Association (CIRIA), London, UK, Ciria C580

Eadington & O'Brien (2011) *Stiffness parameters for a Deep Tunnel –Developing a robust parameter selection framework* Proceeding of the 15th ECSMGE, Athens.

Yeow HC et al (2012) *Achieving more for less at Canary Wharf Crossrail station, London* Civil Engineering Special Issue 165, Issue CE5, pp 50-57

O'Brien & Harris (2013) *Geotechnical Characterisation, Recent Developments and Applications* 12th International Conference on Underground Construction, Prague.

Travers R & Yeow HC (2013) *Canary Wharf Crossrail station cofferdam, London, UK: design, construction and performance* Geotechnical Engineering Vol.167, Issue GE2, pp169-181

Yeow HC, Nicholson DP & Man et al. (2014) *Application of observational method at Crossrail Tottenham Court Road station, UK* Geotechnical Engineering Vol.167, Issue GE2, pp 182-193

C138 Liverpool Street Station, C501 Moorgate Shaft – Verification Report, Crossrail document number: C138-MMD-C2-RGN-C101-50014 v4.0

Gaba et al. (2016) *Embedded Retaining Walls Guidance for Design*, Construction Industry Research and Information Association (CIRIA), London, UK, Ciria C760. To be published

Farooq I. et al (2015) *An innovative Verification Process speeds construction of Crossrail's Moorgate shaft* – Proceeding of ICE, London.

Chen et al. (2016) Development of framework for the Observational method approach. Under preparation

An innovative Verification Process speeds construction of Crossrail's Moorgate shaft

Imran Farooq BSc MSc CEng MICE MASCE, Technical Director, Mott MacDonald. UK
David Place BSc(Eng) CEng MICE, Technical Director, Mott MacDonald, UK
Brian Steele BEng CEng MICE, Cross Rail Engineering Manager, London, UK
Hock Liong Liew BEng MEng PE(Civil) Singapore, Principal Geotechnical Engineer, Mott MacDonald, UK
Sergio Vitorino BSc, Geotechnical Engineer, formerly Mott MacDonald, UK

Abstract

Liverpool Street station was on the critical path for the completion of Crossrail's central tunnelled section; however the start of the station's 42m deep Moorgate shaft was delayed by the prolonged time required to extract the foundation piles of the building that had previously occupied the site. As this delay would have led to knock on delays in completion of the shaft and hence the transit of the running tunnel TBM through the shaft, a carefully defined and controlled observation based Verification Process that makes innovative use of real-time site monitoring data and state-of-the-art 3D numerical analysis of ground-structure interaction, ground movements and movement of adjacent tunnels was developed. The paper describes the application of the Verification Process: iterative analysis using FLAC software; a new constitutive non-linear soil model for clays; field observation during the works and implementation during construction. The outcome was that as the shaft was being excavated the Verification Process progressively demonstrated that each of three levels of propping could be omitted, and the number of excavation stages reduced by two. All of this was extremely good news for Crossrail as it shortened the shaft construction programme by 14 weeks which meant there was no delay for the TBM drive for the westbound running tunnel.

Introduction

Crossrail's Liverpool Street station will serve the City of London and provide interchanges with London Underground's Northern, Central, Metropolitan, Circle and Hammersmith & City Lines, connections to Stansted airport and National Rail services at Liverpool Street and Moorgate stations. Built at deep level to pass beneath LU's Northern and Central Lines, the new platforms extend between the existing Liverpool Street and Moorgate stations with both stations having access to the Crossrail station. With Crossrail operating 24 trains/hour, 26 million people per year will use the new station.

In the morning peak Crossrail services will deliver some 8000 passengers per hour at Moorgate. The existing Moorgate station ticket hall is being enlarged to create an integrated western ticket hall from which a bank of three escalators will descend to an intermediate concourse, whence a further bank of three descends to the platform central concourse 35m below street level. This entailed the construction of the deepest shaft on Crossrail to 42m below street level (113m ATD), all within one of Crossrail's most constrained sites (Figure 1), bounded by the Metropolitan Line to the north, the Northern Line tunnels to the east, listed buildings immediately to the south and the existing Moorgate station ticket hall nearby. The shaft's 8 levels of basement accommodate the upper bank of escalators, lifts and emergency escape stairs from platform level, tunnel forced ventilation (60m^2), draught relief (40m^2), smoke extract (10m^2) and associated plant rooms.

The westbound running tunnel passing through the shaft placed the shaft on the critical path for completion of the 8.2 km long Y-drive tunnels being driven by TBM from Limmo to Farringdon. The shaft had to be completed by the end of November 2014 to allow construction of the launch chamber to speed TBM Victoria on her way to Farringdon. The timely passage of the TBM through Liverpool Street Station was fundamental in meeting the Crossrail critical path programme for the opening of the central London section in December 2018; any delay in the construction of the shaft was untenable.

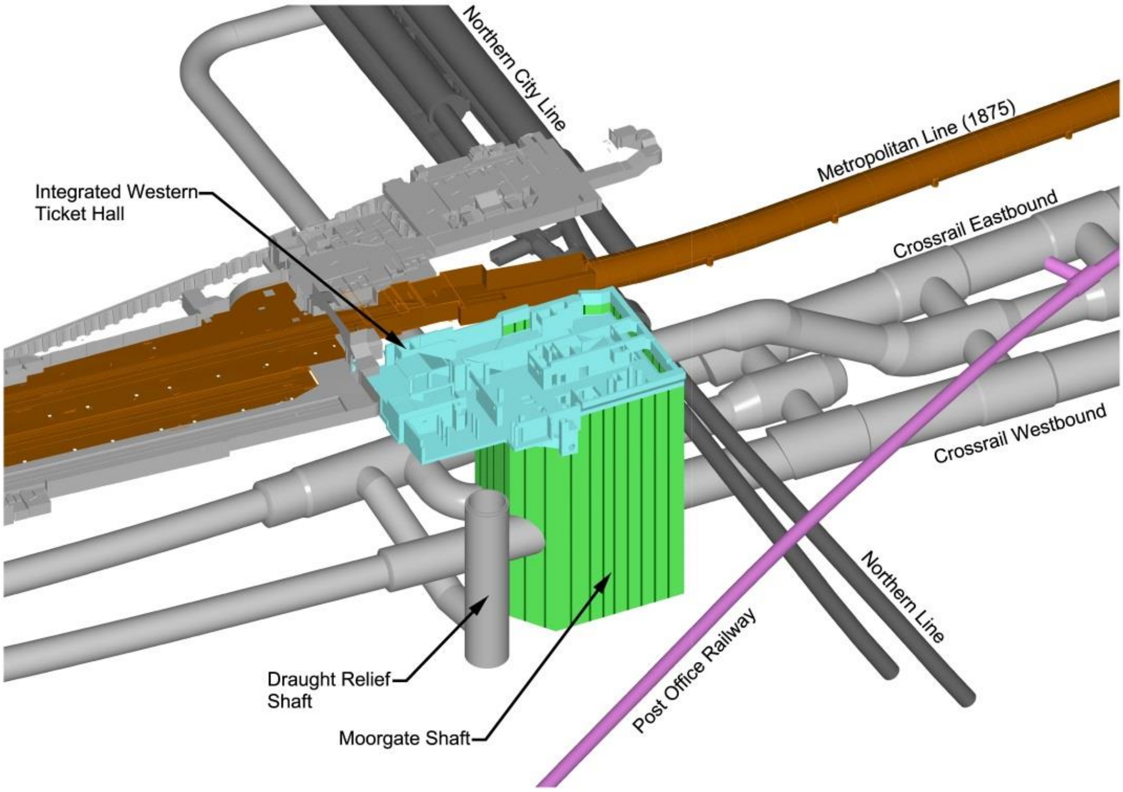


Figure 1 3D CAD model of Moorgate shaft and adjacent tunnels

Design of the shaft

The shaft was designed by Mott MacDonald under the Liverpool Street station design contract C138. It is a diaphragm wall structure, an irregular polygon in plan measuring approximately 35m x 35m as shown in Figure 2, constructed top down with seven reinforced concrete ring beams installed between levels 107.55m and 83.10m ATD as the excavation is taken down in stages. Below the lowest ring beam the westbound running tunnel passes through the shaft and two levels of temporary propping (at 80.5m and 76.0m ATD) were specified to restrain deformation until the 2m deep base slab had been cast at level 74.2m ATD. With the shaft's east wall less than 5m from the 100 year old northbound tunnel of LU's Northern Line beneath Moorgate, limitation of deformation to protect nearby assets rather than strength governed the design of the shaft.

The site geology is shown in Figure 3. One anomaly of note was a scour hollow feature, up to 10 m deep within the London Clay, coincident with the footprint of the shaft, which had to be taken into account in the ground model for the design. The RIBA E and F design of the shaft had been carried out using Crossrail Engineering Design Standards (CEDS). These are a project specific compilation based on Eurocodes and British standards, and in terms of clay strata preclude the possibility of using undrained strength and stiffness parameters on the passive side of the excavation. In the original design use of CEDS had led to a structure that required very heavy internal ring beams to stiffen the upper part of the shaft. Below the ring beams there was a combination of two temporary cross walls and the early build of part of the internal structure - a pair of slab strips (3m wide by 1.5m deep) specifically to restrain movement in the east-west direction. In the lowest part of the shaft the two levels of temporary propping were to be provided for further restraint. The intent was to limit wall horizontal movement generally to no more than 45mm in total, but for the east wall adjacent to the Northern Line the limit was a maximum of 30mm. Because of these constraints, for the original design the construction sequence was prescribed in the Works Information drawings - the 11 stages are shown in Figure 4.

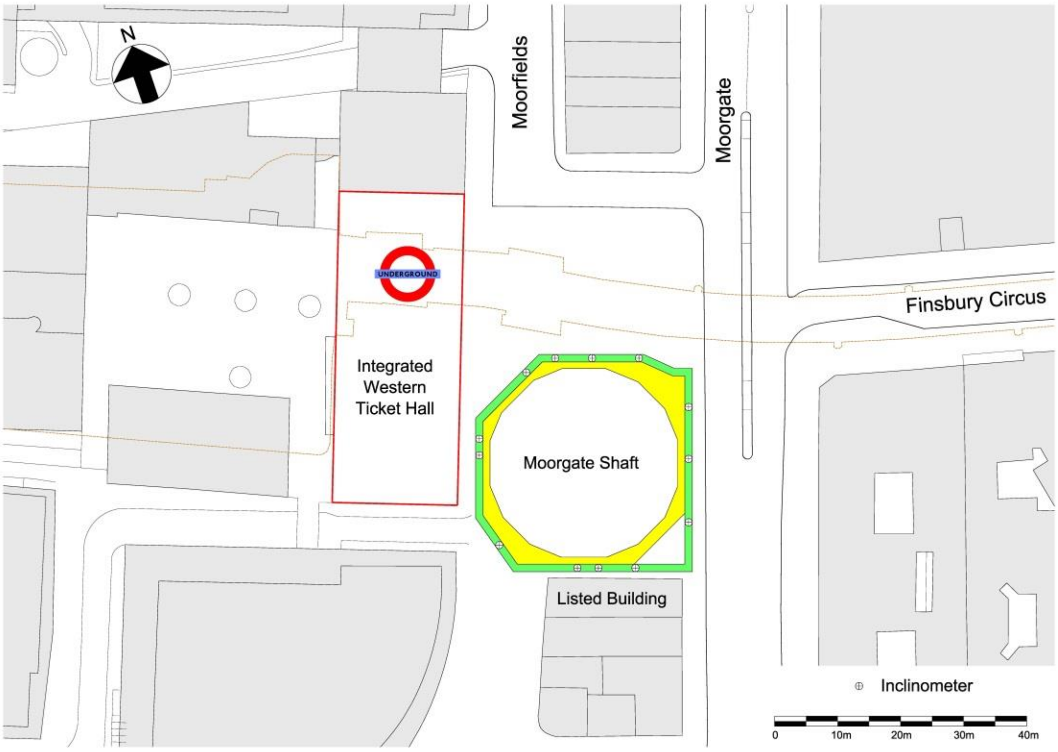


Figure 2 Moorgate shaft site plan and inclinometer locations

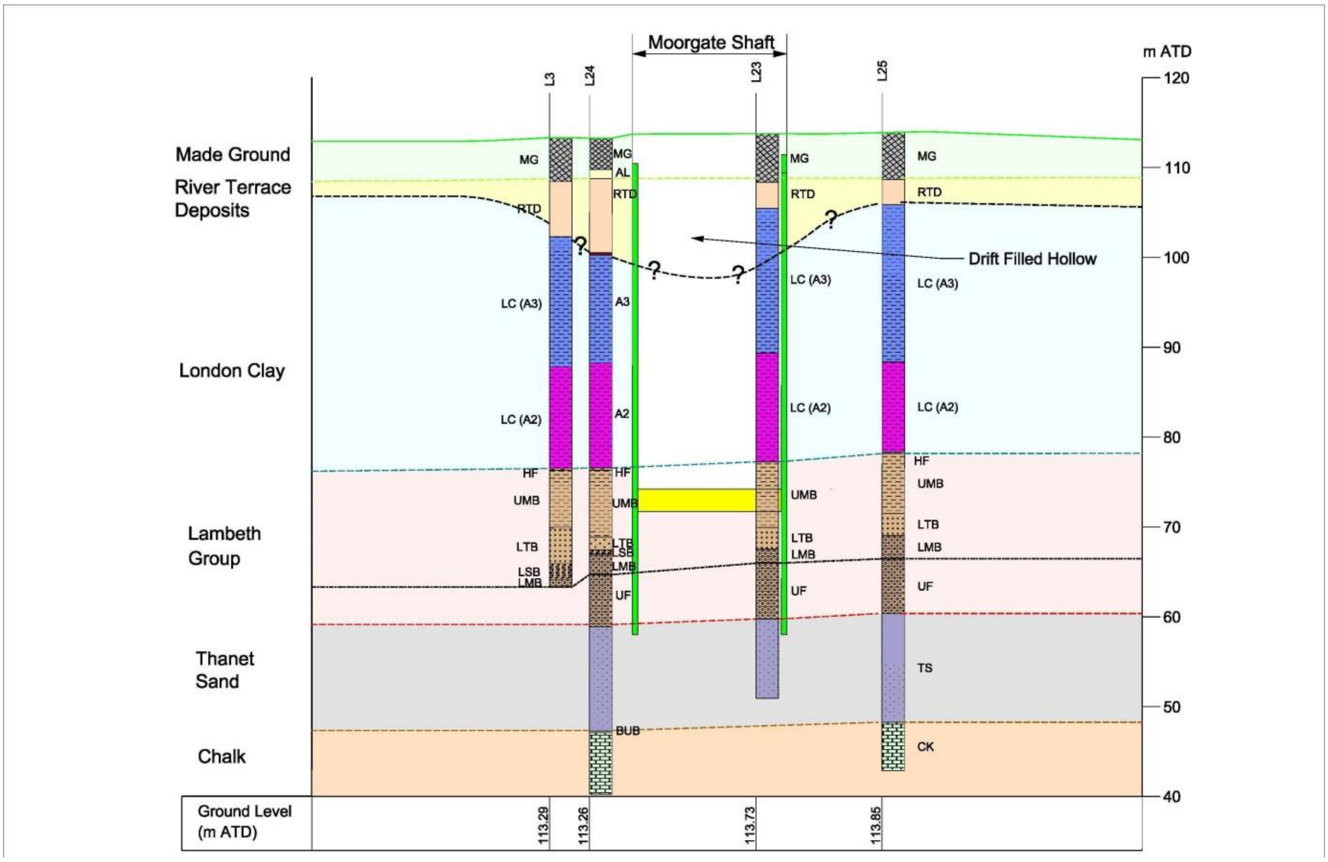


Figure 3 Moorgate shaft geological section

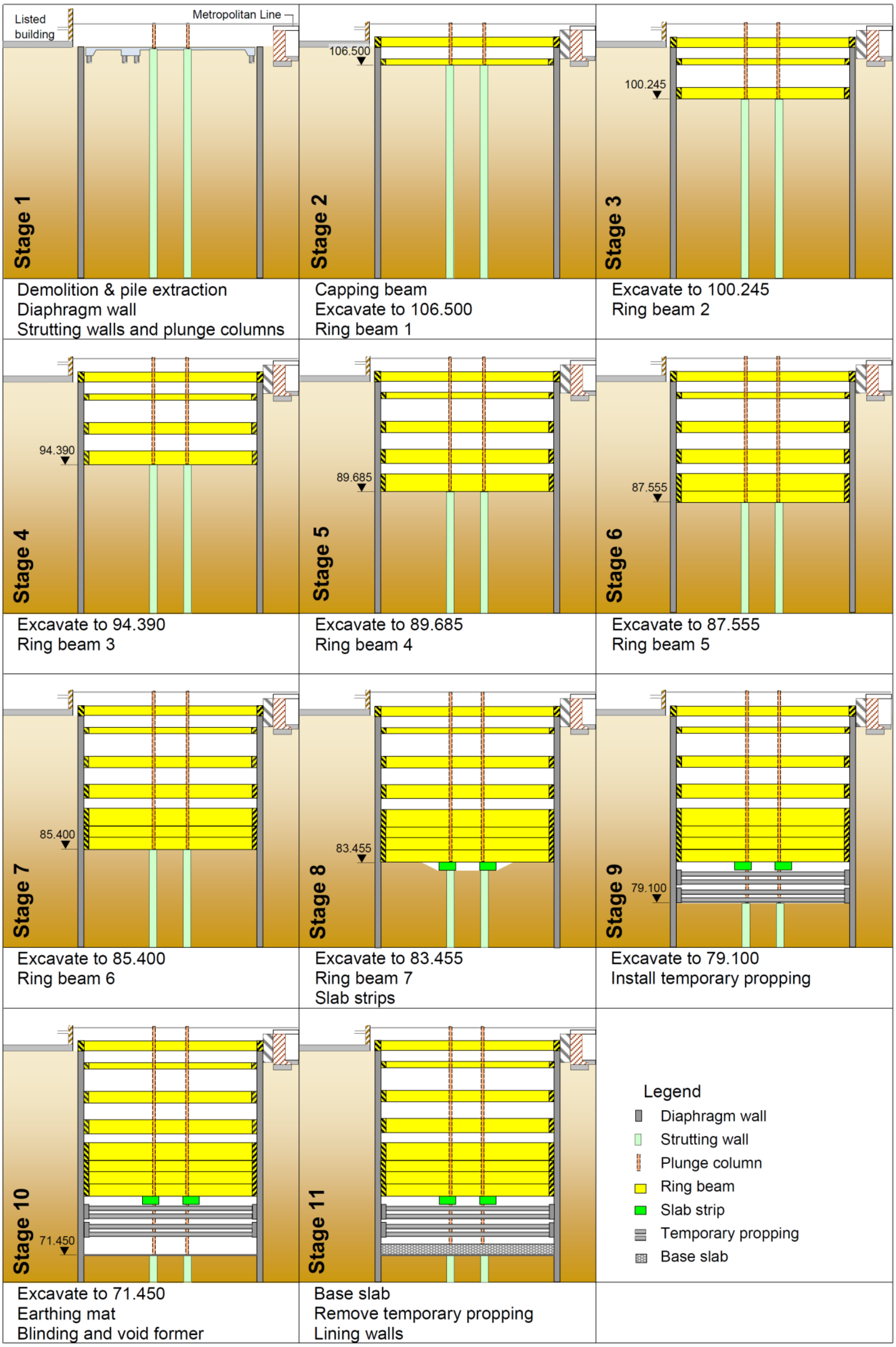


Figure 4 Moorgate shaft construction sequence for the original design

The need for programme acceleration

The C501 construction contract for the Moorgate shaft was awarded in March 2011 to the BAM Nuttall Ltd / Kier Construction Ltd JV (BNK) for a start on site in July 2011. The shaft was to be constructed on the footprint of 89-134 Moorgate, immediately opposite the new Moorgate integrated ticket hall. The building previously occupying the majority of the shaft site was 91-109 Moorgate (Amro Bank), a 1970s 6 storey concrete framed structure with a single storey basement. The building spanned over the LU subsurface tracks and was founded in the main on 900mm diameter bored piles approximately 34 metres long. When this building had been demolished in advance of the C501 shaft construction works contract the piles were left in place. Removal of the redundant piles became a major issue for the construction programme, with diaphragm wall subcontractor Bauer having to develop a bespoke extraction machine to clear the footprint of the shaft before the diaphragm walls could be started. On completion of pile removal the programme delay was 11 months, so work on the 55m deep 1.2m thick diaphragm wall panels did not start until the end of January 2013. By the time the diaphragm walling was completed in August, the project milestone for handover to the C510 SCL tunnel contractor at the end of November appeared to be a forlorn hope.

To mitigate the delay, in April 2013 designer Mott MacDonald had proposed reanalysis of the shaft using non-linear soil parameters, and state of the art soil structure interaction modelling using FLAC 3D. Crossrail granted a concession against CEDs to allow use of undrained conditions on the active side for fine grained soils provided that the duration of the excavation down from the London Clay horizon (100.4m ATD) to completion of final excavation level (71.45m ATD) was less than one year. The designers embarked on reassessment of the shaft including 3D numerical modelling of the actual construction sequence of both the shaft and the adjoining SCL tunnels. The reanalysis used the well-established method of implementing non-linear stress-strain characteristics for heavily over-consolidated soils (via a stiffness degradation curve) in numerical modelling as outlined by Jardine [1] The shaft and SCL tunnels are represented by some 55,000 linear elastic three-noded flat shell elements in the 3D FLAC model as shown in Figure 5 (the approx. 270,000 eight-noded brick elements with non-linear stiffness and Mohr-Coulomb failure criteria representing the soil are not shown). The outcome was that the lower level of temporary propping at 76.0m ATD could be omitted. It was also identified that there was potential for omission of the temporary propping at 80.5m ATD as well so that at the bottom of the shaft the diaphragm wall spanned 12m between the lowest ring beam and the final excavation level at 71.45m ATD.

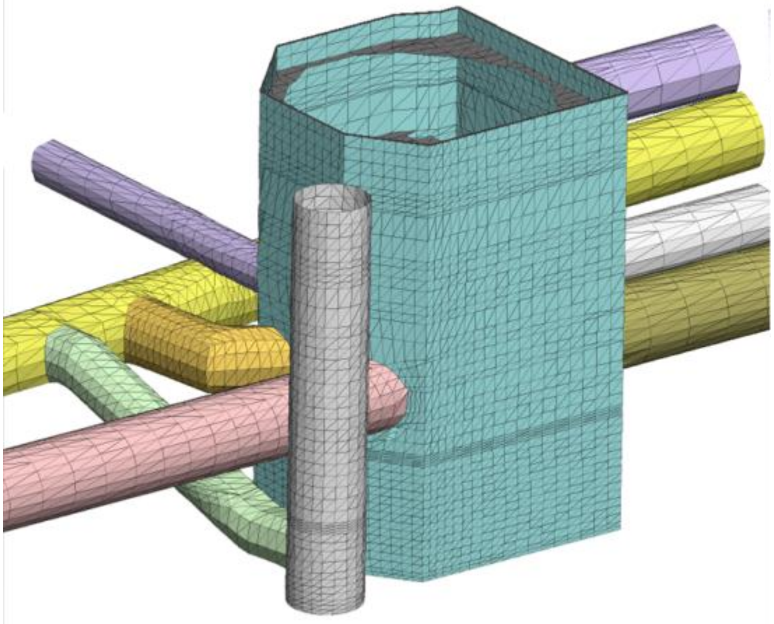


Figure 5 3D FLAC model of Moorgate shaft and adjacent tunnels

At this time the first two of the 11 construction stages had been completed. To maximise future programme benefit, Mott MacDonald conducted a value engineering review of the remaining stages. As the contractor was placing blinding around the perimeter of the shaft to support the construction of each ring beam, recent case history by Powderham on the use of temporary blinding slabs to mitigate softening of the ground and control wall movement for the Heathrow Airside Road Tunnel [2] was integral to this review. It was considered that by placing intermediate blinding slabs across the entire shaft at the completion of the dig for each excavation stage, construction stages 5 and 6 could be combined, (i.e. the excavation for ring beams 4 and 5 could be done in one continuous dig and the ring beams then cast sequentially), and there could be opportunity for the combination of two further stages (7 and 8) and also the omission of the slab strips.

The observation based Verification Process

The highest priority was the omission of the temporary propping at 80.5m ATD as its installation would take several weeks, and whilst in place it would severely constrain access for the lowest level of excavation and subsequent construction of the base slab. Furthermore the TBM could not transit through the shaft with this propping in place. Hence it was necessary to develop a methodology that evaluated value engineering opportunities on the basis that the omission of the 80.5m ATD propping was not jeopardised, and an observation based Verification Process was proposed. With Crossrail's support Mott MacDonald developed a carefully defined and controlled observation based Verification Process. The difference between an observation based verification method and the conventional observation method is that in the latter the contingency measures are only implemented if trigger levels are breached, whereas with the verification process all such measures are included in the design, and will only be omitted when projection based on observations during construction confirm they are not needed. This approach is a practical application of Bayesian inference [3] where the probability for the hypothesis is updated (hypothesis being the potential prop removal) as data is acquired (data being the observed shaft behaviour during construction).

This approach provides the project with a bounded programme. Accordingly, the construction sequence drawings were re-detailed to show four intermediate blinding slabs, the slab strips, and the 80.5m ATD temporary propping all to be constructed as mitigation measures, but with the Project Manager having the authority to instruct their omission should the outcomes from the Verification Process be favourable. Key requirements for the use of the observation based Verification Process are:

1. High quality real time monitoring of wall movement, ground settlements, pore pressures and heave within the excavation are essential.
2. Use of a non-linear ground model capable of capturing soil structure interaction in a calibrated numerical model.
3. Multiple iterations calibrating the soil model to observed behaviour are carried out.
4. Close collaboration between contractor, designer and client is necessary, with frequent meetings to review the outcome from modelling and potential changes to construction activities.
5. At each stage a fully assured set of construction sequence drawings is produced in parallel with the analysis so that favourable outcomes can be instructed and implemented without delay.

For the Verification Process the A* soil model based on the approach outlined by O'Brien *et al* [4] [5] was adopted. For this model the ground stiffness parameters were derived through a holistic framework that considers both field and laboratory test data to determine key ground parameters, and included back-analysis of case histories of tunnel construction for the Heathrow Express and the King's Cross SCL tunnels. This A* model was already being used by Mott MacDonald for the adjacent Crossrail SCL tunnel design, and the monitored SCL lining movements were broadly in agreement with the numerical modelling predictions. Figures 6(a) and 6(b) show non-linear stiffness profiles for the London Clay and the Lambeth Group respectively (the A* model is the blue line and the approach based on Jardine is the green line).

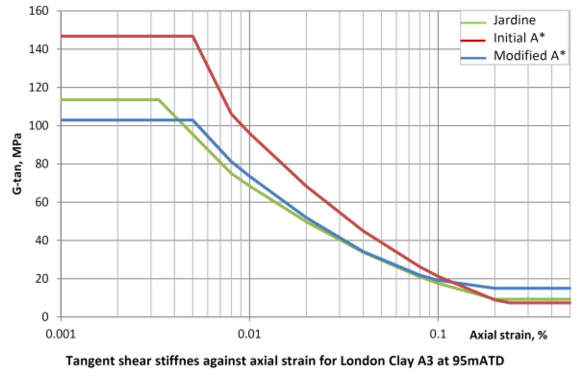


Figure 6(a)
Non-linear Stiffness Profiles London Clay

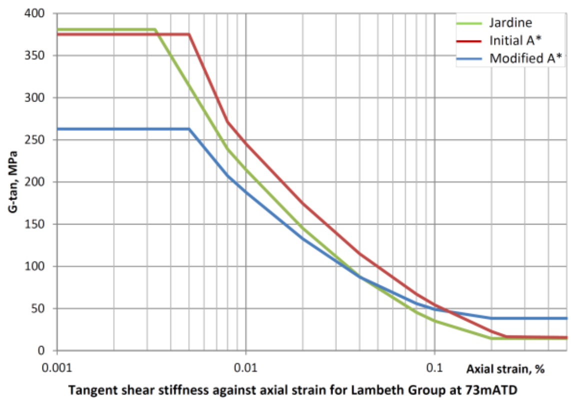


Figure 6(b)
Non-linear Stiffness Profiles Lambeth Group

Monitoring Strategy

A comprehensive monitoring strategy had already been implemented to provide control of wall displacements to pre-agreed limits to ensure the integrity of adjacent infrastructure. This monitoring system was enhanced to provide a real time assessment of the structural performance of the shaft within the footprint of the structure so that it included 13 inclinometers in the diaphragm walls (in-place inclinometers, manual inclinometers and ShapeAccelArrays were installed as shown in Figure 2), magnet extensometers to measure heave, vibrating wire piezometers to monitor pore water pressures in the Lambeth Group, standpipes to monitor groundwater levels in the Lower Aquifer and strain gauges with temperature measurement to monitor stresses in each ring beam. Empty inclinometer tubes had also been installed adjacent to each inclinometer to provide redundancy in the monitoring scheme. To verify inclinometer readings and to increase confidence in the measured wall displacements, mini-prism survey points were installed on the face of ring beams and monitored on a daily basis. Laser extensometer survey points were installed when excavation progressed into the Lambeth Group to monitor convergence of the walls.

Implementation of Observation based Verification Process

The Verification Process was based on real time assessment of data and had to be linked to the defined construction sequence stages as work progressed on site. To manage the Verification Process three verification points (VPs) were defined. Successive calibration of the FLAC numerical model against wall movement data was to be undertaken once the excavation reached 94.19m ATD (VP1), 87,355m ATD (VP2) and 82.9m ATD (VP3) respectively to assess the potential omission of the propping and combination of construction stages, together with variations to mitigation measures should wall movements approach trigger levels.

Additional mitigation measures comprising an intermediate blinding slab at excavation levels 94.19m, 87.355m, 82.9m and 79.1m ATD were introduced. These slabs were specified as 200mm minimum thickness unreinforced 20N/mm^2 structural concrete for early strength gain, to be cast within 24 hours of completing the dig. Initially the scope of the Verification Process considered omission of these intermediate blinding slabs at each of the verification points, however in the event, due to their effectiveness, none of these were omitted.

All mitigation measures were required from the outset to provide certainty for the construction programme's end date, such that any subsequent changes would be beneficial in bringing that end date forward. All the mitigation measures were included explicitly on revised construction sequence drawings, whereas previously only the temporary propping and slab strips had been shown. The construction sequence drawings retained the original top down sequence for constructing the ring beams from Stage 5 downwards in four horizontal slices. Then the Verification Process was started to consider the omission of the mitigation measures and the combination of Stages 7 and 8 (excavation to 85.4m and 83.455m ATD, and construction of ring beams 6 and 7). The construction sequence drawings were modified as necessary as the process progressed to each verification point (VP). To facilitate understanding the Verification Process, a detailed flow chart was produced and presented to the Crossrail delivery team and the Contractor (an abridged version is shown in Figure 7). The desired outcomes and the target verification points for the decision on incorporation are summarised below:-

- ► VP1 Omission of slab strips
- ► VP2 Combine stages 7 & 8
- ► VP3 Omission of propping at 80.5m

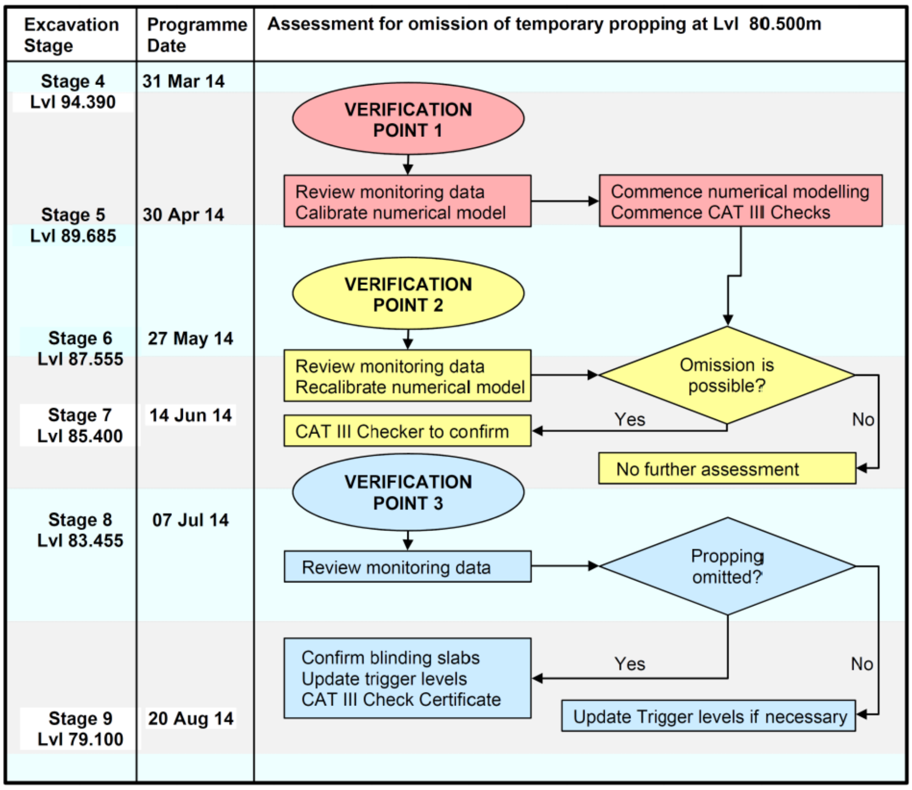


Figure 7 Flow chart for Verification Process

The following activities were undertaken during all three VP periods:

1. Wall displacement data from the monitoring is processed. Daily monitoring data was averaged and anomalies were isolated. Stage-by-stage monitoring data was plotted out and compared with wall displacement previously predicted by the FLAC 3D analysis.
2. The existing FLAC 3D model is updated to incorporate:
   - actual sequence as built and best prediction still to go for SCL tunnelling;
   - more accurate topography of the ground around the shaft;
   - more accurate drift filled hollow geometry;
   - refined ring beam geometry; and
   - modified ring beam and crosswall stiffness to replicate observed shaft behaviour.
3. Revised wall displacement predictions from the new FLAC 3D models were plotted against monitoring results for each stage. Revised predictions for the subsequent stages were also plotted to allow for immediate comparison for subsequent stages of excavation.
4. Category III checks were carried out in parallel to provide an assured design at each verification point, with the real time raw and processed data being provided to the Category III Checker.

Verification Point 1 (VP1) was started on 10 April 2014 when the excavation level had reached 94.19m ATD, and the first 200mm intermediate blinding slab was cast at this stage.
It took about five weeks to complete the VP1 process Based on the results the North, West and South walls showed a reasonably conservative match between prediction and monitoring (i.e. predictions were higher than measured movements). The predicted results for the dig from 94.190m to 87.355m ATD also showed that there was a reasonably conservative match. However results for the East wall were different; prediction was lower than measured movements as shown in Figure 8. The reliability of inclinometer data from the East wall was initially questioned by the design team as

there was a good match on the other three walls. Furthermore the effectiveness of the two unreinforced concrete crosswalls in the east-west direction was questioned.

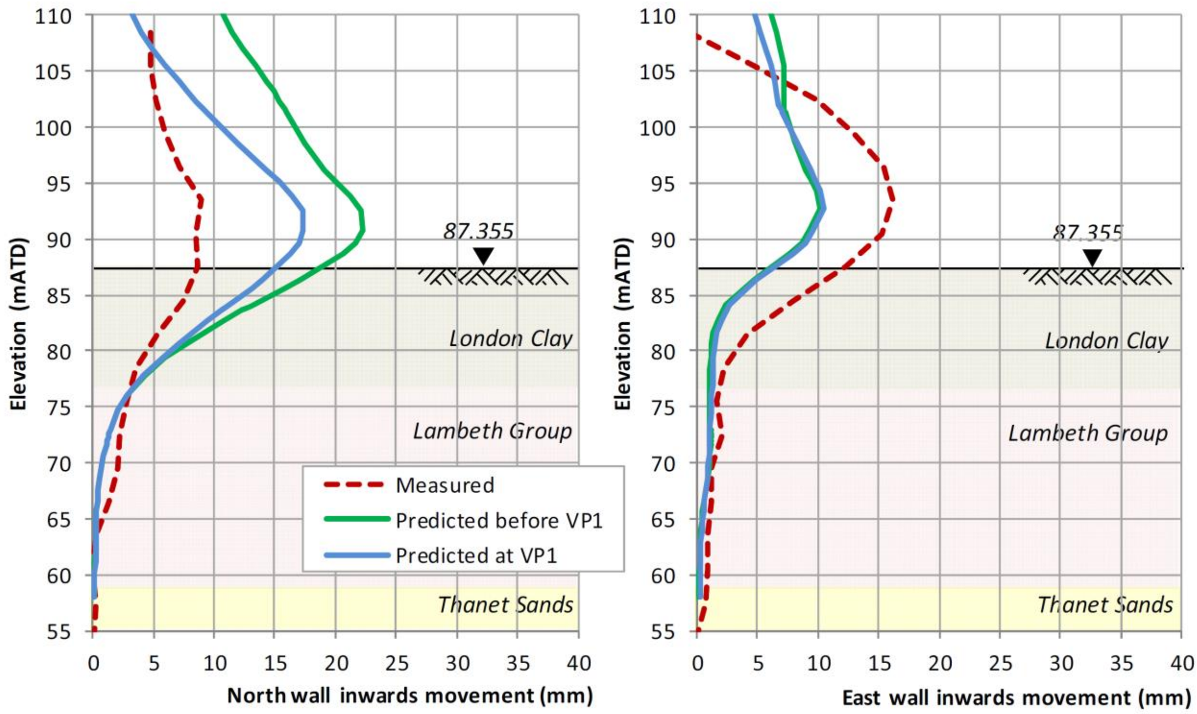


Figure 8 Wall movements at VP1

As part of investigating the cause of the greater than expected East wall movement, it was identified that permeation grouting had been carried out adjacent to the East wall concurrently with the excavation in Moorgate shaft. It was considered possible that the grouting could have been a contributing factor to the higher movements of the East wall, and this view was supported by monitoring data from nearby third party assets. Based on the results from the North, South and West walls, it was concluded that there was a high probability that the temporary props at 80.5m ATD and the slab strips could be omitted. However, as the predicted movements at the East wall were lower than the measured movements, it was recommended that the decision for the omission of the slab strips should be deferred to the next verification point (VP2), and the potential effect of the permeation grouting on the East wall be assessed further.

Verification Point 2 (VP2) was started when excavation had reached 87.355m ATD on 18 May 2014. The 200mm intermediate blinding slab was cast at this level on 19 May. VP2 was completed on 2 June when the excavation level was still at 87.355m ATD. Based on back analysis of the ground movements at the shaft, the constitutive non-linear A* soil model was modified to the red line in Figures 6(a) and 6(b). The key modification was for the calculation of initial isotropic stiffness where the horizontal stiffness at very small strain has been considered in the calculation since the horizontal displacements were likely to dominate the response of the walls of the shaft. Due to the close proximity to the Northern Line tunnels it was essential to understand the variation between the East wall predictions and measured movements, and so the scope of the VP2 was extended to include:

5. Assessment of the effects of permeation grouting on the shaft, modelled by applying grouting pressure over the zone between 104m and 100m ATD in the FLAC model.
6. Sensitivity analyses to assess the potential effects of a “relic shear zone” behind the East wall, by varying grout pressures and increasing soil unit weight within the grouted zone.
7. Assessment of the effect of reduction in crosswall stiffnesses.

Since the crosswall panels that abut the diaphragm wall and the corresponding diaphragm wall panels were constructed individually, there was concern that there could be clay inclusions at the junction between the ends of the crosswalls and the face of the diaphragm wall panels. Field observation of the crosswall junctions was carried out when the excavation level was at approximately 87m ATD and was continued until the excavation level had reached the final formation level at 71.45m ATD.

Continuous clay inclusions were seen to be present between diaphragm wall and crosswalls, however the clay had not failed and there was a load transfer mechanism present (this was also confirmed by calculation). To account for this effect, the FLAC models were run with various crosswall stiffnesses (full stiffness, 10% stiffness and zero stiffness). For the North and South walls, the revised FLAC modelling showed an even better match giving confidence that the modified ground parameters in FLAC were more realistic. For the East wall, both the deflected shape between level 90m and 110m ATD, and the top movement were still not fully replicated by the FLAC 3D models in VP2. Therefore a bounded approach was used for the East wall. The upper bound East wall projection for excavation to full depth increased (with temporary propping removed) to approximately 32mm which was higher than the "Red" trigger level (i.e. 30mm), see Figure 9.

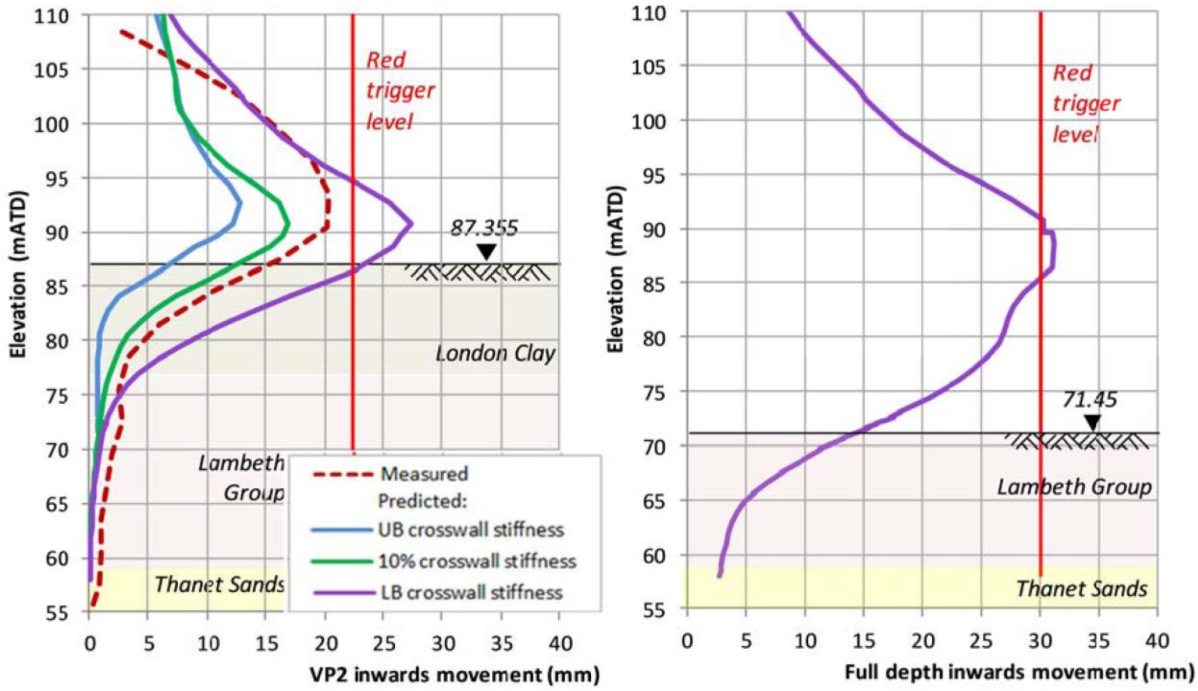


Figure 9 East wall movements at VP2

Based on the analysis results, it was concluded that there was a high degree of confidence that the temporary propping at 80.5m ATD could be omitted and that construction stages 7 and 8 could be combined without jeopardising the omission of that propping. By combining these two stages so that the excavation could be done in one continuous dig and the ring beams then cast sequentially, the total construction time would be reduced, thus limiting secondary ground movement due to softening of the London Clay. However there appeared to be a temperature related ratcheting effect on the shaft walls (more pronounced on the East wall) that required further investigation.

Furthermore VP2 had identified that there was a high risk that the 30mm 'red' trigger level for the East wall could be breached if contingency measure were omitted. Hence the decision to omit the slab strips was deferred until the effects on the LU Northern Line tunnels of ground movement due to all the Crossrail works in the vicinity up to that time had been reviewed, and the residual capacity for the tunnels to tolerate further ground movement had been assessed. Effectively this was to determine if there was scope for a relaxation of the "red" trigger levels for the East wall. Therefore whilst there was a higher degree of confidence that the omission of the slab strips would not compromise the omission of the propping at 80.500m ATD, as an additional contingency the design for an alternative simplified propping slab cast directly on the blinding was to be prepared in parallel in case results from VP3 were not favourable.

The diurnal temperature related movements that were observed when grouting was being carried out during late spring when large daily temperature changes were occurring were unexpected. The magnitude of accumulated movement at the top of the east wall was not observed to the same degree on the other three walls where grouting was not carried out. The designers postulated that likely explanation for this anomaly was that whenever the shaft moved inwards due to decreasing ambient temperature, the grouting taking place locally behind the east wall would stiffen the ground. Consequently the east wall could not move back into its original place during temperature reversal, and in each subsequent diurnal temperature cycle would move slightly further in; effectively there was a slow ratcheting movement inwards into the shaft (this will be subject of a separate paper).

Verification Point 3 (VP3) was started when excavation reached 82.900m ATD on 2 July 2014, and the 200mm intermediate blinding slab was cast at this level on 3 July. VP3 was completed on 25 July three weeks later. The following additional activities were introduced during the VP3 period:

8. Diurnal temperature related movement of the shaft (leading to ratcheting) was replicated by using reduced ring beam stiffness to match the contracted state of the ring beams.
9. The geometry and stiffness of all the ring beams was further optimised to incorporate the potential restraining effects of the temporary construction jetty.
10. Pore pressure measurements and measured ring beam convergence were compared to the predictions from the updated FLAC modelling.
11. Additional parametric studies were undertaken to assess the potential effects of limiting suction in the FLAC analysis and sensitivity analysis of potential changes in Lambeth Group geology.

Meanwhile meetings between London Underground and Crossrail's asset protection consultant had led to agreement on the relaxation of the "red" trigger level for the East wall from 30mm to 40mm. The results of the FLAC modelling for all the walls were plotted and compared with monitored movements. All movement predictions including the East wall were within the new "Red" trigger level of 40mm as shown in Figure 10. The results also showed that the potential effects on the Northern Line tunnels due to shaft excavation-induced movements were minimal. The predicted short term settlement of the tunnels due to the cumulative effect of the shaft excavation was less than 10mm.

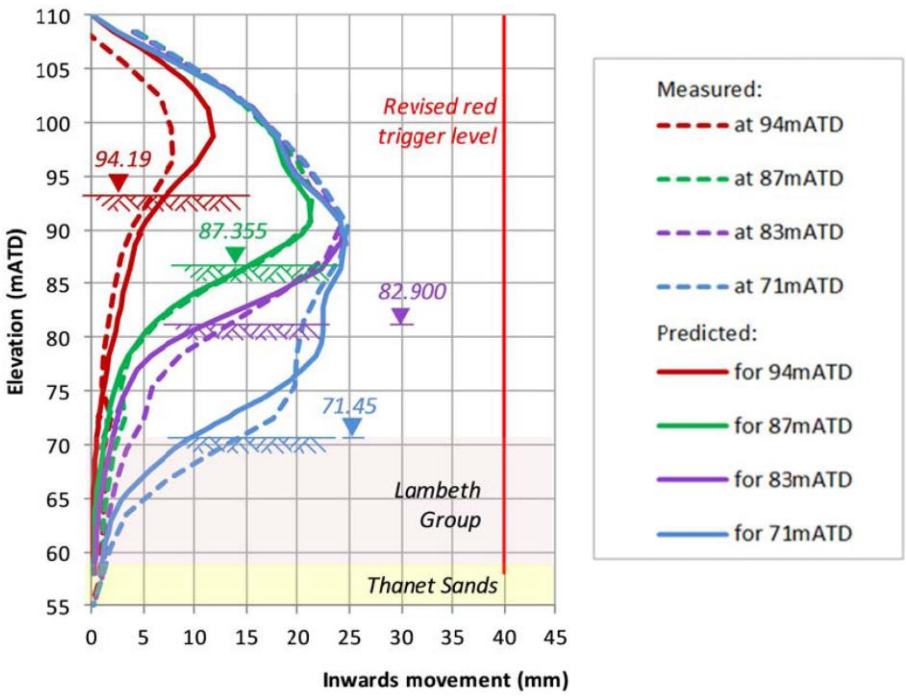


Figure 10 East wall movements at VP3

The results at the end of VP3 showed a good match to the accumulated East wall movements, representing a significant improvement from VP2. Previously anomalous movement at the top of the East wall were considered to be most likely due to combined effects from diurnal temperature (ratcheting) and permeation grouting together with reduced crosswall stiffness. Therefore it was concluded that the temporary propping at 80.5m ATD and the slab strips could be omitted without having to provide the alternative simplified propping slab. The progression of the construction sequence from the original design to the actual as-built sequence after completion of the Verification Process is shown in Figure 11.

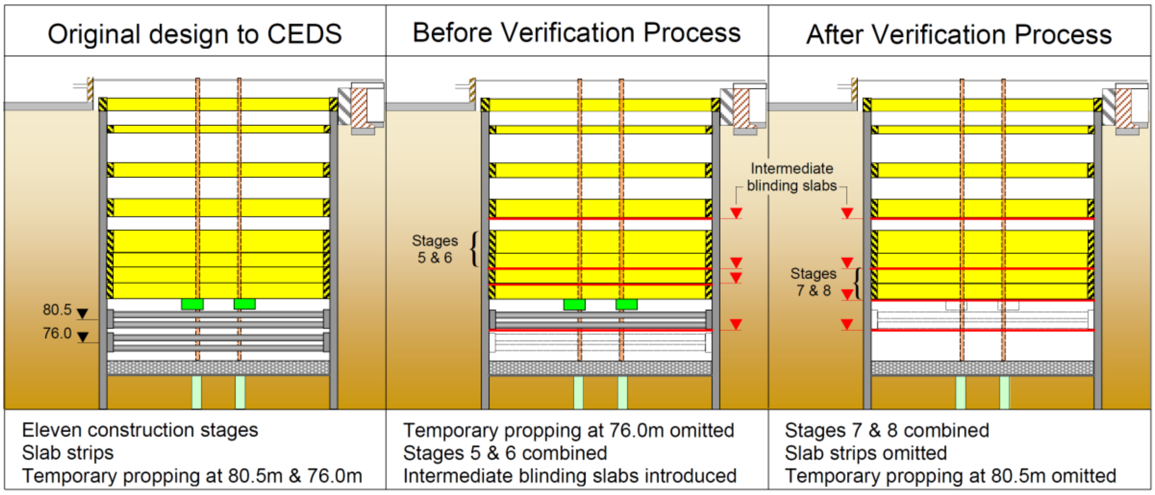


Figure 11 Changes to the construction sequence

Conclusion

The Verification Process was implemented successfully and achieved the required objective of an accelerated construction programme that allowed the Project critical milestone for handover of the shaft to the SCL tunnel contractor to be met. The Moorgate shaft excavation was bottomed out on 1st October (Figure 12) and the 1800m^3 concrete pour for the base slab was completed on 1st November, two weeks ahead of programme [6]. Crossrail Director Central Section Bill Tucker said: “This achievement is a culmination of an extraordinary story of collaboration between the contractor, designers and CRL. Looking back six months ago there were many doubts about the ability to excavate, construct and bottom-out the shaft on programme and the teams did it!” TBM Victoria transited through the shaft in April 2015 and was launched on her way to Farringdon on 13th April 2015. In June 2015 the Accelerated Programme for the Moorgate Shaft at Crossrail’s Liverpool Street Station won the Editor’s Award at the Ground Engineering Awards

Figure 12 Moorgate shaft excavation to 71.250 m ATD

The entire process was peer reviewed both internally by Mott MacDonald, and also by Crossrail Central Engineering Group (CEG) and their independent consultants. Throughout, the construction sequence drawings were progressively revised, and each revision was subject to Category 3 check, thereby maintaining full design assurance. Due to the small magnitude of the wall movements, typically only a few millimetres at each stage of the excavation, the Verification Process required careful consideration of all aspects of the as constructed geometry, and soil and ground parameters. After a number of iterations, and following the process through the three verification points, the design team were able to arrive at validations and predictions that were reliably close to measured movements, and hence give the assurance required to omit the propping and combine excavation stages.

The following key technical points can be made:

- Use of blinding slabs has been an effective way of controlling wall movements. Their use reduced ground softening and provided propping through diaphragm action. Conversely the strutting walls were less effective than expected.
- Expedited construction limits the amount of softening occurring in clays. Installing temporary propping during excavation (although this stiffens the structure) can be counter-productive as the time required to install the propping itself can allow the clay to soften, (especially when complex temporary propping is required). Hence when temporary propping is required to limit wall movement, use of an observational approach can provide a quicker and more cost-effective way of achieving the same goal.
- Use of an observational approach requires close collaboration between Designer, Contractor and Client's site team. With this approach the construction drawings have to be progressively updated and assured, and issued to the contractor for construction within very short time frames.
- The combined effect of grouting to stiffen the ground prior to tunnelling, and diurnal temperature variations contributed to the atypical behaviour of the East wall, causing a slow ratcheting in the wall movements.

Acknowledgements

The authors would like to acknowledge the permission of Crossrail Ltd for the publication of this paper. Views expressed by the authors are however their own. They would also thank their colleagues in the Crossrail project team, contractor BAM Nuttall / Kier JV and Category 3 checker Hyder who contributed to the successful outcome of this project. Last but not least, thanks to our colleagues, especially Tony O'Brien, within Mott MacDonald, who provided much valuable assistance.

References

[1] Jardine *et al* (1986) *Studies of the influence of non-linear stress-strain characteristics in soil-structure interaction*. Geotechnique 36, No. 3, 377-396, ICE, London.

[2] Powderham (2009) *The Observational Method – using safety as a driver for innovation*. Vienna Terzaghi Lecture, Österreichischer Ingenieur-und Architekten-Verein, Austria.

[3] Fienberg (2006) *When did Bayesian inference become "Bayesian"?* Bayesian Analysis Volume 1, Number 1 March 2006.

[4] Eadington & O'Brien (2011) *Stiffness Parameters for a Deep Tunnel – Developing a robust parameter selection framework*. Proceedings of the 15th ECSMGE, Athens.

[5] O'Brien & Harris (2013) *Geotechnical Characterisation, Recent Developments and Applications*. 12th International Conference on Underground Construction, Prague.

[6] Douglas, Smith & Hunter (2015) *Delivering the Moorgate shaft base slab on Crossrail, UK.* ICE Proceedings, November 2015.

MONITORING & INNOVATION

Monitoring

A Piling Contractors Perspective

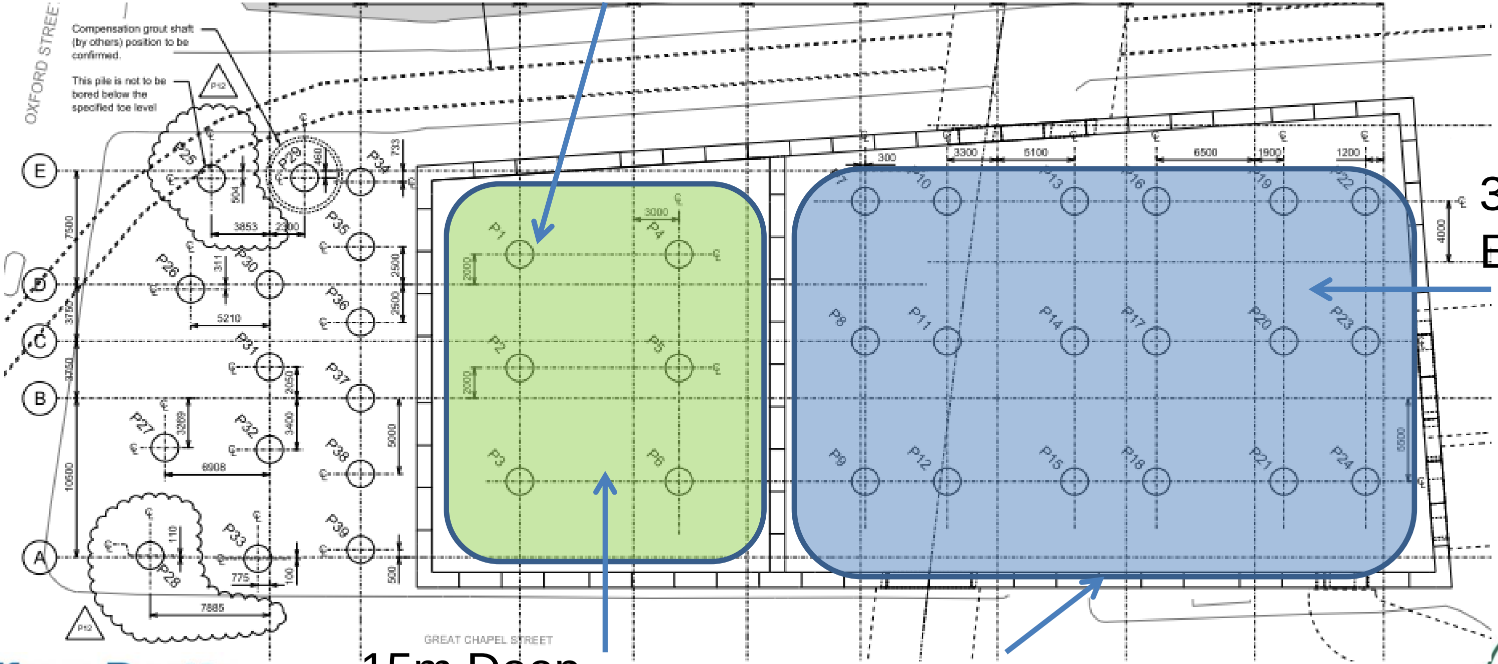
Mark Pennington

Introduction

- Project Scope
- Monitoring Scope
- Proposed Monitoring
- Actual Monitoring
- Summary

Scope of Piling Works

39 No Large Diameter Piles



30m Deep Excavation

15m Deep Excavation

70 Diaphragm Wall Panels

Balfour Beatty Ground Engineering

FPS 50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Crossrail

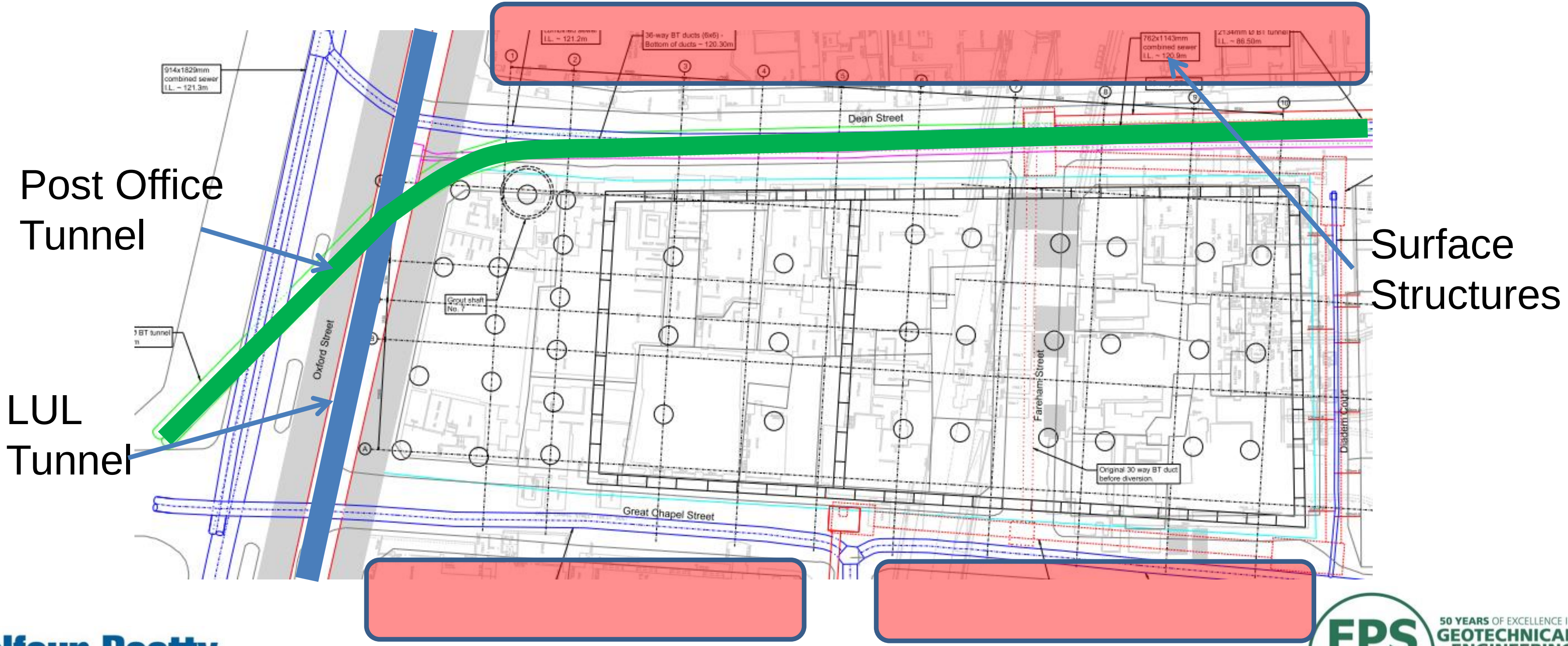
Station Box Location

Crossrail

Balfour Beatty
Ground Engineering

FPS
50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Monitoring Scope



Piling Works and Excavation Compared

Balfour Beatty
Ground Engineering

FPS
50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Crossrail

Planned Monitoring

- Automated Monitoring in the LUL Tunnels
- Automated Monitoring in the Post Office Tunnel
- Surface Monitoring of all Structures
- Clear Baseline and Trigger Levels

Actual Monitoring

- Limited Baselines Established
- Manual Monitoring Established in Post Office and LUL Tunnels
- Surface Monitoring points established
- Trigger Levels Agreed
- Monitoring Panel Set up and then Handed Over`

Summary

- Detailed Monitoring Established for Short Duration
- Monitoring should be Developed around the Critical Construction
- Baseline and Standardisation is key
- - Presented from a piling contractor perspective

IncloView

Paul Thurlow

Getec UK

Abstract

The visualisation of inclinometer data over time has consisted of basic line graphs with very basic provision for time dated excavation depth, prop positions, soil profiles or wall performance design curves. The data is usually viewed with dongle activated software which makes relating data to excavation or other site activities a time consuming process which has the potential to create miscommunication and lead to delays and increases in cost whilst the data is reviewed away from site, discussed in meetings and then any feedback relayed back to those on site. This paper will discuss the benefits of using technology to visualise and inform engineers on inclinometer data and surrounding site activities.

With the recent growth of the smart phone generation, a thirst for access to 'live' data has become more and more popular. By providing all the relevant information relating to inclinometer measurements in one, simple to use smart device application known as IncloView, the whole process or reviewing data becomes an efficient process. It allows for data to be shared amongst the site team at the same time, regardless of proximity, and is always updating with the most recent measurements with the excavation level and prop locations. The result of which is that at any time no one is out of touch with the live events unfolding on site, allowing for engineers and designers to review inclinometer measurements real time, and check data against their designs in line with the Observational method.

Introduction

The large amount of excavations on the Crossrail projects required the use of inclinometers to measure wall deflections during excavation. The use of spreadsheets and dongle-activated software to view data from inclinometer instrumentation has been common practice for the last 40 years.

Data would be either manually downloaded or collected automatically via GSM/GPRS data connections and then loaded into spreadsheets of ever increasing size. This data would then be processed and graphs edited to present the correct data. Over time, these spreadsheets would become so large that they would stop responding or become slow to use, this resulted in data having to be removed from the spreadsheet to improve functionality but providing an incomplete picture of the project.

By the time a project is completed there is a potential for there to be large numbers of spreadsheets with thousands of lines of data within each one, making it very hard to find a specific measurement and then relate that back to site works for that moment in time.

The recording of excavation areas and depths and their relation to the wall or pile movement is an area where detail is lacking. In fact, this is the most important part of the observational approach to monitoring. During the development of the software, it was apparent from site meetings that propping and waling positions were not always visible in reports, which is an important factor for feedback to designers in wall performance.

It has been the authors concern that these elements should all been in one place for ease of making decisions on site and providing data to designers. The experience for all the design of the software is based on real management of large excavation projects

Concept

The initial spreadsheets comprised of many tabs and annotations that were difficult to work with. The system was not secure and could be open to bad data inputs. The design of the app had to address the following;

- Fast data presentation that would work on smart phones and tablets as well as be available as a link from the GIS based software. Ability to add trigger alerts and inform users by e-mail.
- Define the soil profile, dynamic excavation and prop locations and make searchable to minute accuracy. Importation of design lines for each section of works.
- Compatible with inclinometer data from all manufactures software outputs including Manual readings, In– Place MEMs Inclinometers and Shape Accel Arrays (SAA).
- Easy to use interface and functions and minimal set up time. Ability to expand functions in the future and to work in any main monitoring software.
- Data to be exported easily to insert into design software.

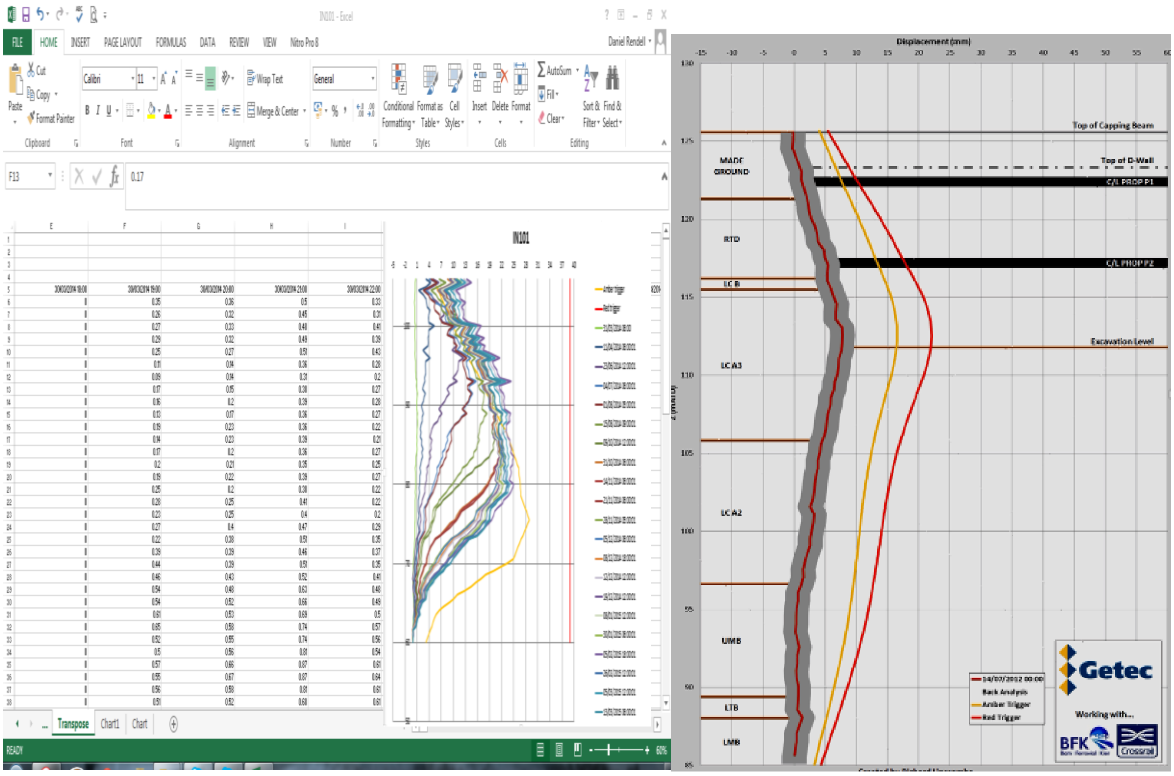


Figure 1 - Initial Development from Spreadsheet to Real Time App.
The plate on the left models a SAA inserted in a diaphragm wall panel at Tottenham Court Road station.

IncloView Application

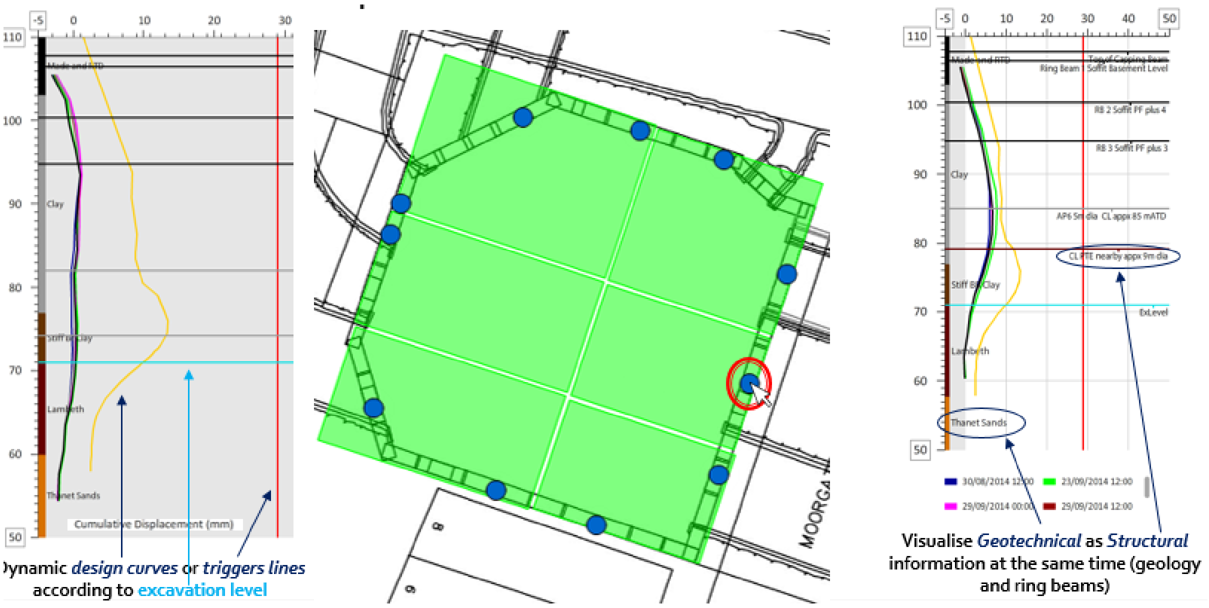
Advancements in technology have seen the rise of the smart phone/tablet generation. Site engineers now complete quality checks through tablets and construction progress is managed using BIM models.

The IncloView app loads data direct from a secure database that is prepared in advance of the works. The database is used by all users on site and therefore all data is current. Data can be imported in AGS4 if required, which assists in loading historical data.

Each time a new measurement is recorded it is first uploaded from the site logger units to the cloud before being downloaded and inserted into the database via a set of complex routines and queries.

Typically, a site will have an excavation plan. This allows a controlled dig to take place and will take in consideration the requirement of berms or temporary shores. It is essential that this plan is adhered to and considered during shift meetings. The IncloView app allows the user to draw and shape boxes to suit and to change the depth as excavated. This depth change is registered on the inclinometer plot and will change according to the date. It is rare that excavations are carried out at the same level on the same day, so the excavation box when drawn, will pick up the closet position within its boundaries and report these levels.

When the application is opened the plan view of the site is displayed along with all instrumentation locations shown by coloured interface icons. When an icon is selected, the sensor name is displayed and the user presented with an option of which parameter to plot. This user-friendly interface allows anyone to immediately pick up the application, and begin plotting data with very little training being required beforehand.



Upon plotting a chart, the user is presented with several visualisation options. Measurement dates can be selected either manually by the user or generated automatically by the application based on a defined time interval and the most recent measurement within the database. If the manual option is taken, the user can immediately see from the date selection window which dates have measurement information available for plotting. Any dates with no data will be indicated by red text and so the user can use this to optimise their working and investigate any gaps in data without searching through thousands of lines of data. This process saves time, and provides excellent quality control.

Figure 2 - IncloView Plan View

One of the most common questions asked during review meetings when data is being reviewed is "how does it relate to the trigger levels?". The application allows for trigger levels and the design lines associated with each instrument location to be visualised alongside the data and assigned to a specific phase of the works, for example the installation of structural propping at a specific excavation level. This information is saved within the database and assigned a timestamp; this can then be accessed at anytime when carrying out post works reviews. As works progress on a project so do the conditions on site, because of this it is often required to revise the design curves assigned to specific structural elements, a diaphragm wall for example, to account for these new conditions and to ensure that the factor of safety is not compromised. By the end of a project there can be several variations of design curves, all of which need to be readily available when reviewing the historical data. When an historical date is selected within the application the database will search its records and load the correct design curves and any relevant propping or slabs that were installed at that time.

Where inclinometers are concerned, the incremental data is just as important as the cumulative data as this allows engineers to identify potential issues at specific depths within the sensor and determine if perceived movements are genuine or not. Users are able to switch between the incremental and cumulative views with the click of a button, removing the need to manually re-select data and maximising productivity.

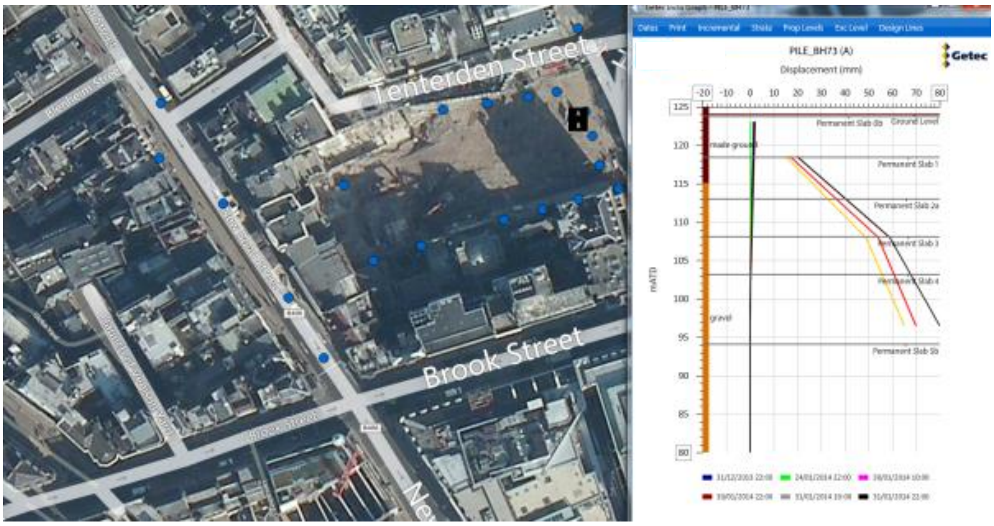


Figure 3 - IncloView Charts with Design Curves Showing Incremental Data

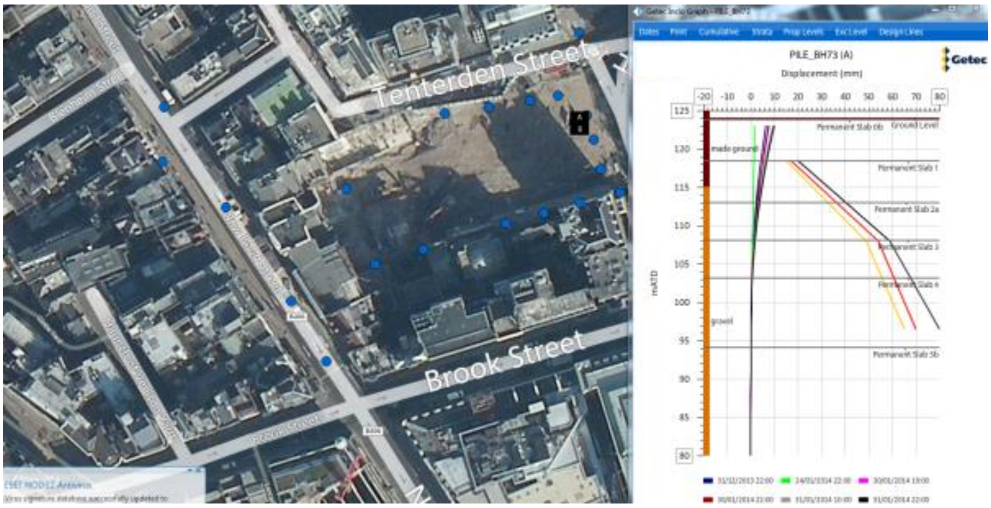


Figure 4 - IncloView Charts with Design Curves Showing Cumulative Data

Construction information and geotechnical records that are relevant to a particular instrument location can have an impact on the observed results. For example as excavation works advance the wall can begin to deform around an installed prop level. If this data was simply viewed as a curve with no additional information this could lead to confusion if the one reviewing the data was not fully briefed on the current site progress. By displaying features such as the ground strata, prop levels and slab levels within the chart view a correlation can be quickly established between elements both within and outside of the excavation and the measurements being recorded. This is further enhanced with the ability to add the current excavation level. In a similar way to the design curves, the excavation level is not a static element and as the project advances so too does the dig level. For this reason each time a new excavation level is entered within the application it is time stamped. The result is that each time a chart is produced all the critical information is displayed at the same time in one view.

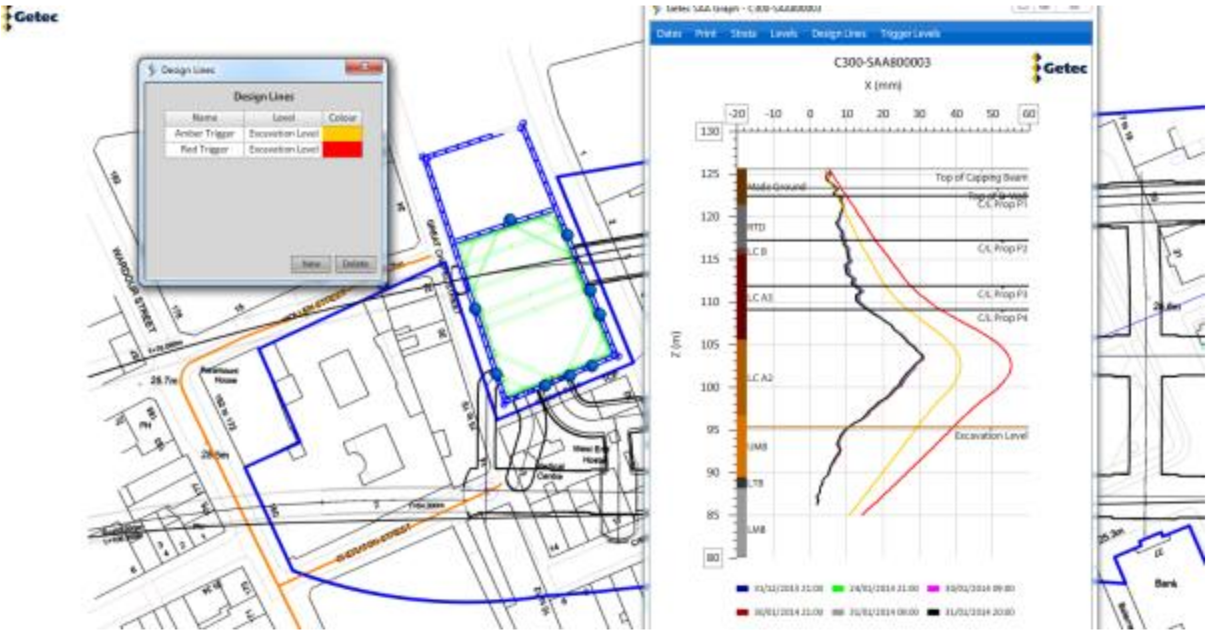


Figure 5 - IncloView Display Elements

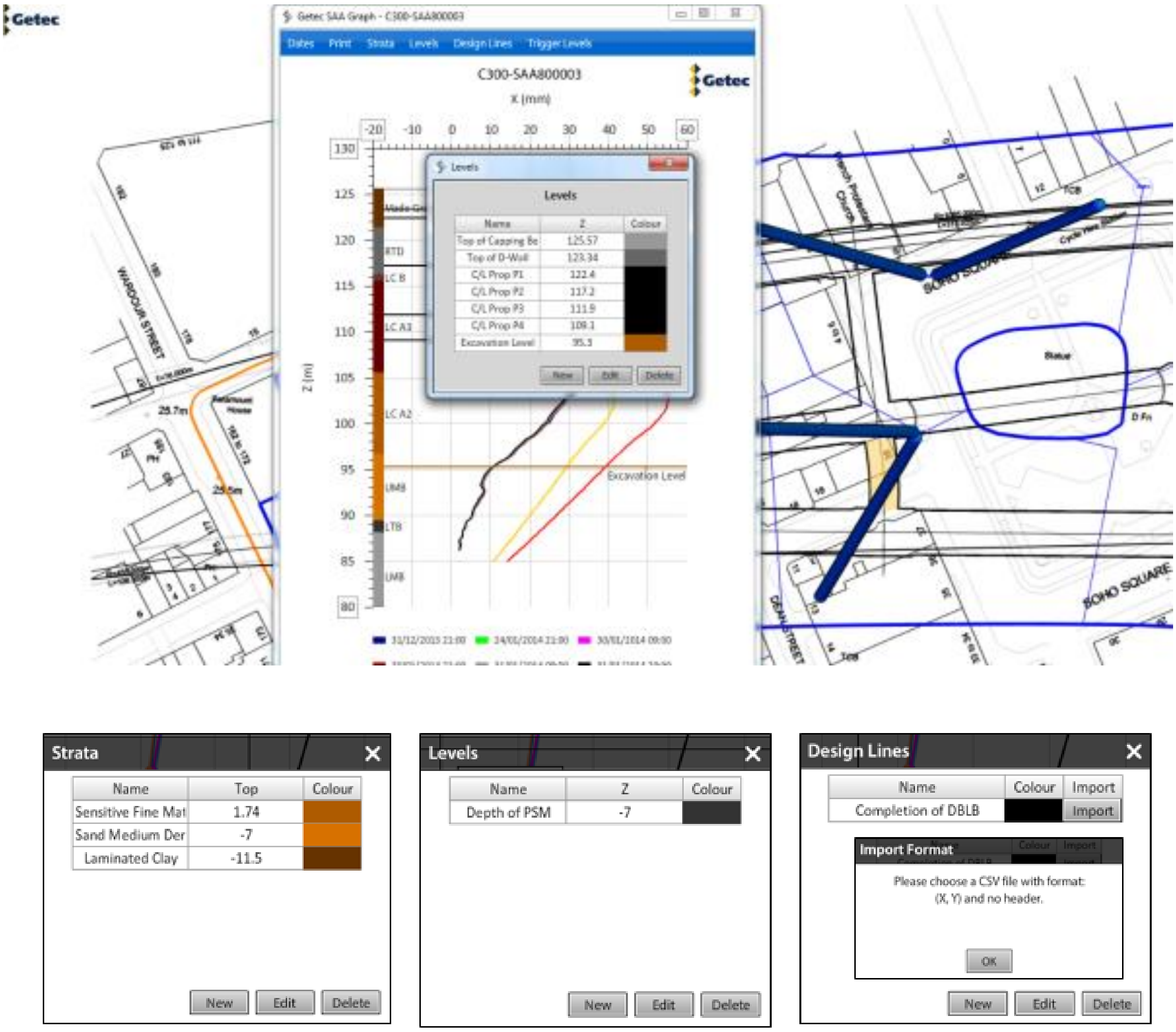


Figure 6 - Add Element Screen

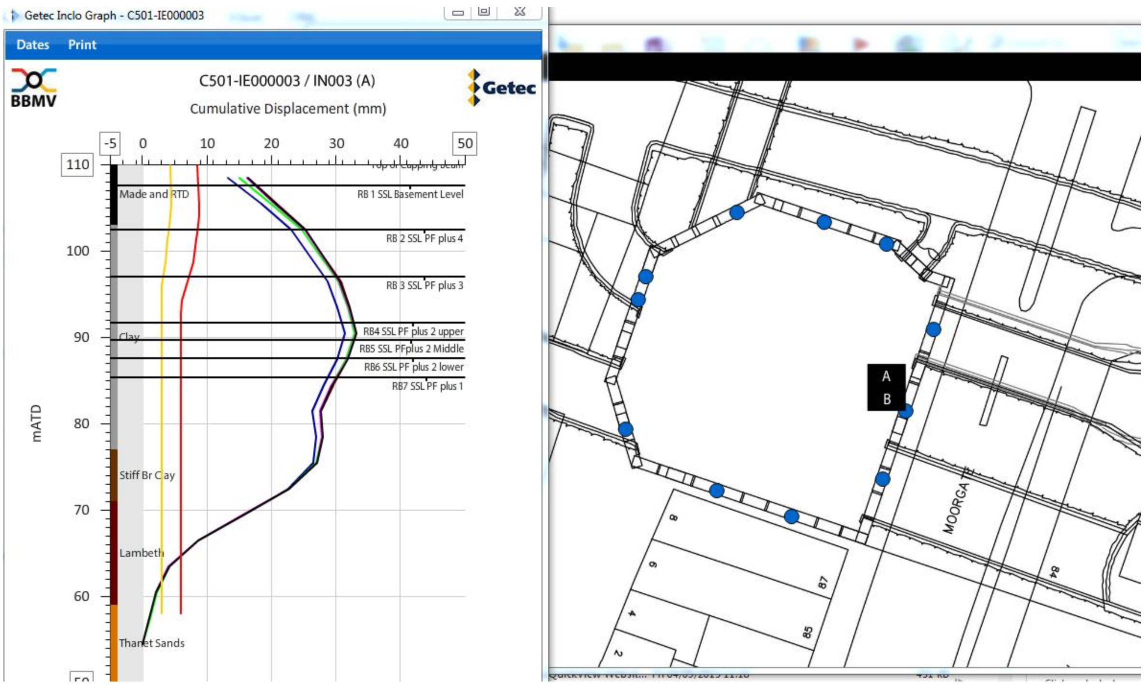


Figure 7 - Site Elements Shown Within Inclinometer Plot

Triggers and Alarms

The primary purpose of instrumentation and monitoring is to ensure the safety of others and surrounding assets. During construction works, within an excavation for example, there are often large groups working in close proximity at any one time. Evacuation procedures may take time to enforce so for this reason early warning systems are needed that will alert all concerned parties if any instrumentation begins to detect excessive deformations on site. The use of applications such as this allow for the engineers to continually review their designs and identify early indications of deformations outside of those anticipated for that phase of works, regardless of the time of day.

In the event triggers are breached the automated alarm system that is linked to the application notifies the specified mailing list and control measures can be put in place

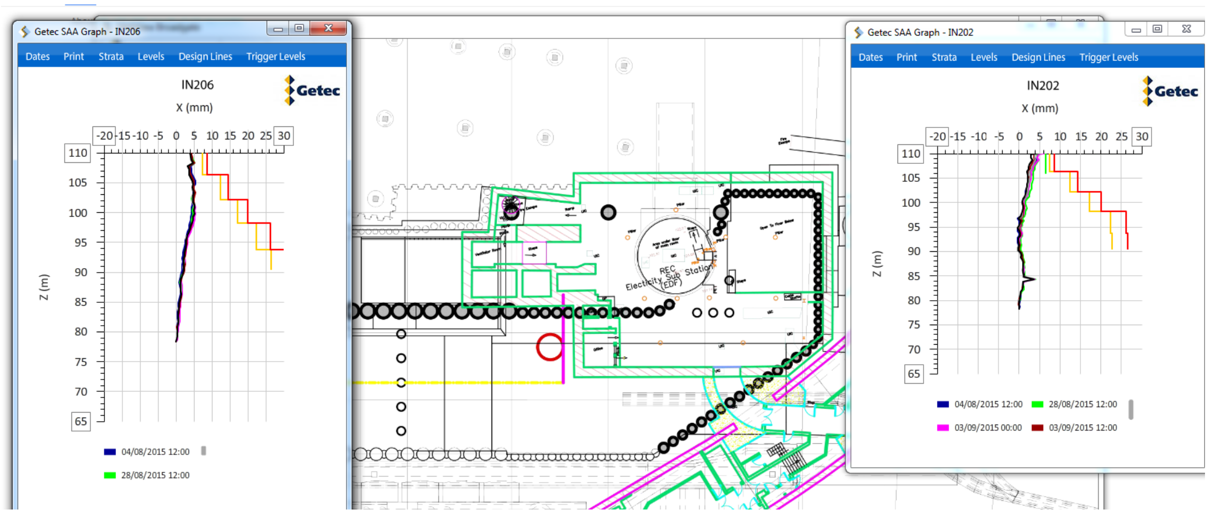


Figure 8 - Views of Inclinometers at Broadgate Station

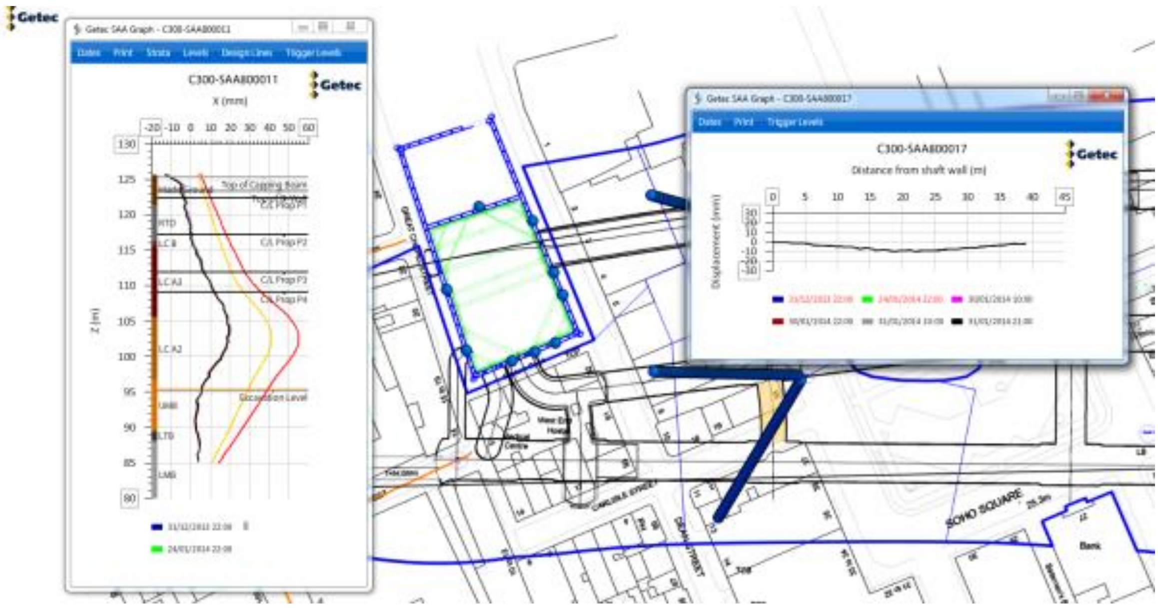


Figure 9 - IncloView Reporting Vertical and Horizontal Data

Use on Crossrail

Four different contracts have used the application extensively on Crossrail at Tottenham Court Road where the western ticket hall's diaphragm walls have been monitored by shape arrays for the last three years. It has also being used by engineers at Bond Street as part of their excavation works at the Eastern Ticket Hall where data from manually read inclinometers have been uploaded daily to the database and presented in the shift review groups. IncloView was used at Moorgate Shaft to review the accelerated dig works using In –Place Inclinometer and SAA data. Liverpool St station Broadgate Box uses the IncloView software to visualise inclinometer data.

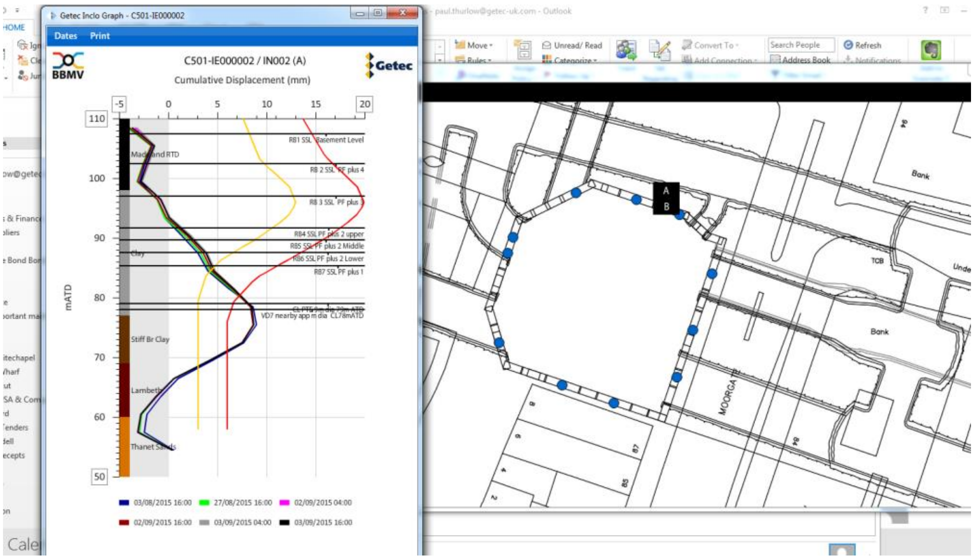


Figure 10 - Example Crossrail Project Use

Conclusion.

The use of applications such as this can also promote efficient working on site, when hold points are reached during different phases of work the site teams can review the data quickly, inspect work areas, and then continue with minimal delays to programme. In the past, this same data would need to be reviewed back in the site office, on a laptop with basic graphs and very little detail.

Miscommunication and a lack of understanding about instrumentation and how it relates to site conditions and elements can lead to delays during works. With the use of an application such as IncloView all monitoring data, ground strata, design curves, construction progress and structural elements are displayed in one view that updates automatically with minimal input from the user.

The application has been upgraded to work in a BIM environment and has been successfully used on a large rail embankment project in Australia and within a large excavation in Chicago.

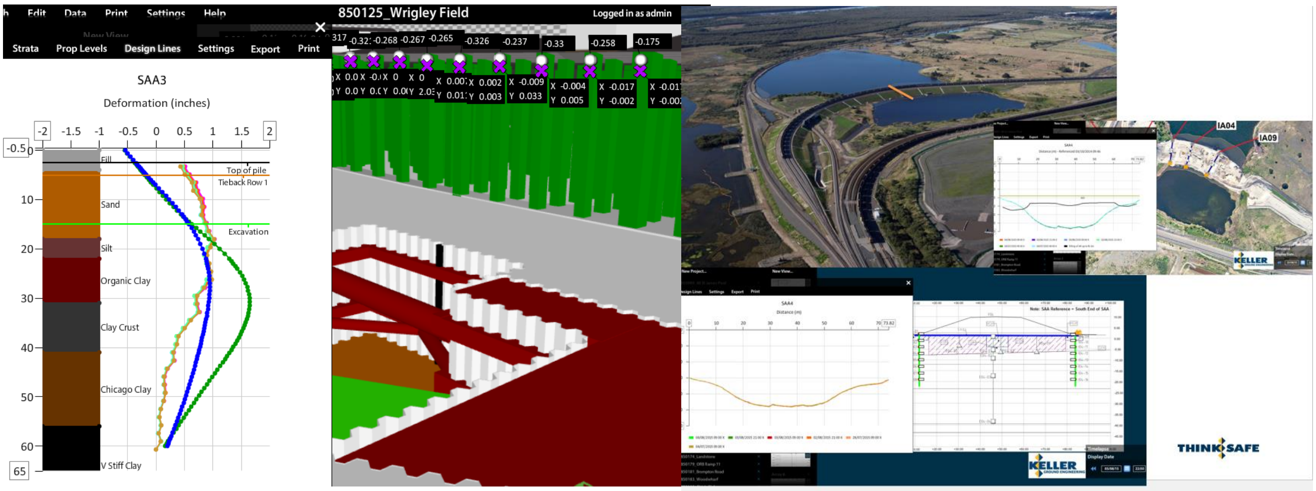


Figure 11 - BIM Integration

REVIEW OF MONITORING METHODS AT THREE CROSSRAIL STATIONS

By: Peter Hewitt, Celina Peeters, Robert Roles & Ben Brewster; Select Monitoring

Introduction

Laing O'Rourke have undertaken the construction of three Crossrail Stations; Custom House (C520), Liverpool Street (C502) and Tottenham Court Road (C422). On each of these projects the monitoring work has been self delivered through Select Monitoring. This paper reviews the monitoring undertaken at the three projects, looks at innovations introduced as part of the work and investigates ways the monitoring could be improved. The report will concentrate on the following areas:

- Modifying installation systems to suit the construction environment through the use of square section steel tubing
- Comparison between automated and manual inclinometer monitoring, looking at instrument accuracy and rate of movement.
- A comparison of Geodetic manual and automated monitoring and issues relating to maintaining an automated system
- A review of environmental impacts on automated instrumentation and how this impacts setting of monitoring thresholds by referring to strain gauge installation in concrete beams

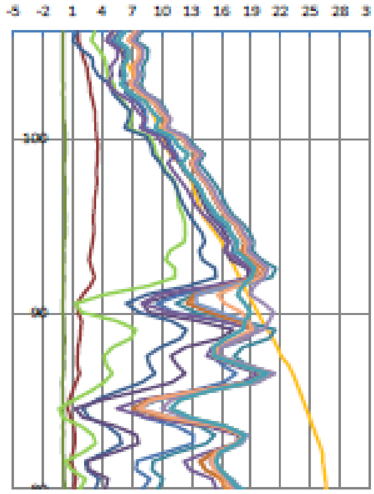
Following the review comments will be made on likely improvements to monitoring of future projects.

Improvements in Installation Methods

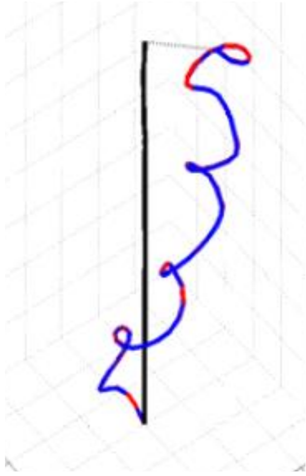
Inclinometers are widely used in tunnelling projects as one of the most accurate ways of monitoring relative movements in piled shaft walls. However, the standard method of installation, grouting a grooved plastic pipe into a reservation tube can be prone to difficulties. The use of plastic piping was originally developed as it is cheap to produce, easy to produce to tight specifications and would not be stronger than the surrounding ground when trying to monitor a deflection in soft or unstable ground. When installing an inclinometer into a pile the strength of the materials is not of significant concern and normally involves the installation of a 150mm steel reservation tube which is installed at the time of concreting the pile, which is then followed on by grouting in the standard inclinometer tube at a later date.

Two problems occur with this installation method; the first is that levels of reinforcing in piles is on the increase as the desire for stiffer piles occurs. This means that in many piling situations there is insufficient space to fit a 150mm reservation tube whilst still correctly pouring the concrete into the pile. The second issue is ensuring the inclinometer tube is correctly grouted into the reservation tube. As the inclinometer and pile get longer so the risk of gapping in the grout or snaking in the inclinometer tube increases. Snaking is where buoyancy within the inclinometer tube causes the pipe to spiral up the reservation tube and can cause inaccuracy with the inclinometer readings.

Typical Installation Detail



Impact of spiralling

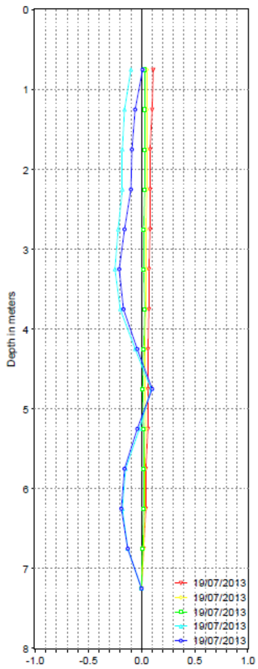


3D Plot of Spiral Install

The picture above shows a typical inclinometer installation, with black reservation steel reservation tube, orange inclinometer tube and the gap between grout filled. The extract of a an inclinometer graph above shows the impact of spiralling with a 15mm deviation over a 4 to 5m depth.

To overcome these issues and based on experience in Spanish rock quarrying, it was proposed to replace the three part standard installation with a single square section steel tube. The corner angle of the square section providing the groove channels for the inclinometer probe or IPI installation.

Trial Arrangement



Trial Plot

Typical Pile Cage Installation

Crossrail agreed to allow a trial to be undertaken by installing 3nr square section tubes against an existing wall to allow checks to be made to demonstrate that the use of the square section tubing would provide the necessary specified accuracy.

The trial of the square section steel tubing was successfully completed. The maximum recorded variation from the baseline was 0.63mm and the maximum variation between readings was 0.73mm. These values all fall within the machine specified accuracy and the CRL KX10 specification. The accuracy and repeatability of the readings was further demonstrated by the ability to identify the thermal expansion of the tubing which occurred between readings taken in the morning and the afternoon. The trial has satisfactorily demonstrated that the Square Section Steel Tubing provides a suitable material for use as Inclinometer tubing. Interestingly the greatest variation which was recorded can be seen on the plot above and was caused by thermal expansion generated when the wall on which the tubing was clamped went from shadow to sunlight. The bowing in the profile is where the tubing was clamped on the wall. Clearly this level of thermal expansion will not occur when the tubing is embedded in concrete.

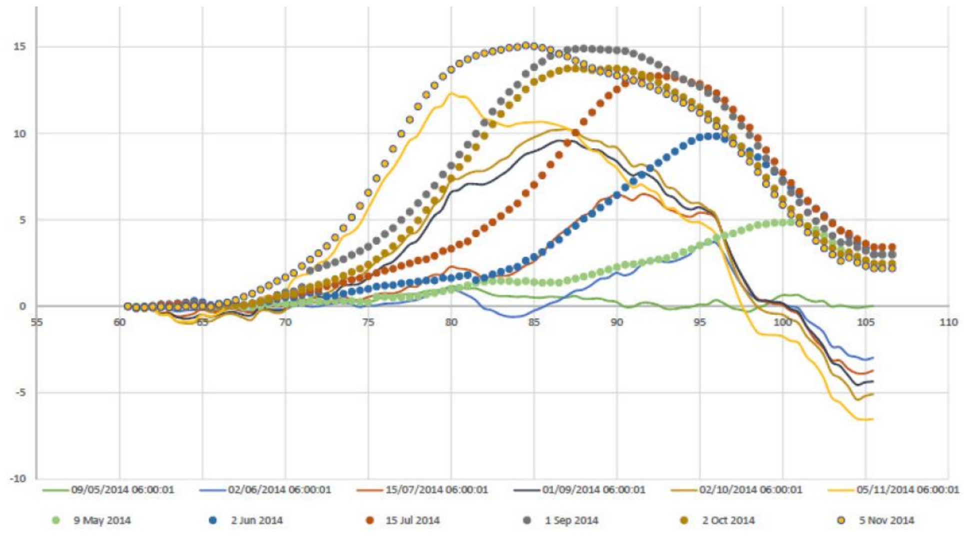
Following the successful trial Crossrail gave approval for the use of the square section tubing. Since the trial in 2014 this system has been installed in over 15 other projects. Because the installation can occur off site during the reinforcing cage construction it provides a safer and cheaper installation which in many cases is less prone to installation difficulties, so providing more accurate readings. When utilising instrumentation which has been developed for the geotechnical environment, care needs to be taken to ensure installation is modified to suit the construction environment.

Manual and Automated Inclinometer Readings

The Blomfield Box at Liverpool Street was excavated to a depth of 43m and was constructed using top down construction techniques, with an excavation depth on average of 5m between levels. Both automated and manual inclinometers were installed in the shaft.

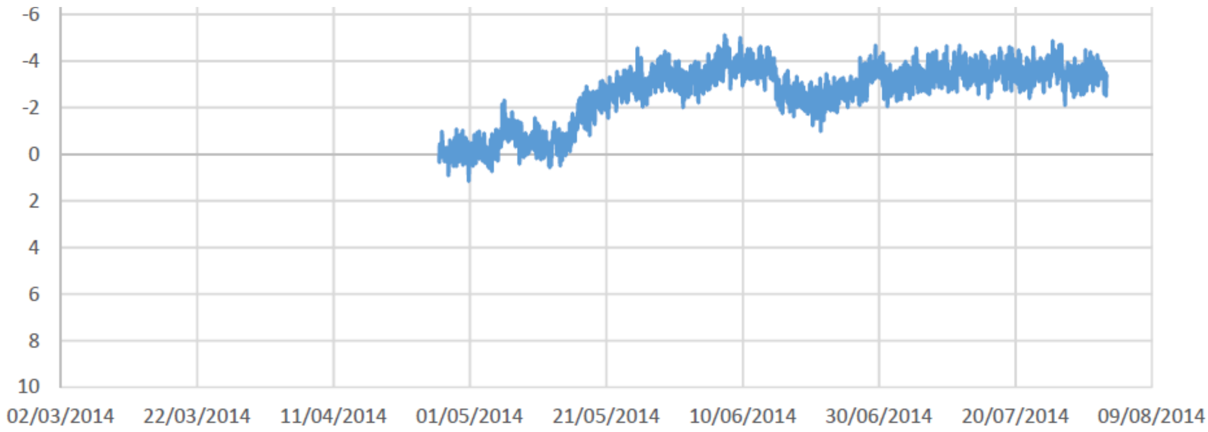
Shape Arrays were used to provide the automatic inclinometer data. Providing data on an hourly basis (reduced from an originally specified 15min cycle). Manual monitoring was undertaken in inclinometer tubes in adjacent piles using a standard manual inclinometer probe. Manual readings were taken at the beginning and end of each level dig.

The maximum recorded deflection from the inclinometers during the shaft excavation was 22mm. The maximum rate of deflection during the excavation was 0.5mm/day. A comparison of manual and automated results is given below.



A number of observations can be made on these results. All of which come with the benefit of hindsight.

The automated inclinometers were found to have a recorded instrument error of +/- 2mm over a days recording. This was within the accepted instrument error as specified and based on an up to 51m length is not unacceptable. However, when the movement between 5m dig levels was generally less than 5mm over the 2 week dig period, this meant that early in the excavation a significant amount of effort was expended in looking at instrument error.



Typical Inclinometer Fluctuation

These movements were at or below predicted movements so the rate and extent of movement was not unexpected. During excavation work data was initially submitted showing hourly results, this quickly was reduced to data every 6hrs and then towards the end of the dig this was reduced to data from once a day. These changes were made to allow a clear view and understanding of the data.

Had the automated monitoring system been replaced with daily manual inclinometer reading the cost would have been about equivalent. However, if the manual monitoring was reduced weekly during the no dig periods and daily during dig periods the cost would have been reduced by three quarters saving several hundred thousand pounds on this one installation alone. Just because automated monitoring can be done, doesn't mean it should and the frequency of monitoring should be matched to the expected rate of movement with the accuracy of the automated systems should be taken into account.

Comparison of Automated and Manual Geodetic Data Collection

With the increased demand for geodetic monitoring on construction sites, particularly those which interface or pose an integral threat to nearby buildings, services or infrastructure, the use of automated solutions is growing in relation to its traditional, manual counterpart. One such solution that has experienced a distinct increase in popularity with the progression of the Crossrail project is the automated total station – or ATS – system.

ATS systems facilitate the remote control of either a single, or multiple networked total stations, from a central control. The instruments are fixed in position and observe distance and angle measurements to survey prisms installed at pre defined locations, with a view to determining trends of movement over time.

The appeals of such a solution become evident when the limitations of a similar, manually operated monitoring scheme are considered. In theory, when establishing an automated monitoring regime, the only time a technician should have to enter the work area is when setting up the instrument and undertaking essential maintenance. From this point, every connected total station can be remotely – and automatically – operated by a control computer off site. Essentially, a surveyor who may have in the past spent a long time on site measuring points can now observe and operate an entire network of total stations without leaving the office. In turn this has health and safety benefits, and the potential to reduce costs for a large scale monitoring programme.

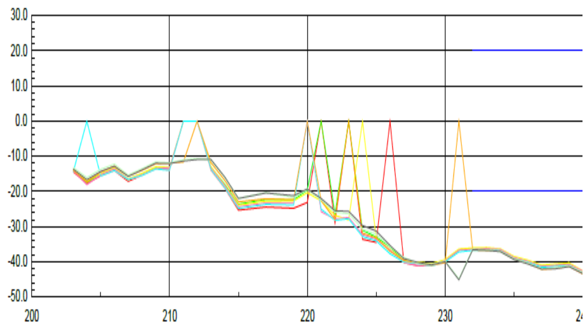
Furthermore, an automated system allows for 24/7 monitoring irrespective of staffing constraints, site limitations and even – in most cases – the weather.

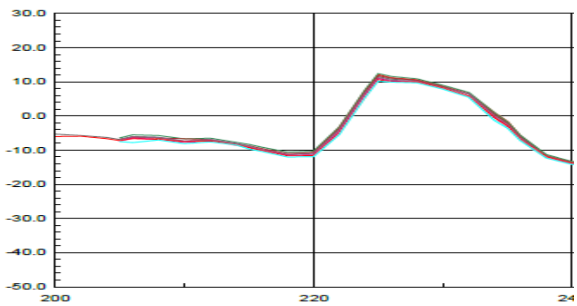
These are a few of the many reasons that design consultants have chosen to embrace ATS systems for their projects on Crossrail, which incidentally includes the largest network of automatic total stations in operation in Europe, at Paddington Station, however it is not necessarily the case that automation is always the best option.

One case that highlights this fact well is the monitoring of the Docklands Light Railway track at Custom House. The information gleaned from this system, which is actually a small part of a much larger network of ATS's, has been used to determine the effect of nearby construction works at Custom House station on the track and structural station elements. Due to the safety critical nature of the scheme, trigger values were established for alarms relating to horizontal and vertical movement of all elements and relative movement of adjacent track points (cant and twist). As a rule, any breached trigger value had to be treated with the utmost attention by all parties involved.

During the course of the construction programme, triggers were breached quite regularly, and each time had to be investigated. In most cases it was found that the recorded displacement values were not indicative of true movement, and were instead reflective of the numerous sources of error evident in such a system. Mostly these errors were introduced away from the site location, and transferred in via the network. These errors included damage or poor quality of reference prisms – leading to inaccurate resection, debris and dirt on the face of monitoring prisms, trains blocking sight lines to reference prisms and excessive vibration caused by railway traffic. For these reasons, during works that presented a particular risk to the integrity of the railway, the ATS system could not be relied upon for purposeful readings, and a more traditional approach was adopted.

Given that these works could only be undertaken during times in which trains were not running, it was possible to have a surveyor perform manual readings to fixed points on the track and platform using a digital level. The frequency of these readings was determined prior to the commencement of the works, and included both an initial reading prior to work beginning and a closeout reading following completion of the works ready to hand the site back to DLR. In practice, this methodology allowed for the specific monitoring of a key location, at a frequency sufficient to detect excessive movement before posing any risk to site personnel or the public. No points were missed out, and a number of check points were established to confirm the expected accuracy of the survey, with +/-0.2mm repeatability at each instance of the survey.





Automated Monitoring Graph | Manual Monitoring over the same Chainage

The example above is of a comparison between Automated and Manual monitoring of the same section of track is given below. Ignoring the difference in baseline starting deflections, what is highlighted is the number of "spike" readings which can be seen on the automated graph, which is absent from the manually derived data.

Overall at Custom House no movement of the track could be determined from the Crossrail Station construction. Any movement which was observed was down to maintenance work undertaken by DLR, including Kango packing the rails.

Clearly, ATS systems have many advantages over manual methods of monitoring, however as with this example, there are occasions in which automation may not be the best solution. Or perhaps when a system is referred to as automated the extent to which manual maintenance is required to continue the automated operation is often underestimated. With the added benefit of specificity, immediate (on site) error checking and the ability to mobilise quickly, manual monitoring should always be a consideration.

Strain Gauge Monitoring and Thresholds

At C502 during the construction of the Blomfield Box concrete beams were installed at each level. This consisted of nine permanent beam locations, site installed concrete embedded strain gauges in clusters of four per position. The gauges were arranged with one in each corner of the concrete member allowing bending strain to be eliminated by averaging the individual measurements. This gives a good accuracy for measuring axial load.

Monitoring data for the reinforced concrete props was collected from the moment the concrete was poured. Raw data included the gauge temperature and the vibrating wire frequency. The post installation strain was calculated according the characteristic performance for the type of strain gauge: $\varepsilon = k(f-f_0)$ where k is the gauge factor. The assumed load can then be calculated by applying the stiffness of the concrete and area of the prop.

Observations of the data showed interesting behaviour related to the R.C. props, which needs to be taken into account to obtain accurate information about the load. The following four effects are identified as requiring specific attention, in order to correctly interpret loads from strain gauges readings.

1. Temperature response of the system.

Vibrating wire strain gauges are temperature sensitive in a predictable way. When the vibrating wire heats up, the length increase, tension drops and the frequency falls. This will appear as a compressive strain to the gauge. In addition, as the prop temperature increases, there is a tendency to push against the pile wall, inducing some real axial compression within the prop as the expansion is restrained. This would be invisible to the strain gauges, as there is no accompanying strain. In the real system, the walls are flexible and therefore the props can expand to some extent, which will appear as tension to the strain gauge.

2. Temperature differences within the prop.

The prop itself has a considerable cross-sectional area, and therefore sizable temperature differences across the section. The strain gauges are located at the surface, and with no information about the average temperature of the member, it is difficult to accurately measure thermal strain and loads. When the concrete is curing, the temperature difference will be the largest but also particularly sunny days can affect the accuracy of the readings.

3. Changing stiffness of the prop.

When the baseline readings from the strain gauges are taken, the wet concrete has a negligible stiffness. As the concrete gains strength, its stiffness also increases. This early strain which occurs in the early days is incorrectly converted to a higher load, by the assumption of a constant modulus elasticity applied within the calculation.

4. Concrete shrinkage.

Concrete undergoes fairly rapid autogenous shrinkage (±40$\mu\varepsilon$ in the first 10days; increasing to 75$\mu\varepsilon$ by 3 months). This shrinkage will appear to the gauge as an additional compressive strain, however due to the restraint by the pile wall, we know that the shrinkage will actually induce a tension force to some extent or reducing the compressive force. The prop size can easily enhance the magnitude of shrinkage and show easy a 1500kN “phantom” compressive force in the first 10 days. Drying shrinkage also occurs at a slower rate of ±25$\mu\varepsilon$ over 3 months, which adds a further 3500kN compressive load.

The four effects described will have a substantial influence on the accuracy of the measurement and in combination, the overall effect is complex and difficult to correct or compensate for without an independent way of measuring the load (e.g. using a jack).

The trigger levels at Liverpool Street were set on the expected loads from the WALLAP (analysis) model, which considered all construction and used soil properties derived from the most likely ground conditions encountered. It should be noted that the strength safe working load of the props far exceeded the trigger values set.

The above example shows the Axial Load in Red and the Temperature in Blue. It provides a clear indication of the change in Load as the Concrete cools and hardens and then at later stages the correlation between temperature and load.

Most of stain gauge locations measured a load value out of the specified threshold levels. The main purpose of trigger values on loads in props should be to ensure safety of the structural member and should be set as a percentage of the safe working load of the member.

Several alternative approaches are recommended to avoid false alarms on load trigger values:

1. Re-base the strain gauges after the early effects have occurred, but before excavation progress. Two weeks would be a reasonable timeframe for early thermal, shrinkage and stiffness gain effects to be mostly complete, but this will more than likely have a detrimental impact on the excavation program if the props have adequate strength. Re-basing the gauge would not impact safety as the early effects are internal locked-in effects, rather than those due to external loads.
2. Set higher trigger values, based proportionally on a safe working load of the member, rather than a low expected value from a simple analysis. Consideration should be given to the proportion of SLS load to set red and amber triggers.
3. Instead of setting trigger values on load, use strains directly as an estimate of wall movements. Strains could reasonably assumed to be constant and summed up over the length of the member. The overall wall movement calculated via this approach can be cross-checked with the direct wall measurements taken from the in-place inclinometer readings.

There is an important question as to the value of obtaining and monitoring load data, at some considerable expense to the project. When a real problem with excessive retaining wall deformation does arise, it is very useful to have as much data as possible about the performance of the support system. Therefore having instrumented props is of considerable value in understanding where the deficiency in the system lies and in informing the best course of remedial action to address any such deficiency. If at the outset of the project, the deformation monitoring regime recognises this fact, it should be possible to design and specify a suitable level of instrumentation, without incurring the significant additional and ongoing cost of intensive data monitoring, while keeping much of the benefit of the data should it be needed.

Conclusion

Advancements in monitoring generated by the Crossrail Project has significantly advanced our understanding of how to monitor construction projects of this type. By looking at the monitoring across three Crossrail Station developments Laing O'Rourke and their in-house monitoring team at Select have gained exposure to most current forms of instrumentation, visualisation and interpretation. By reviewing this data a number of observations can be made:

- Installation techniques should be modified to take account of the construction operations being undertaken and the type of information which is to be collected.
- Methods of monitoring and frequencies of data collection should match the rate and value of predicted movement. Manual monitoring methods may be more accurate and less expensive than automated options once this has been taken into account.
- The environment into which an automated monitoring system is to be installed must be carefully considered to take account of the impact of obstructions, changes in environment and public/mechanical interaction. Maintenance of automated systems must be carefully reviewed to ensure that the automated systems continue to give accurate and reliable data.
- Setting of thresholds should be carefully reviewed to ensure they are meaningful, take into account of environmental influences and the actual accuracy of the monitoring equipment.

Thanks

Our thanks go to the Laing O'Rourke construction team and members of Crossrail together with C122 Asset Protection Engineers and C138 Liverpool Street Design Team.

Behaviour of a thermal wall installed in the Tottenham Court Road station box

Kenichi Soga, University of Cambridge
Rui Yi, University of Cambridge
Duncan Nicholson, Arup

Abstract

For new building and infrastructure developments, it is possible to incorporate the mechanism for heat transfer between the building and the ground through the foundation elements (e.g. piles and diaphragm walls). This geothermal underground infrastructure approach is considered as potentially cost-effective due to small additional installation cost and less occupied land/underground space. This paper describes a case study of a thermal wall system installed at a recently constructed Crossrail's underground station box at Tottenham Court Road station. The effect of heating and cooling on the short- and long-term mechanical performance of the thermal wall was investigated. Results of the thermo-hydro-mechanical (THM) finite element simulations of the thermal wall indicate that the mechanical performance of the wall requires examination of the effect of (i) strain differential in concrete expansion within the wall, (ii) variations in earth pressures acting on the wall, and (iii) soil shrinkage or expansion due to overall changes in the ground temperature after many years of GSHP operation.

1 Introduction

Ground source heat pump (GSHP) system is a technology that can provide heating and cooling to buildings and infrastructures with geothermal energy. The ground is used as a heat source for heating or a sink for cooling and the balance of the two can keep the average ground temperature to be constant for reliable long-term operation. A typical GSHP system used in urban settings consists of a closed pipe system buried in the ground and filled with thermal transfer fluid. When the fluid travels around the pipe loops, it absorbs heat from, or gives heat out to the ground. Experiences have shown that a GSHP system has the potential to save up to two thirds of conventional heating costs.

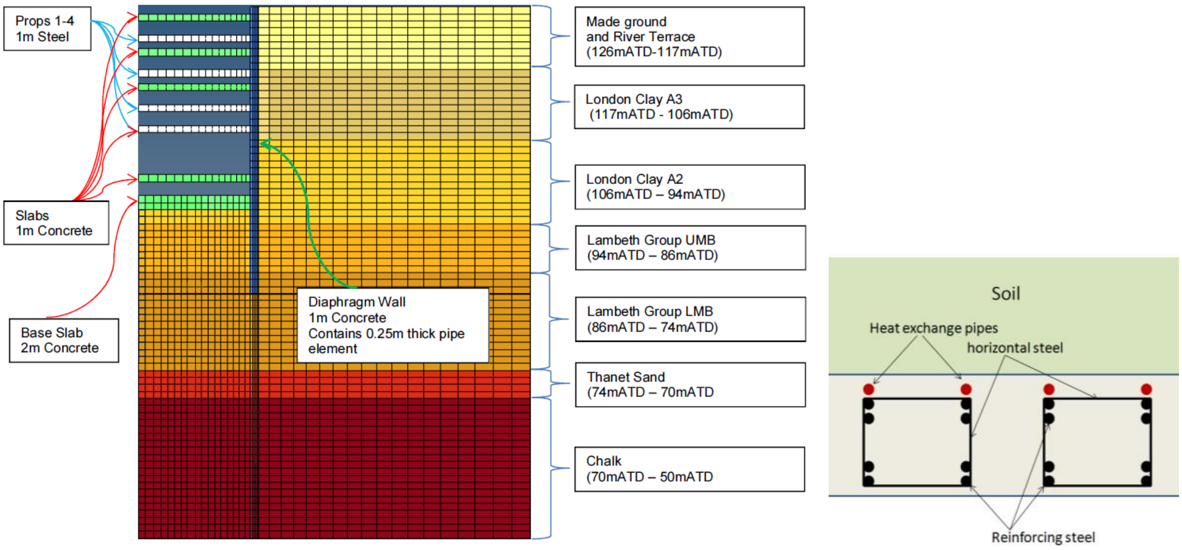
For new building and infrastructure developments, one way of reducing this cost is to install the ground source heat pump system through the foundation elements such as piles and diaphragm walls. Such "geothermal underground infrastructure" functions not only to carry a building's mechanical load but also as a heat exchanger. This geothermal underground infrastructure approach is considered as potentially cost-effective due to small additional installation cost and less occupied land/underground space.

There have been many studies into thermal piles, but limited research has been performed on thermal walls. Brandl (2006) described heat transfer in the ground, and between the absorber fluid and the concrete/soil. Temperature-induced changes in soil properties or of the thermal pile/thermal wall behaviour were also discussed, and recommendations for design and operation were given. In Brandl's case, geothermal loops were installed in the U2/2-Taborstraße metro station in Vienna, Austria. This success encouraged the Vienna Underground, which further used GSHP coupled foundations in several station refurbishments. Amis (2009) described the installation of a thermal wall at the Bulgari Hotel in London, and discussed the potential effects of thermal changes during operation. It was found that, once operational, daily loop temperature fluctuations will be considerably less than those of exposed concrete during a winter's day.

With the operation of GSHP in a wall, the cyclic thermal loadings would affect the structural performance of the wall. During winter cycle (soil cooling), the soil skeleton and the pore fluid contract. Because the thermal expansion coefficient of water is greater than that of the soil skeleton, the pore pressure decreases in undrained conditions, and the total stress acting on the wall may change as a result. During summer cycle (soil warming), the soil is heated, so the opposite trend is expected. The soil skeleton and the pore fluid expand but in different degrees, increasing the pore pressure in undrained conditions. The total stress acting on the wall may again change. More importantly the temperature difference across the wall would induce thermal strain variations within the concrete structure, resulting in an increase in the curvature and hence bending moments. The magnitude of such thermal effects on the mechanical performance of the wall needs to be examined.

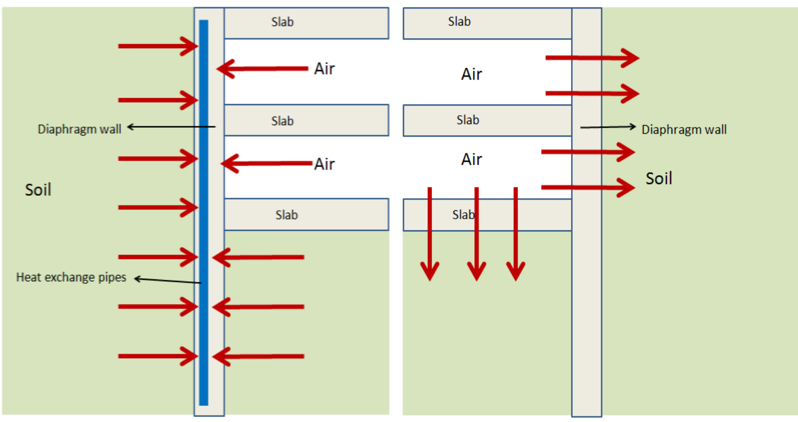
2 Tottenham Court Road station box

Tottenham Court Road station box is situated at the intersection of Tottenham Court Road and Oxford Street. The thermal walls were constructed using the 'bottom up' method. A 1 m diaphragm wall (up to 40 m depth) was first installed by excavating a trench to the required depth, as shown in Figure 1a. The absorber pipes were attached to the reinforcement cage and lowered into the trench. The position of the pipes within the concrete is usually close to the soil side (see Figure 1b), which can reduce the thermal resistance of the concrete and improve the overall thermal performance of the thermal wall. Concrete was then poured in to cast the diaphragm wall. The soil inside the diaphragm box was excavated 25 m deep and temporary props were added to support the excavation. Slabs were cast from the bottom and work proceeded upwards, replacing the props with slabs to form the station box using five levels.



(a) Thermal wall (b) Geometry of thermal wall

Figure 1 Thermal walls installed in Tottenham Court Road station box



Winter Summer

Figure 2 Possible GSHP operating mode for underground station

There are two possible GSHP operating modes for station boxes: (a) both heating and cooling, and (b) heating only (Figure 2). If a thermal wall is used for basement, then the interior side of the wall is insulated to ensure that the heat from the exchangers transfers into the soil rather than into the basement. By doing so, it can cater for both heating and cooling of the aboveground structures. For underground railway stations, there is a possibility to extract heat in winter time from both sides of the wall (station and soil) because stations often have excessive heat generated by train operations. In summer time, the GSHPs are not used and the excessive heat from the station is transferred into the soil. The heat stored in summer then can be used in winter for heating the aboveground structures. The latter case is considered in this study.

3 Thermo-hydro-mechanical behaviour of the station box

For the purpose of analysing the THM response, the station box was simplified into a 2D model to save computation time. By assuming the box is symmetrical, only half of the box was modelled as shown in Figure 1a. The top soil is 9 m of made ground and river terrace, underlain by 23 m thick stiff London clay. Below this are mixed soil layers of clay, silt and sand, called the Lambeth group. The thermo-mechanical behaviour of the soils were modelled using a non-linear stiffness gradation model with Mohr-Coulomb yield criterion. The thermal expansion behaviour of soil skeleton and pore fluid was also included in the model. The model parameters followed the values recommended for the Crossrail design. Further details of the model can be found in Rui (2015).

The box side is 16m wide and 29m deep. The diaphragm wall is 1m wide and 41m deep. In the FE model, the soil is 76m deep and extends for 80m laterally from the wall. It has four temporary props (white) and six slabs and one slab is a direct replacement of a temporary prop that was included in the original design. The absorber pipe is placed down to 40m depth, and its centre-centre distance with the wall is 0.25m towards the soil side. The bottom 1m of the wall does not have pipes.

The scope of this project involved the analysis of the seasonal operation effects of a thermal wall on the structural performance of the wall. The analysis was split into two phases; (a) the Construction Phase (Hydro-Mechanical Response) – to calibrate the governing model parameters for soil behaviour using the displacement data obtained during the construction of the wall, and (b) the Operation Phase (Thermo-Hydro-Mechanical Response) – to analyse the complex THM interactions between the soil and diaphragm wall during heating and cooling cycles in order to assess the structural response of the wall by GSHP operation. To deduce whether the THM response in the soil would affect the wall performance, the earth pressure and ground deformation in the soil and the bending moment and displacement of the wall were computed. The FE numerical simulation of the model was performed using a FE code developing at Cambridge University Engineering Department for solving fully coupled THM problems (Rui, 2015).

Construction stage

For the simulation of the construction processes, the lateral pressure ratio K_0 of the soil was set as 1, considering the wall installation effect. In addition, the water table was kept constant at zero pressure at the soil surface for simplicity. The pore pressure distribution was hydrostatic for all elements initially, and the far-field soil boundary was kept hydrostatic throughout. Drainage was allowed at the bottom and the right hand side boundaries for the fluid-soil coupled analysis. In addition, during excavation, drainage was not allowed at the excavation surface. The maximum negative pore pressure within the soil was capped at -100 kPa.

The model was calibrated first with the actual monitoring data to verify whether an appropriate set of stiffness in the soil stratum was selected for the simulation. Figure 3 shows the lateral displacement profile of the wall when the excavation was made to the bottom. The black thick line is the computed displacements, whereas the red thick line is the relative displacement from the bottom of the wall. The thin lines are the measured lateral movements of the wall from the bottom, which were deduced from the readings of the inclinometers placed inside the wall. In general the pattern and magnitude of displacements are similar between the computed results and the measured data, providing some confidence in using the model.

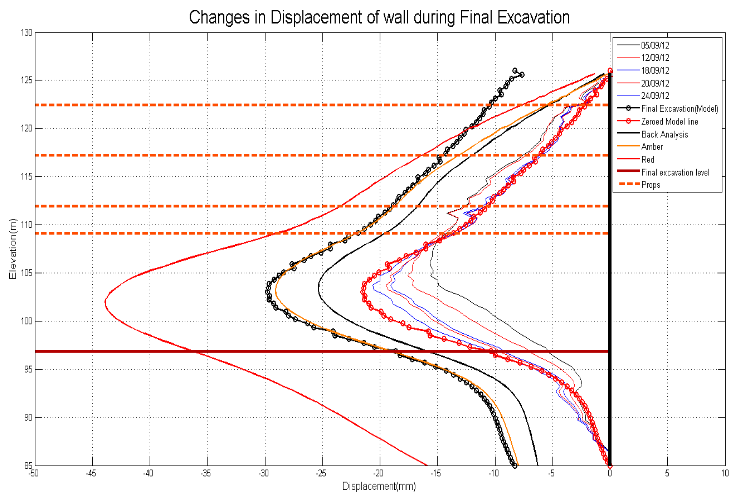


Figure 3 Lateral displacement profile of the wall after excavation

During the excavation, the horizontal total stresses reduce due to ground movement towards the excavation side, as shown in Figure 4. The largest change occurs when the final excavation takes place. This excavation stage removes ~12m of soil, which is more than twice the amount of soil removed in the second largest excavation (~5 m). The total stress on the soil side reduces to 120 kPa at elevation +103 m. The total stress in the excavation side starts at different elevations due to the removal of soil during excavation.

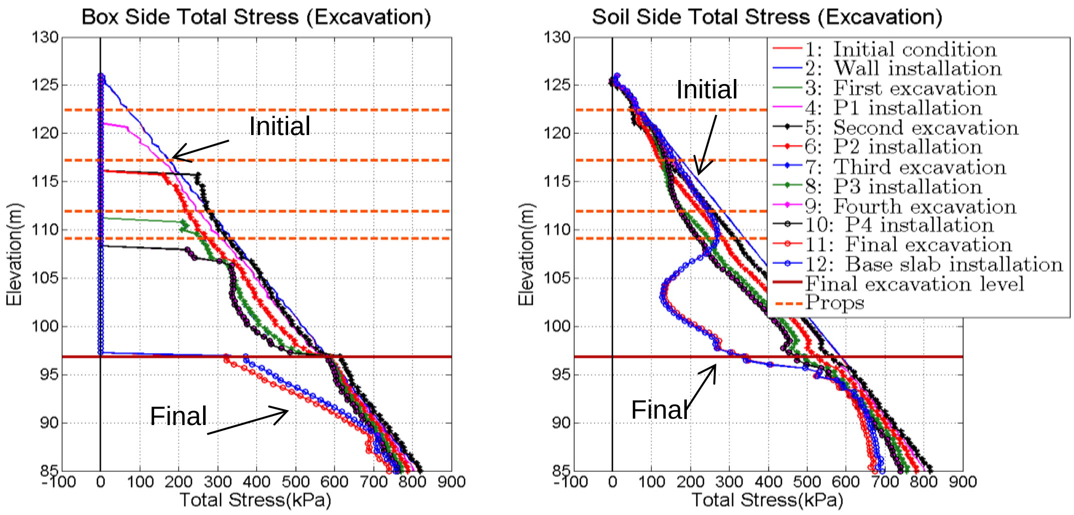
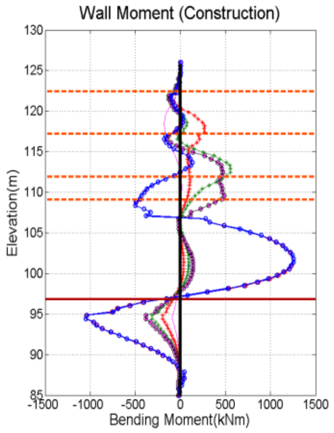
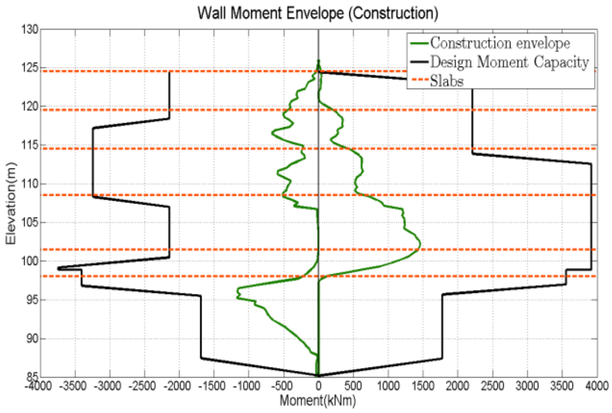


Figure 4: Profiles of changes in horizontal total stress on the excavation side during excavation (left) and on the soil side during excavation (right).

Figure 5(a) shows the bending moment distributions in the wall at different stages of excavation. The magnitude of bending moment increases as the excavation progresses. The temporary props restrict the movement of the wall as the turning points of the moment curve mostly lie on where the props are. The final excavation has the biggest displacement and curvature at +103m, so the maximum moment is at this elevation as shown in the figure. When the props are replaced with slabs, the moment does not vary greatly, except that the turning points shift from the prop locations to the slab locations. Figure 5(b) shows that the moment envelope after the construction stage (including slab installations) is well within the maximum allowable moment envelope used for the wall design.



(a) Bending moment profile



(b) Bending moment envelope

Figure 5 Wall performance during the construction stage

GSHP operation stage

To simulate the 20 year GSHP operation stage, the temperature boundary conditions were varied between winter and summer for 20 cycles. The temperature boundary conditions applied is summarised in Figure 6. The temperatures of the station box and the far-field soil were kept at 18 ˚C and 12 ˚C, respectively at all times. The initial soil temperature was 12 ˚C. The temperature in the pipes were varied between winter and summer cycles; the pipe temperature was set to be 2 ˚C and 18 ˚C for winter and summer, respectively. To quantify the effect of GSHP operation on wall performance, another simulation was conducted by having no GSHP operation for 20 years. In this case, the soil was allowed to swell by the dissipation of the negative excess pore pressure generated during the excavation, so that the long-term "drained" conditions can be achieved.

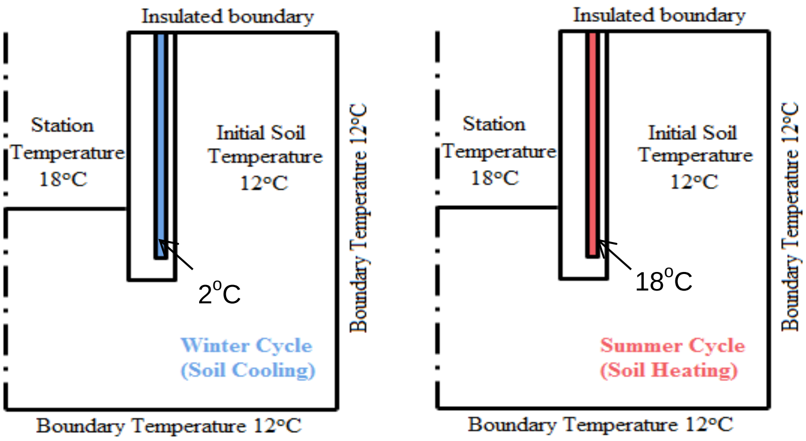
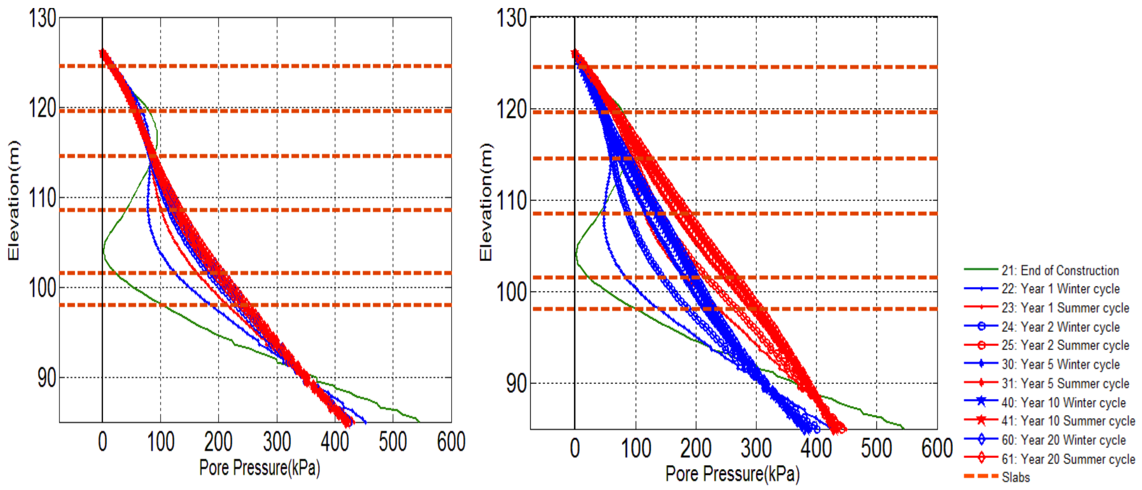


Figure 6 Temperature boundary conditions of the thermal wall (Not to scale)

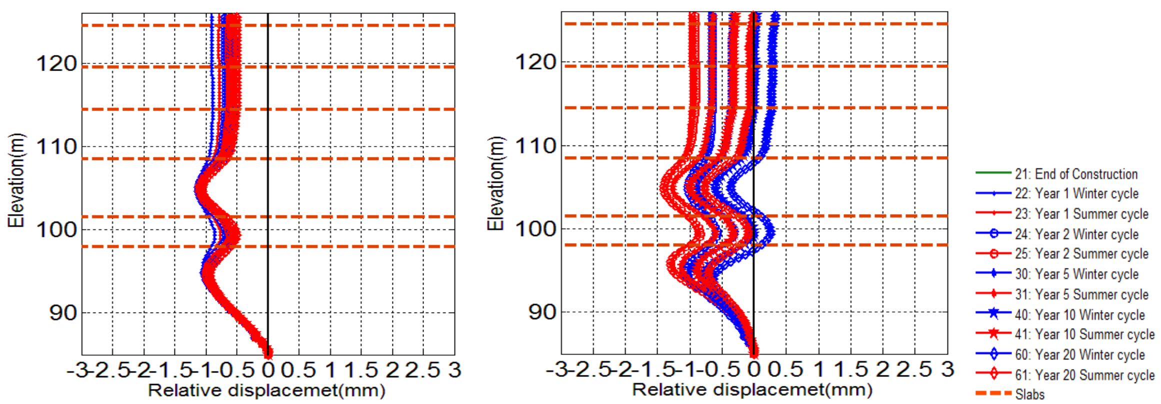
Figure 7 shows the pore pressure profiles at the soil side of the wall for the two cases: with and without GSHP operation). The pore pressures slowly converge to the initial hydrostatic distribution with time. For the case of no GSHP operation (Fig. 7(a)), the negative excess pore pressures developed during the excavation stage dissipate within the first 10 years. If the GSHP system were to be operated, the pore pressures fluctuate seasonally near the wall (Fig. 7(b)). Because the thermal expansion coefficient of water is greater than that of soil skeleton, negative excess pore pressures are generated in winter when the impermeable clay is cooled. This delays the time for the excess pore pressures in the soil to dissipate. The maximum difference in pore pressure between summer and winter cycles is 50kPa at +95m.



(a) No GSHP operation (b) with GSHP operation

Figure 7 Pore pressure changes at the soil side of the wall

Figure 8 shows the relative horizontal displacement profiles of the wall for the cases (a) without GSHP operation and (b) with GSHP operation. The bottom of the wall is taken as the datum. For the no GSHP case, the horizontal displacements do not vary greatly during the post-construction stage because there is no significant changes in the total stress. The wall gradually shifts to the right with time, which is caused by the dissipation of the excess pore pressures generated during the excavation. The GSHP operation of the wall makes the wall move in a cyclic manner; but the seasonal variation is very small (0.5 mm at the top). In winter, the pipe temperature is cold and the station side of the wall is hot, so this causes thermal strain variations in the wall, causing the wall to bend toward the soil side. In summer, the temperatures of both faces of the wall are similar, bringing back the wall toward the excavation side. Although the magnitude is small, the long-term movement of the wall is toward the soil side. This is due to soil contraction by the permanent cooling observed at locations slightly away from the wall.

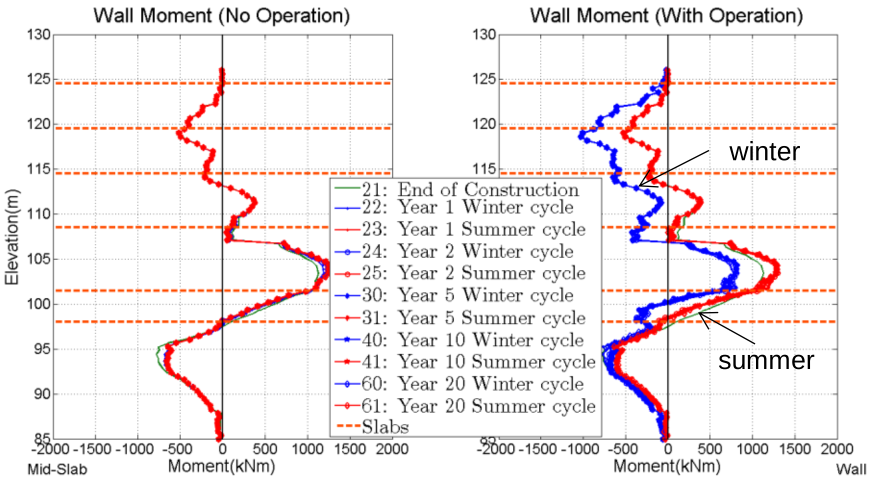


(a) No GSHP operation (b) with GSHP operation

Figure 8 Changes in the lateral displacements of the wall

The bending moment in the wall is caused by the lateral stresses from the soil and the slabs and also by the thermal stresses generated by the GSHP operation inside the wall. The pipe circuit is installed near the soil side. During winter, the coolant is 2 ˚C and the far-field soil temperature is 12 ˚C, so the soil side of the wall would shrink. The temperature at the excavation box side is always maintained at 18 ˚C. This temperature gradient inside the wall causes contraction in the soil side of the wall, inducing additional bending moment. However, in summer, the temperature across the wall is more or less uniform, which brings back the bending moment close to the initial condition.

Figure 9 shows the bending moment profiles between winter and summer for (a) no GSHP case and (b) with GSHP case. There is a clear difference in the bending moment between winter and summer cycles. Below the base slab level (+97m), the wall is restrained at both sides by the soil, so the difference between summer and winter cycles is less compared with the portion above +97m where the wall is only restrained by the soil on one side only. There is an offset of about 400kNm between winter and summer at the base slab level.



(a) No GSHP operation (b) with GSHP operation

Figure 9 Changes in the bending moment profile with time

The bending of the wall is primarily due to the thermal gradient within the wall during winter when heat is extracted. Figure 10 shows the bending moment distributions in the wall when simulations were conducted with different thermal expansion coefficient values of concrete. Results show that the change in bending moment is mainly governed by the thermal expansion of the concrete in the wall. In winter, the temperature difference is the largest, resulting in large increase in curvature and therefore moment. The thermal expansion coefficient of concrete is governed the type of binding material (e.g. cement) and aggregates used, which can be part of design consideration of thermal wall. In this case, the thermal expansion and contraction of the soil causes negligible bending moment in the wall. Therefore the thermal expansion of the soil has limited influence on the structural performance during the GSHP operation phase.

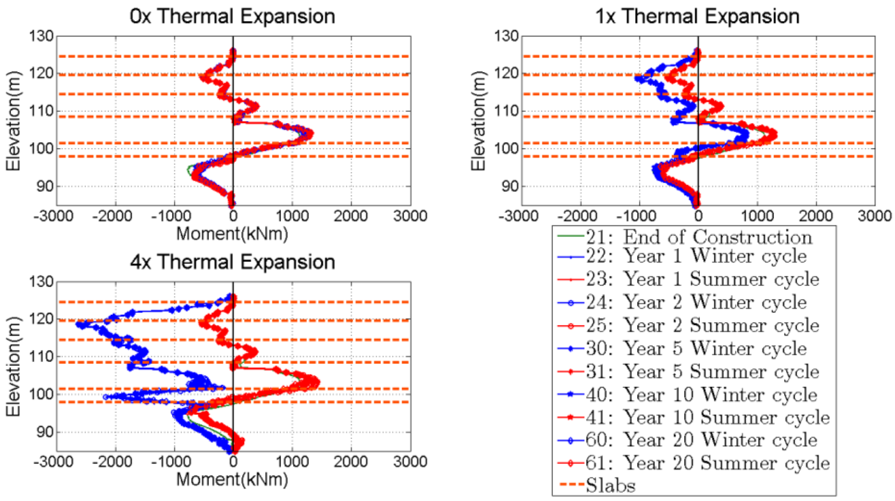


Figure 10 Bending moment profiles with different thermal expansion coefficients of concrete

The profile of maximum bending moment in the wall during different stages of the wall is shown in Fig 11. The magnitude of positive bending moment in the GSHP operation stage is lower than in the construction stage because the wall had less lateral supports during the construction stage than after construction where more slabs are added and replaced temporary props. The bending moment predicted using this FE model is well within the maximum design bending moment envelope used in designing of the wall.

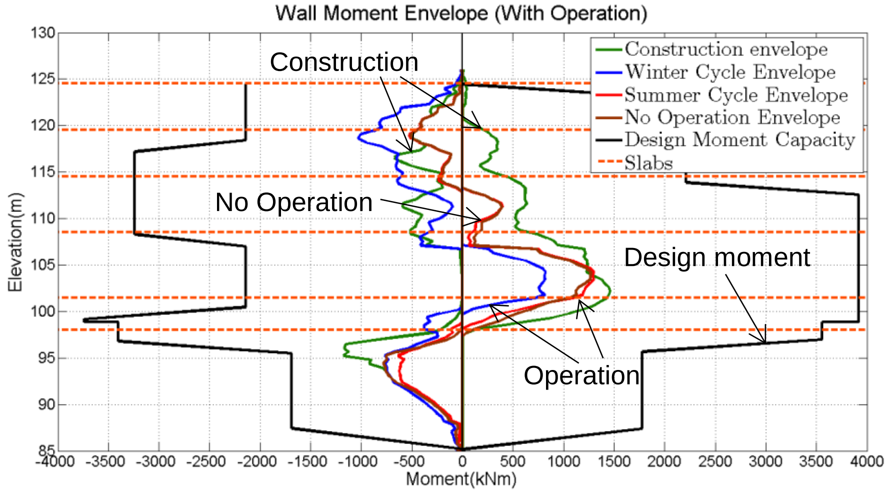


Figure 11 Bending moment envelope during GSHP operation

4 Conclusions

With no operation of GSHP, the excess pore pressures developed during the excavation stage returns to almost hydrostatic after 10 years. The wall gradually moves back toward the soil side with time due to swelling of the soil surrounding the wall. With GSHP operation, cold coolant is injected in absorber pipes in winter. Soil shrinks and small changes in earth pressures acting on the wall are observed. In summer, the ground temperature recovers by the heat flux from the station box and the heat is stored for the following winter cycle. Although pore pressure changes are observed due to low permeability of the soil as well as the differences in the thermal expansions of soil and water, the change in total stress acting on the wall appears to be small. Results show that the change in the bending moment of the wall due to seasonal GSHP cycle is mainly caused by the thermal differential across the wall during the winter. This is when the one side of the wall is exposed to the warm station temperature and the other half of the wall at the soil side is cooled by the coolant in the buried pipe. During the 20 year operation of GSHP, a small wall movement (about 1 mm) occurred toward the soil side. This is due to soil shrinkage at a permanent cooling zone located slightly away from the wall. The case study showed that the mechanical performance of thermal wall during GSHP operation requires examination of the effect of (i) concrete expansion differential within the wall, (ii) variations in earth pressures acting on the wall, and (iii) soil shrinkage or expansion due to overall changes in the ground temperature near the wall after many years of GSHP operation.

This paper describes the findings from the analysis investigating the long-term mechanical performance of the station box wall during GSHP operation. The performance of a thermal wall is very much influenced by the heating and cooling inputs from the GSHP system, which depends on the heating and cooling demands of above ground structures. Some discussion on this particular issue is given in Soga et al. (2014).

References

Amis, T. (2009) : “Geothermal business buoyant,” Geodrilling International, (3): 24.

Brandl, H. (2006) : “Energy foundations and other thermo-active ground structures,” Geotechnique, **56**(5): 81-122.

Rui, Y. (2015) : Finite Element Modelling of Thermal Piles and Walls, PhD thesis, University of Cambridge

Soga, K., He, Q., Rui, Y. and Nicholson, D. (2014) : “Some considerations for designing GSHP coupled geotechnical structures based on a case study,” The Proceedings of the 7th International Congress on Environmental Geotechnics.

EQUIPMENT & INNOVATION

1. Precast stop end verticality

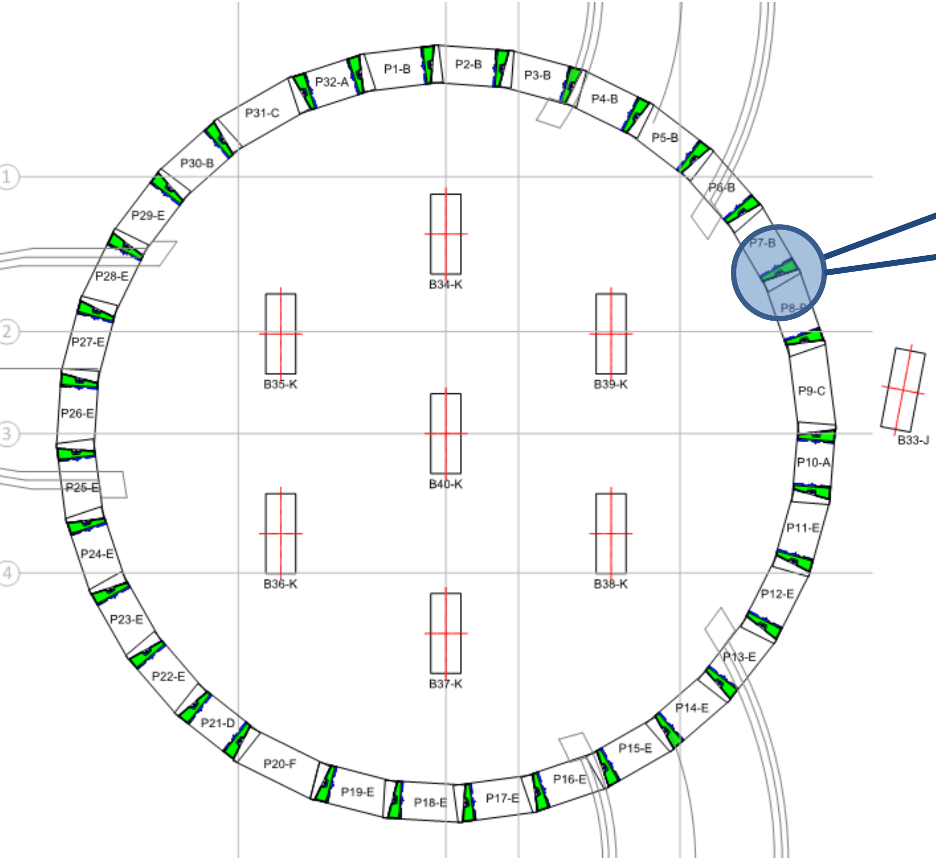
Projects:

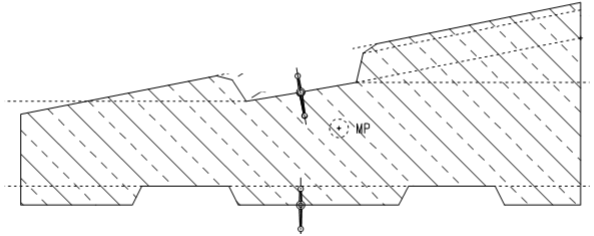
- C501 Liverpool Street (Moorgate Shaft)
- C511 Whitechapel (Cambridge Heath Shaft)

Gustav Jahnert – Project Manager

Precast stop end verticality

Panel layout





<u>Precast stop end string:</u>

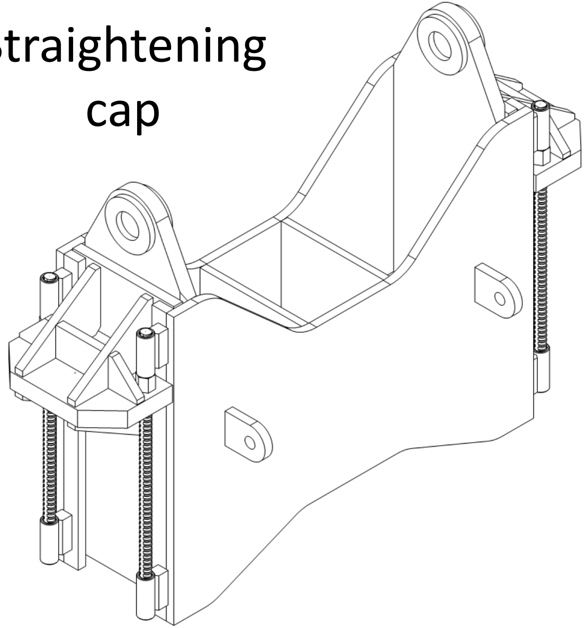
- Segmental
- L_{max} = 60m
- W_{max} = 1.5m
- M_{max} = 65 t
- Verticality: 1:300

Precast stop end verticality

Straightening of precast stop ends

Crossrail

Straightening cap



BAUER

FPS 50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Precast stop end verticality

Result

2. Dummy panel

Project:

- **C501 Liverpool Street (Moorgate Shaft)**

Gustav Jahnert – Project Manager

Dummy panel

Benefits

Crossrail

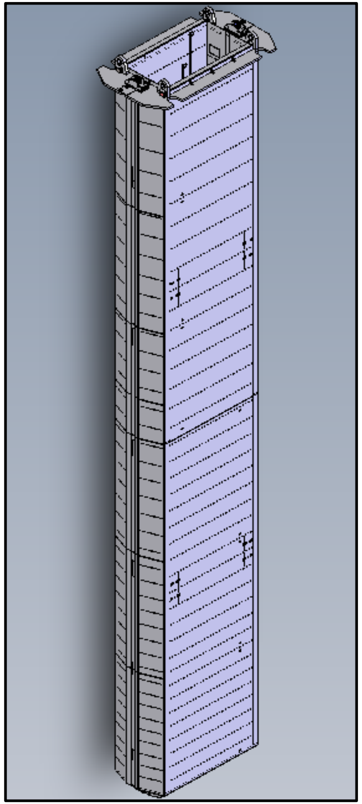
- Reduce panel construction time
- Optimise space on congested site

BAUER

FPS 50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Dummy panel

Result



3. Vibrationless pile removal

Project:

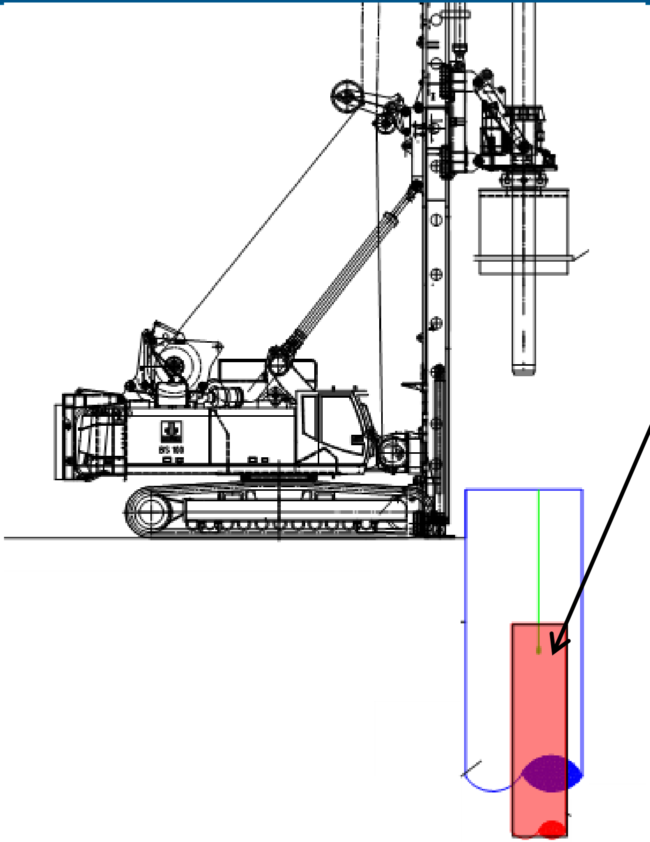
- **C501 Liverpool Street (Moorgate Shaft)**

Gustav Jahnert – Project Manager

50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

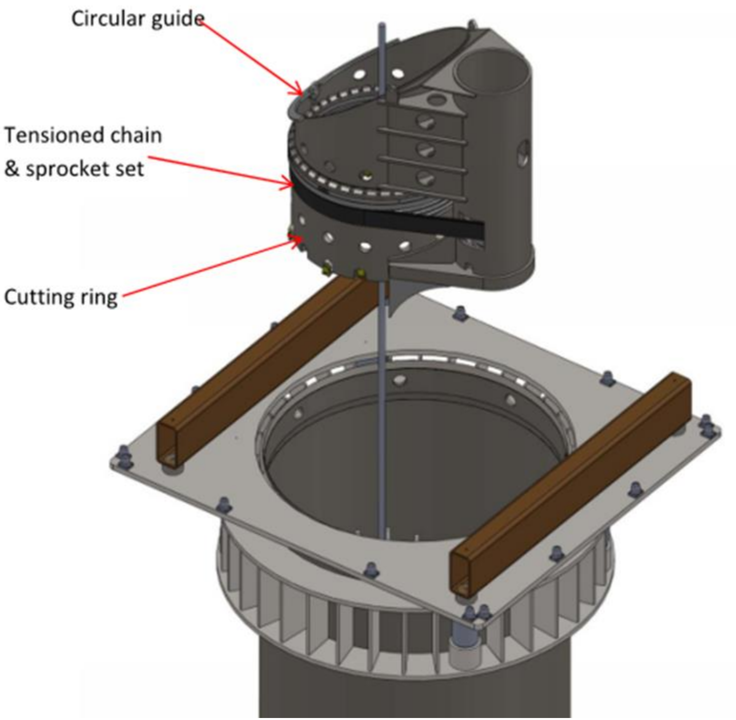
Vibrationless pile removal

Removal strategy



Vibrationless pile removal

Annulus cutter

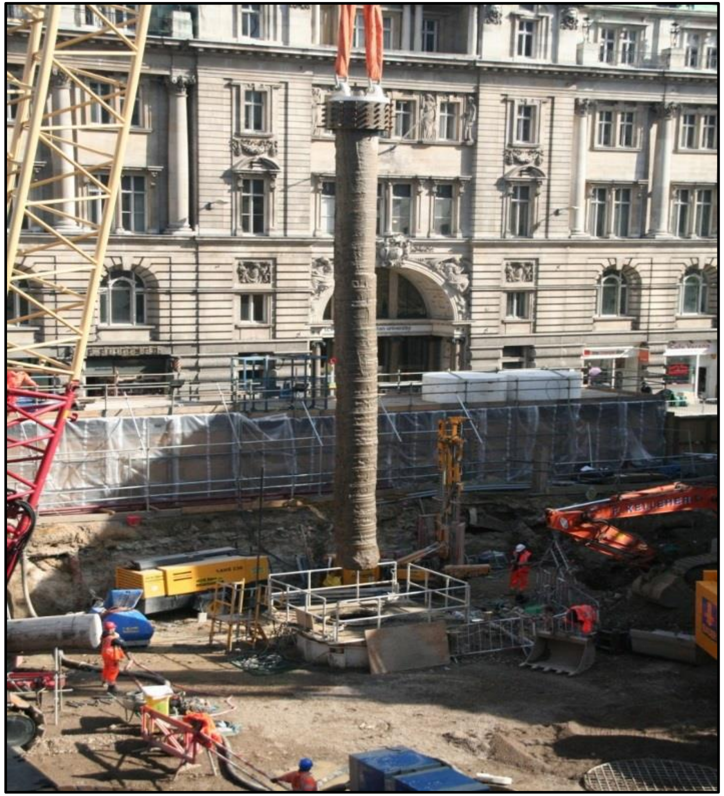


BAUER

FPS
50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Vibrationless pile removal

Result



Questions?

- Precast stop end verticality
- Dummy panel
- Vibrationless pile removal

Gustav Jahnert – Project Manager

Enhanced Capacity of Threaded Rotary Bored Piles at Paddington New Yard

David Hard, Area Design Manager, Bachy Soletanche Ltd

Abstract

The materials used in constructing piles form a significant part of the cost of the construction and have high environmental impact through a large amount of embedded carbon. The largest part of this is the concrete, which typically forms around a third of the cost. It is not normally the case however that pile sections work to their structural maximum as the pile design is controlled by the amount of load that can be transferred into the ground.

This paper will address a new technique that has been developed to improve the pile efficiency through the modification of existing rotary bored pile techniques. This is achieved by the creation of a thread around the main straight shaft of the pile, enhancing the interface with the ground. This mechanism allows a greater load to be carried than the equivalent full concreted section whilst reducing the amount of materials involved in the pile construction. Remote access inspection techniques, based upon those used in the inspection of under ream piles, are used to demonstrate the effectiveness of the threading process, combined with static load tests to validate the design assumptions.

The successful implementation of the pile method on the Crossrail site at Paddington New Yard has allowed the verification of the technique as a practical alternative to traditional rotary bored piles. The use of static load tests has been used to verify the design parameters and predicted 40% enhancement in shaft friction produced through the creation of the threads.

Introduction

The use of open bore rotary bore piles has long been a viable solution for deep foundations in stable ground conditions such as London Clay. The load that can be carried on such piles however is limited not by the structural capacity of the section, but rather by the geotechnical capacity. These piles are at their most efficient when relatively long and slim and working predominantly in shaft friction.

The shaft friction is a function of the adhesion between the pile and the clay and is controlled by the condition of the interface. There are several factors that effect this, including but not limited to, the relaxation of the bore with time, the possible softening due to seepages and ratio of length to depth.

The use of a thread on the pile helps counter act these and improve the interaction between the pile and the surrounding clay. This is due to the interface being improved mechanically by the thread which causes the two to be interlocked. The result of this is that the failure plane is pushed out to be in undisturbed ground away from the initial bored shaft and so the possible softening and relaxation effects are reduced. The effective geotechnical diameter of the pile is also increased through this process.

The potential of using threads on the shaft of such piles, as a method of increasing capacity, had been recognised by the industry but it had proven difficult to implement such a technique. This has in part been possibly due a reluctance from Clients to deviate from tried and tested methods, and in part by a belief within the industry that the process of creating the threads added complication to a construction process that relies on efficiency to return profit.

As part of an in house R&D programme, Bachy Soletanche Ltd (BSL) spent time developing a tool and process that would allow them to produce a threaded rotary bored pile. This was taken to sites and pile tests carried out to determine the degree of enhancement that could be expected. At the end of this process a patented threaded pile type, called SolThread for marketing purposes, had been brought to the point of being ready for industrial implementation.

Implementation at Paddington New Yard

The proposed piling for the new structures at Paddington New Yard offered a potential location to use the SolThread technique at full industrial scale on a live contract.

The ground conditions were suitable, with a short depth of superficial materials overlying the London Clay, and the Engineer's choice on diameters already meant that traditional open bore rotary piles were the preferred solution.

Just as importantly however was the contractual arrangement that allowed the Main Contractor to put forward alternatives to the Client, Crossrail, through the use of an OCI period. This reflected the approach of an open minded Client who was keen to investigate value engineering options, plus support the use of innovative methods to the benefit of the project.

During the tender period BSL had identified to the Main Contractor, Costain, the potential for savings through the use of threaded piles. Upon award of the contract this option was further investigated and the decision made to present it to Crossrail for their consideration.

This entailed technical presentations to both the Crossrail site team and their technical team in order to explain the benefits being put forward, and to give them the confidence in the new system.

The proposal for preliminary static load tests to be carried out, combined with the historical information from previous tests carried out by BSL, allowed sufficient confidence for the scheme to be taken to construction. Final approval of the scheme was dependant on the results of the pile tests.

Benefits to the project

The use of threaded piles meant that less concrete was required to carry the same load, compared to the conforming scheme. This of course then also meant a reduced volume of spoil to be removed from site.

Both of these reductions in volumes lead to a reduction in the amount of truck movements that would be needed on the site. This had potential benefits both to the project and to the surrounding community.

The site is long and narrow in shape with the access route running from end to end. This arrangement created a long potential interface between site workers and traffic. A solution that reduced the risk on this through less traffic had safety benefits for site operatives in addition to any environmental improvements.

A similar benefit was to be gained for the local community. The site entrance was immediately adjacent to both a bus station and a tube station as well as busy roads. This combination meant a significant number of potentially vulnerable pedestrians would be interacting with the site traffic. The reduction in the number of trucks that needed to traffic through this area would therefore reduce the potential risk to these individuals, plus through reduced emissions aid in improving local air quality. As the concrete plant used for the project was local this also produced an indirect improvement to local air quality through the use of less concrete meaning that there is less CO_2 produced by the plant.

It was these benefits that encouraged Crossrail to proceed with the SolThread alternative, in addition to the technical and commercial benefits.

Construction methodology

The aim of the development of SolThread was to increase the efficiency of the pile section so as to minimise the amount of concrete required, or to increase the load carried for the same pile size. At the same time, in order for the new technique to gain acceptance it needs to have similar or better levels of productivity when compared to existing traditional methods.

The process is based upon the normal construction technique for an open bore rotary bored pile, where a conventional 750mm diameter pile is constructed using a Kelly bar and auger to excavate the pile to the required depth. The SolThread tool is then placed inside the open bore and lowered to the bottom of the pile shaft. Once the pile threading process is ready to commence, the teeth are mechanically extended outwards to give a thread cutting diameter of 900mm diameter and the thread forming process is initiated. The thread forming procedure inside the pile shaft wall is performed by rotating the tool and at the same time extracting the tool from the pile bore at a given rate to produce a pre-defined thread pitch.

The design incorporates a so called crumb bucket beneath the cutting tool to catch the spoil produced by the thread formation, and hence reduce the amount of cleaning required at the end of the pile threading. This bucket minimises the amount of times tools are required to be placed within the shaft after thread formation and so minimises the chance of any damage to the thread prior to concreting of the pile.

Remote access inspection techniques, based upon those for inspection of under ream piles, are used to allow a visual inspection of the completed bore prior to moving to the next stage of cage installation. This forms an important part of the quality control process as it gives the opportunity to demonstrate the effectiveness of the threading process. At Paddington New Yard, the first five works piles were checked after threading using this technique to give confidence to the Crossrail site team. Based on the results that were seen this it was deemed acceptable to relax this and the rate was dropped to one in every twelve piles.

A secondary and immediate check on the effectiveness of the threading process can be made during construction when the tool is withdrawn to empty the crumb bucket. If the contents at this point do not match the amount of pile that has been threaded then it is an instant indicator of a potential problem.

The process of creating the threaded pile is protected under patent by Soletanche Bachy, and the product is marketed in the UK under the name “SolThread”.

Fig 1: CCTV unit, portable generator, Crumb Bucket, Threading Tool on Stand, Traditional Auger and Cleaning Bucket.

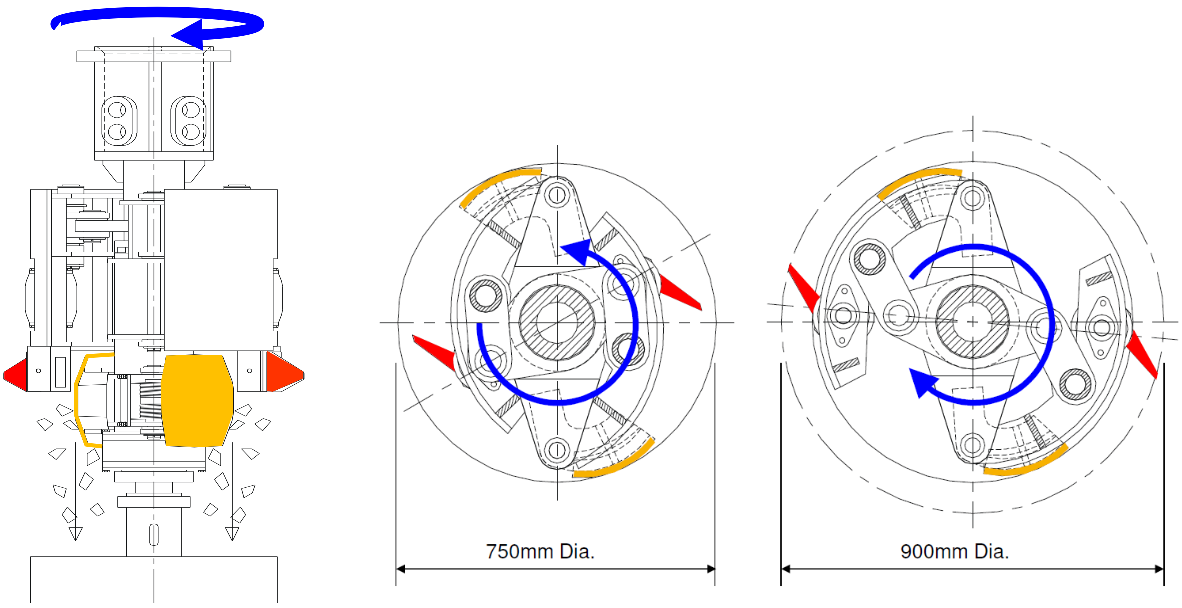


Fig 2: Threading Tool – Opening and Closing of Tool.

Design verification

A critical part of getting the alternative foundation solution accepted was the proposal to carry out sacrificial pile tests to verify the design assumptions. Due to the site geometry it was proposed to carry out tests in two locations at either end of the site. At each set up both SolThread and traditional piles would be installed and load tested.

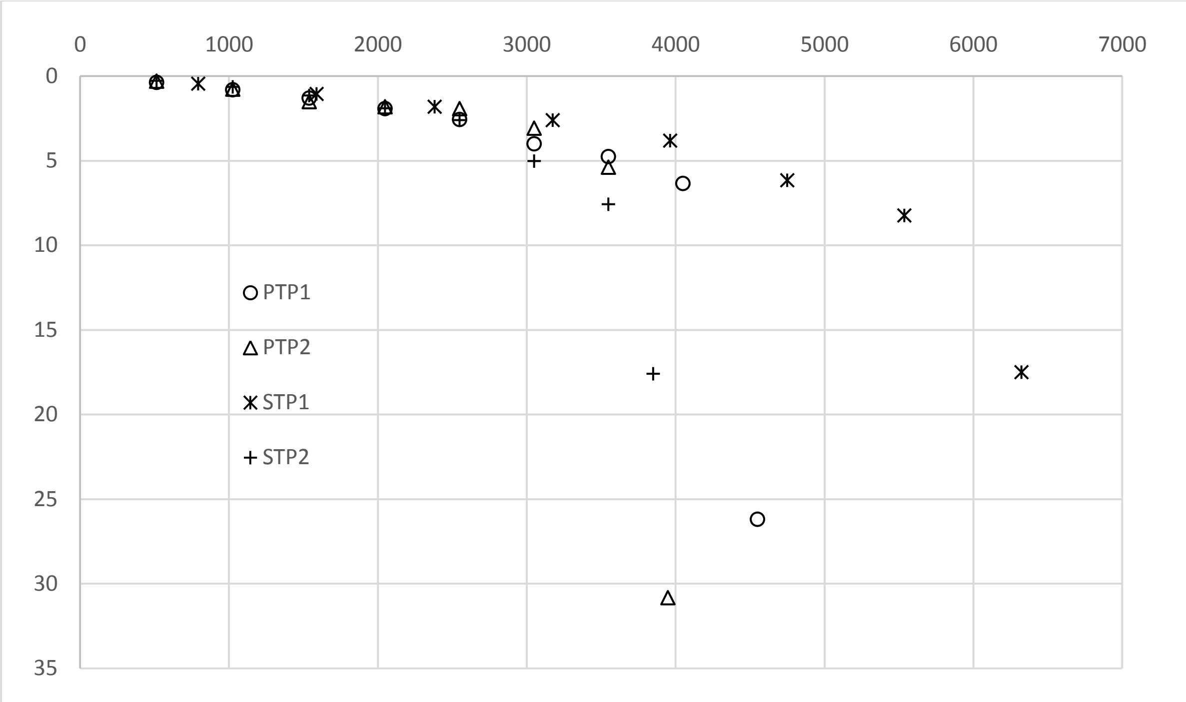
The graphs below compare the performance of the straight shafted rotary piles with that of the associated threaded piles.

PTP1: 750mm diameter, 21.50m deep

PTP2: 750mm diameter, 21.00m deep

STP1: 750/900mm diameter, 21.50m deep

STP2: 750/900mm diameter, 15.50m deep



The results show that where the threaded pile is the same length as the straight pile (PTP1 vs STP1), there is a significant increase in load carried. The shorter SolThread pile, which was designed to have the same capacity as the longer traditional pile (PTP2 vs STP2) has shown very similar performance but on a significantly shorter pile.

The design of shaft friction of a pile in clay is a function of the undrained shear strength of the clay and the interface between the pile and the ground, represented by an adhesion factor called alpha. The creation of the thread around the pile shaft meant that the traditional values were no longer valid and new ones had to be determined through the testing process. The results from previous testing carried out by Bachy Soletanche meant that the design principles had already been established but needed to be refined for this particular project.

The testing carried out has proved that the shaft adhesion factor can be increased from the usual value of 0.5 up to 0.8 through the use of the thread. This results in an increase in shaft capacity of 40%. The volume of concrete used in the thread is around 10 to 15% above the volume of the core of the pile, depending on the exact diameter used. The current tool used at Paddington New Yard was based on a 750mm diameter central core, threaded out to 900mm diameter. The table below compares the volumes of 1.0 metre depth of pile for standard 750mm diameter pile against a standard 900mm diameter pile and a 750/900mm diameter SolThread pile.

Diameter	Volume	% increase (volume)	% increase (shaft capacity)
750	0.442	0	0
900	0.636	44	20
750/900 SolThread	0.508	15	40

The design of the pile has to make an allowance however, for the proportion of the pile which is threaded, and that which is not, to reflect the fact that the bottom 3 meters over the depth of the crumb bucket cannot be threaded. The upper portion of the pile above the top of the clay which is supported by a temporary casing is also not threaded, although this may not be load bearing depending on the ground conditions. Due to the bottom of the pile not being threaded, the base capacity is calculated on the basis of the smaller core diameter and not the larger threaded diameter.

On the Paddington project the increased capacity of the SolThread pile resulted in the piles being shortened from 21.0m for standard 750mm diameter piles to 15.5m for the 750/900mm SolThread, a saving of 27% on length, and 15% on volume.

An assessment of the carbon savings using the EFFC “Carbon Calculator” indicates that this corresponds to a reduction of 10% for the SolThread pile compared to an equivalent straight shafted pile.

Lessons Learnt

The first lesson, and probably the most important, to be learnt from the project is that the use of an OCI period allowed the time for alternatives to be fully investigated by the Contractor and then presented and discussed with the Client. This meant that the benefits and reasoning could be thoroughly examined without time pressures cutting the process short before the full potential could be established.

This process still required both the Client and their Engineer to be open to the proposal for a new technique to be used on the project. It also required the site representatives from these organisations to be educated in how the new technique varied from the processes they were more familiar with.

The enhancement in skin friction of threaded piles over straight shafted have been verified through the use of pile tests. These tests also allowed the use of lower design factors in the pile design with additional savings resulting on pile length.

The initial construction process had a significant amount of check points to verify the threading had been carried out successfully, most notably through the use of the CCTV to allow Crossrail site staff to satisfy themselves as to the end result. Initial dialogue had identified however a willingness to allow this to be relaxed to a regime of occasional spot checks if the site team could be convinced of the consistency and quality of the process. A process for managing expectations on both sides ensured that the necessary requirement to prove the process and quality did not result in an unwieldy system of checks that would negate the benefits of the reduced volumes through the increase in time spent on construction.

Conclusions

The degree of enhancement in alpha value has been demonstrated through tests. As the degree of enhancement taken in the design is significant it was important that the piles performed in line with predictions for the credibility of the alternative. Further testing in other over consolidated clays, such as Oxford Clay, will supply more data on how much this effect varies with clay type and plasticity.

It is clear however that following this successful implementation there is now scope for this technique to be used on other projects. This will be both in the current diameter and also potentially in other larger ones as well.

The savings in materials used and subsequent sustainability benefits are real and of a level that allows Clients to consider the environmental benefits to their project as well as the commercial ones. This covers not just the material coming into site but also the spoil to be removed, with the associated knock on benefits that this produces.

The ability to carry the same load on shorter pile through the use of the threading means that there will be sites where the toe level of the piles will no longer penetrate into strata requiring the use of support fluid. This is a situation commonly seen in the London area where the penetration of piles into the potentially water bearing Lambeth Group beneath the London Clay can have a significant impact on the pile construction technique. If the piles can be kept up in the “dry” strata then the whole process will be quicker and the environmental impact of having to dispose of spoil classified as “contaminated” is removed.

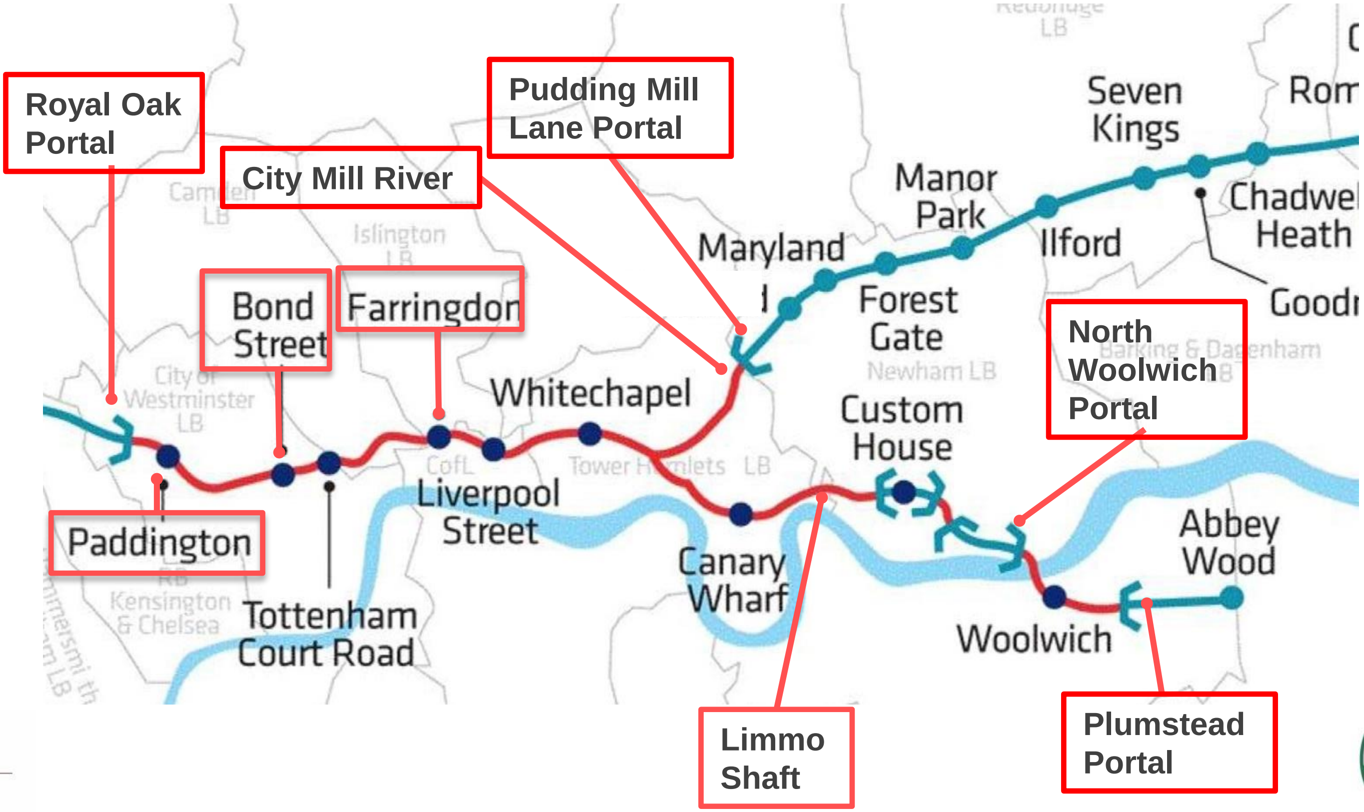
The construction process is neutral in terms of time as the increased time on the threading process is offset against time saved on the initial shorter pile bore. This is important as confirms that there is no negative impact on the programme.

CROSSRAIL LESSONS LEARNT

Delivery of Foundation Packages on Crossrail

Stephen Clarkson

Cementation projects



Cementation SKANSKA

FPS 50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Low headroom grab

Crossrail

Cementation
SKANSKA

FPS
50 YEARS OF EXCELLENCE IN
GEOTECHNICAL ENGINEERING

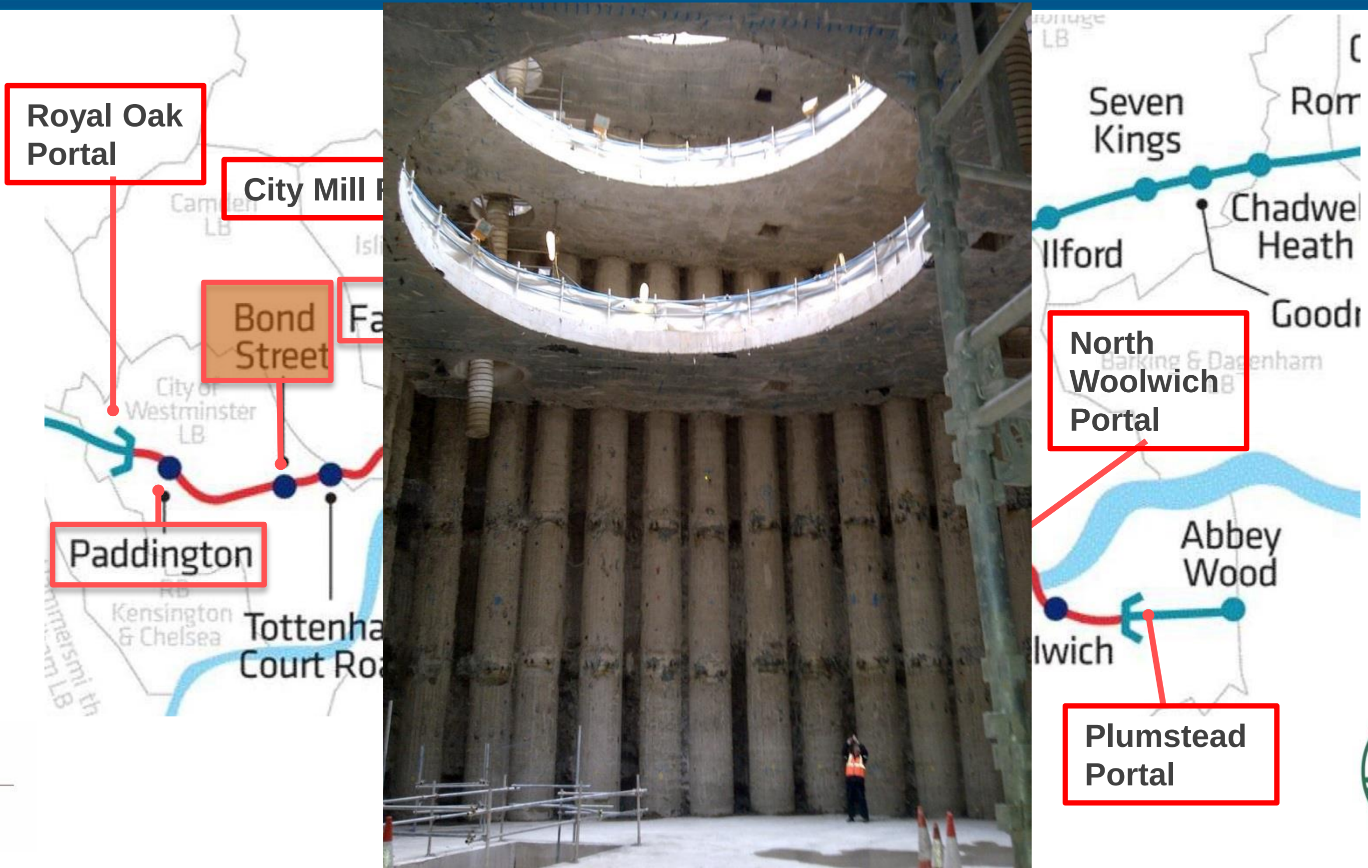
24-hour working

Crossrail

Cementation
SKANSKA

FPS
50 YEARS OF EXCELLENCE IN
GEOTECHNICAL ENGINEERING

Deep secant shaft



Low-headroom piling

Crossrail

Cementation
SKANSKA

FPS 50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

GFRP Soft-eye

Crossrail

Cementation
SKANSKA

FPS 50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Precast stopends

Crossrail

Cementation
SKANSKA

FPS 50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Site fabrication

Crossrail

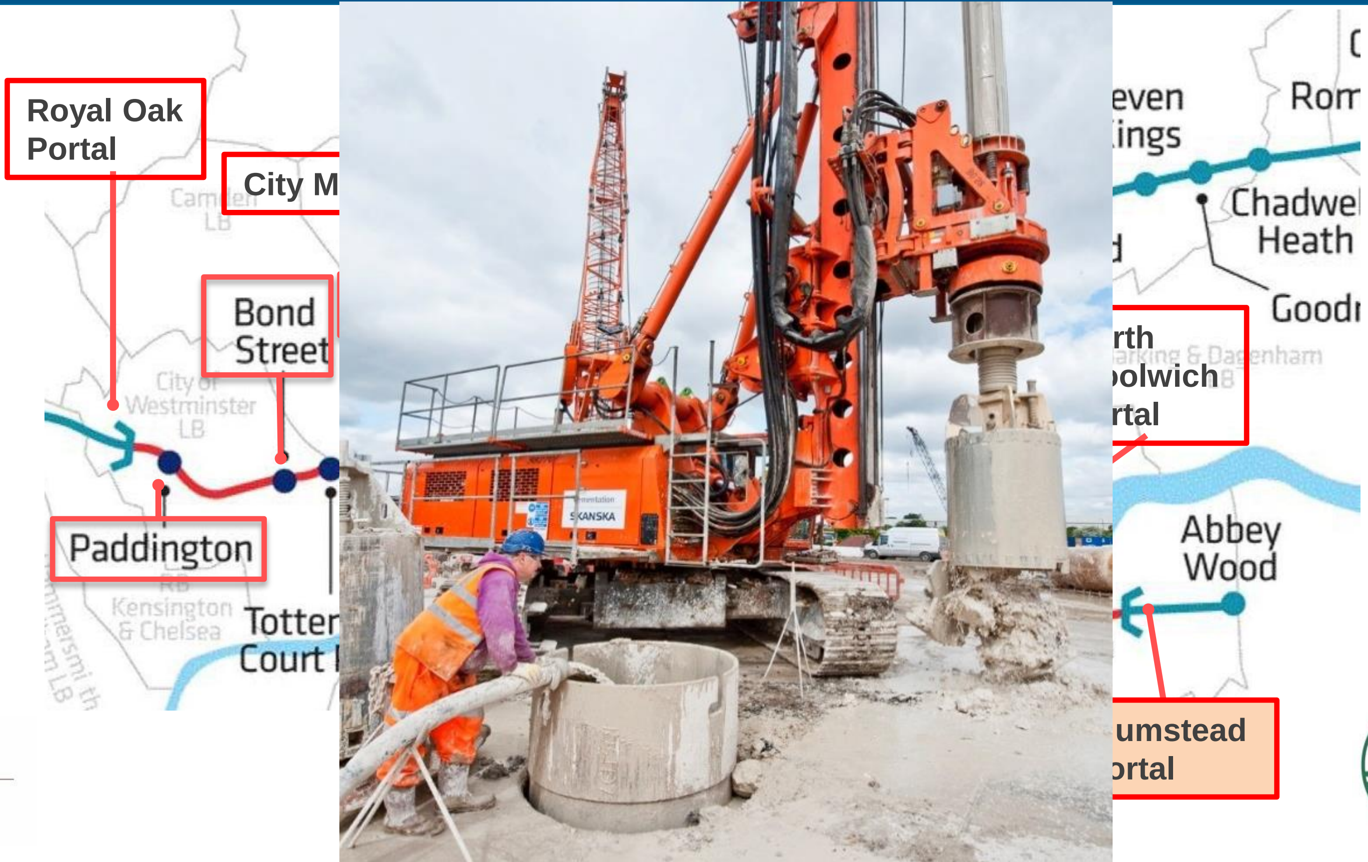
Royal
Portal

Cementation
SKANSKA

FPS 50 YEARS OF EXCELLENCE IN GEOTECHNICAL ENGINEERING

Polymer

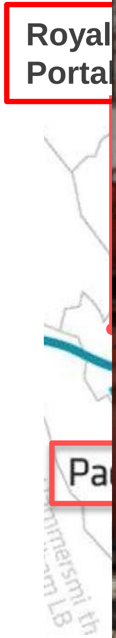
Crossrail

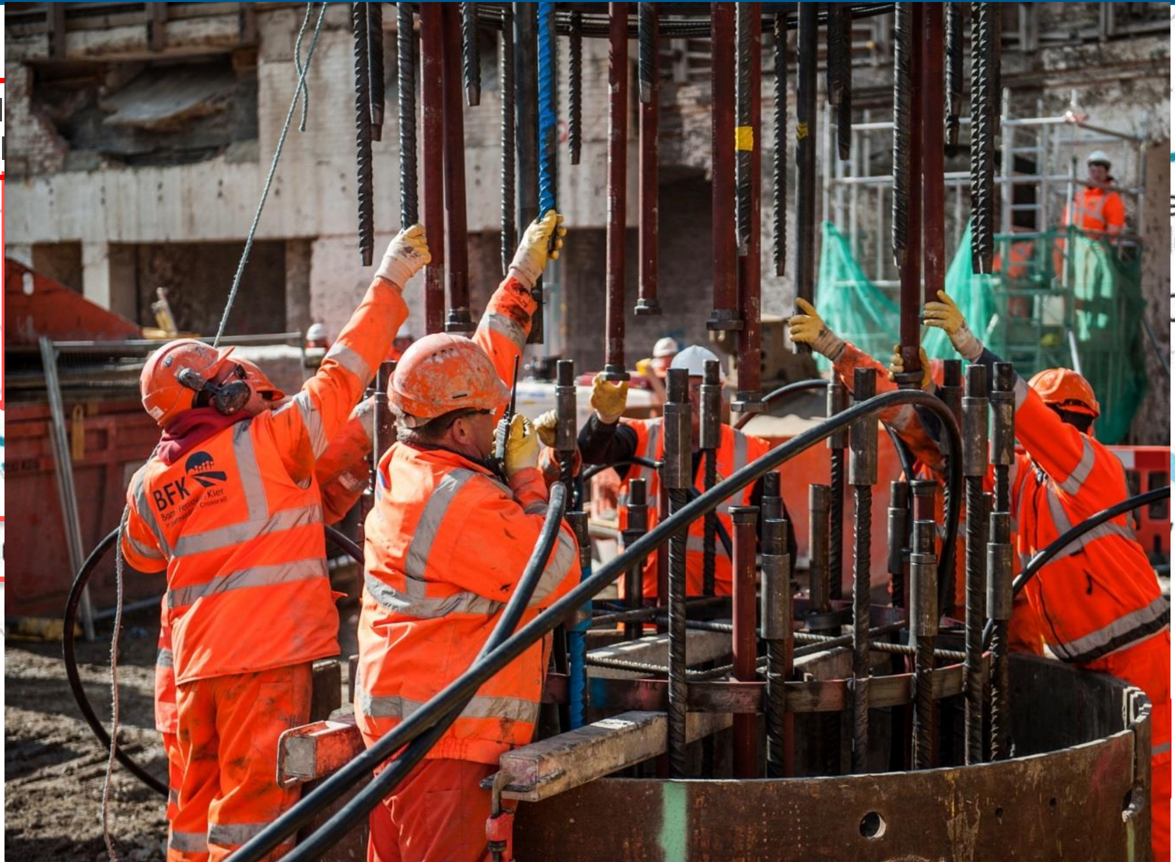


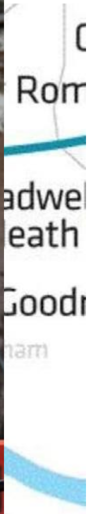
Cementation
SKANSKA

FPS
50 YEARS OF EXCELLENCE IN
GEOTECHNICAL
ENGINEERING

Base grouting







Crossrail Conference – 10th November 2015

Base Grouting, Osterberg Cell Test and the use of Fibre Optics in large diameter rotary bored piles

Kirstie Broadbent – Cementation Skanska Limited, UK

ABSTRACT: This paper describes works undertaken by Cementation Skanska Limited (CSL) at Crossrail's Farringdon Station, contract C435. The scope covered 26 large diameter rotary bored piles, using conventional slip casings, under bentonite support fluid founding in Thanet Sands. The piling works involved base grouting all 26 piles, of which 15 were permanently lined, and undertaking a bi-directional load test utilising Osterberg Load Cells (O Cell). These foundations were constructed to support Farringdon's western ticket hall, as part of the central section of Crossrail. Both the base grouting and load test were monitored using fibre optic technology. The O Cell working test provided strain data from vibrating wire strain gauges which were compared to data from the fibre optic instrumentation. Measurement being taken during the base grouting operations is understood to be the first such use of fibre optic technology in the UK. Cementation Skanska Limited, as specialist contractor in collaboration with the University of Cambridge Centre for Smart Infrastructure and Construction (CSIC), carried out this innovative instrumentation work. This paper will describe the specification requirements and considerations that need to be made when undertaking specialist works of this nature, as well as identifying areas of research which should be beneficial in developing improved standards for reference across the industry and future large infrastructure projects. The paper aims to explain some technical aspects, to provide a level of background understanding, with a focus on the practical applications and required considerations in planning and successfully executing such works.

1. INTRODUCTION

The site is situated east of London, approximately 700m north of the River Thames. The ground conditions comprised London Clay over Lambeth Clay becoming Lambeth Sand over Thanet Sands into which the piles were founded. Due to the extensive, existing and proposed, underground network of infrastructure within London, the use of permanent liners to bearing piles are increasingly required to prevent any impact on these tunnels due to the increasing possibility of surcharge loads. As demolished structures are replaced with higher rise buildings, piled foundation demand, and the loads to be resisted, has increased.

Due to high capacity of the Thanet Sands, both in shaft and end bearing, it optimises design if these heavily loaded piles found within this strata. As the Thanet Sand is often limited in thickness, underlain by weaker, less reliable strata, such as chalk, the length of pile can be restricted to get the optimum efficiency of capacity which leads to greater reliance on the end bearing resistance.

In order to mobilise the end bearing the pile must settle. The level of settlement generally exceeds the allowable structural design requirements. Standard pile design for piles founding in the Thanet Sand is to adopt a factor of safety (FoS) of greater than 1.1 times the Safe Working Load (SWL) on the shaft capacity alone. Where the Thanet sand thickness is limited and liners are used this can be difficult to achieve if the loads are high. The settlement when mobilising end bearing is directly affected by the cleanliness of the base. Predictive settlement calculation in designs use the soils elastic modulus which is generally derived from the Standard Penetration Test (SPT) 'N' value. Thanet sand is a fine and almost single size marine silty sand, which has a high relative density indicated by the SPT 'N' value in excess of 100 for 300 mm penetration. It is standard practice to cap the 'N' value when defining the elastic modulus. In these circumstances, as at Farringdon, Base Grouting is often considered a viable option to ensure a higher elastic modulus of the base and incorporating it into the design to reduce settlement and take the loads.

The piles were excavated as 2.4m diameter piles to the liner toe for installation of the dual sleeves. Below the toe of the liners, the 2.1m diameter pile was excavated, using support fluid once outside the clay. Once toe level is reached and the base cleaned, reinforcement was placed and the pile concreted. 4No. base grouting Tube-A-Manchette (TAM) circuits and two extensometer tubes were fabricated into the reinforcement cages, enabling grouting of the base, post installation.

2. BASE GROUTING

Background reading highlighted two contradictory theories of the benefits of base grouting. Theory A, (Fleming, 1993), suggests uplift negatively loads the soil, increasing pile shaft capacity as well as improving end bearing while still noting no overall increase in capacity in the ground. Theory B, (Troughton & Stocker, 1996) suggests base grouting reduces settlement, and allows lowering Factors of Safety (FoS) on shaft, through permitting greater inclusion of end bearing into the pile design.

The current Industry Specification, commonly referenced for base grouting, is the Institution of Civil Engineers Specification for Piling and Embedded Retaining Walls (ICE SPERW) which defines uplift as the key demonstration of successful base grouting with a prescription of no less than 0.2mm and no greater than 2mm under clause B3.5.11.4.

The guidance notes within this specification under clause C3.5.11.2 highlight that uplift may not occur if the pile is long and thin. However, there is no reference as to what ratio of diameter to length can be defined as ‘thin’ in order to advise practitioners of the discipline or Designers wishing to incorporate the discipline into their design.

A predicted upwards movement was calculated using in-house theoretical design software, Bearing v1.1.1 to determine shaft capacity and CEMset v4.01 for the settlement, based on parameters from information within the Geotechnical Baseline Report (GBR) and relevant boreholes, taking end bearing as zero and likely parameters and appropriate factors of safety as in standard design practice. The upwards force was initially calculated, assuming 100% contact of constant maximum achievable pressure (assuming equal loss of system pressure and increased head pressure for simplification), with the self-weight force deducted. Initial calculations suggested 3.4mm uplift could be expected, in line with ICE SPERW.

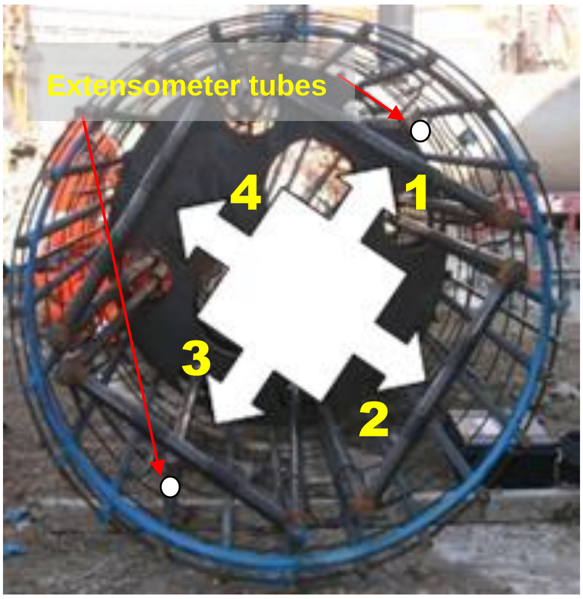


Figure 1: Base Grouting Circuit Arrangement

4No. base grouting Tube-A-Manchette (T-A-M) circuits and two extensometer tubes were fabricated within the reinforcement cages to allow the grouting (Figure 1). The day after pile construction, while the concrete strength is low, the seal in the T-A-Ms is broken by pumping water under pressure. The pressure is monitored and the T-A-M seal is considered cracked once pressure drops. After a minimum of 3 days of concrete curing, the piles were base grouted. Optical Fibre Sensors (OFS) were installed the length of the pile along the edge of the cage in the working test pile. The instrumented test pile’s performance under grouting was captured by OFS at different locations around the circumference of the pile, however, the details of these results will be captured in another paper. At Farringdon, four grouting circuits were used (As shown in Figure 1). In this case the conventional sequence is to grout circuit 1, followed by 3, then 2, then 4 (i.e. opposite sides of pile). The use of 4 circuits should be reviewed on the basis of the pile diameter and project specification. Should the criteria for grouting not be achieved within the first phase of grouting then it is repeated. Standard practice is to limit the number of phases to 3 as little or no grout is able to be injected past this point.

By reflecting on practicalities of procedure, steady reductions of injected volume and quicker pressure cut offs after each circuit were noticed. This suggested disturbed soil voids were being filled from initial circuits, reducing area available for grout to flow and possibly lesser contact area than originally assumed for the final phase. From recalculation the expected uplift was 0.4mm. In addition, based on theory B, the purpose is to permit less conservative SPT values to be incorporated into the design to permit a greater elastic modulus of the soil at the base in order to reduce the settlement necessary to mobilise the base resistance. It should therefore be considered that other criteria can be used to determine whether the actual outcome is achieved.

More recent research of base grouting uncovers the Arup Ranking method (Patel et al, 2009), differing from ICE SPERW. This method was proposed, yet it was initially rejected at C435 due to limited use within the industry and uplift being deemed the only way to demonstrate sufficient cleanliness of the base to confirm design assumptions. However, as uplift was consistently not achieved, CSL followed the procedure for this method as a second check. The method requires monitoring of several parameters as follows:

- Base condition prior to concreting
- Total Volume of Grout injected
- Maximum Pressure
- Residual Pressure
- Uplift

These items are reviewed against a set of criteria as detailed in Patel D et al, 2009. Each item is assigned a score in accordance with the method, based on the output result. The final score is totalled and provided the overall score is greater than 7 the base grouting is deemed acceptable.

With the load test confirming greater than expected shaft resistance it is considered that minimal uplift may not mean poor base grouting but rather greater shaft capacity or simply too high a shaft resistance to permit movement. As a result, going forward on future projects with base grouting, a more all-encompassing set of criteria similarly to the aforementioned ranking method may be a more suitable approach for inclusion in inspection and testing.

3. OSTERBERG CELL TEST

The key differences between the Osterberg Cell Test (O Cell) and a standard static load test are:

1. Bi-directional loading, permitting separate assessment of shaft from end bearing resistance and;
2. Maximum test load.
3. Potential re-use of the load cell permitting stopping and restarting of the test.

Figure 2: Osterberg Cell Plate/Tremmie Guide from top

The preparation for this test mainly lies with off-site works. The Osterberg Cell is only produced overseas and therefore currently needs to be imported. The cell is welded into position into the reinforcement cage. The position of the cell(s) needs to be such that it permits a tremmie pipe

together by reinforcement bars in the temporary condition to permit lifting. Once in the vertical position over the pile the bars are cut (Figure 4) and the bottom of the cage is held in place through the welds on the cells connecting them.

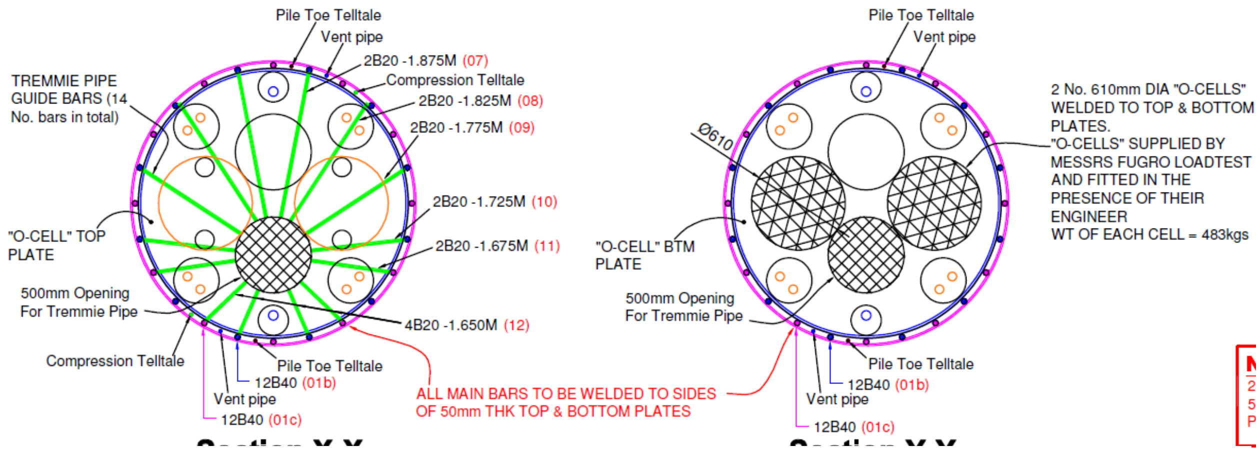


Figure 3: Osterberg Cell Fabrication Drawing

(Figure 2 & Figure 3) to enter through the plate for concreting both in wet and dry installation conditions to allow for the event of water ingress.

In addition the plates between which the cell is welded need to permit the flow of concrete. To prevent air build up, which may affect results, during testing, there also vent pipes installed. Furthermore, there are tell-tale bars to permit measurement of the cell expansion and the movement in both the downwards and upwards directions separately (Figure 3). In addition to this, as with base grouting reference beams and dial gauges are also used at the top of the pile to determine uplift. The base supports for these beams were 12m apart to ensure the surrounding ground was not moving with the pile and affecting the readings.

All tubes and reinforcement bars within the working test pile were scored and cut, respectively, once the cage was fabricated. This is to prevent the steel from adding to the resistance capacity in preventing the expansion of the cell and affecting the results. The tubes are scored to a depth that will still permit their use during base grouting and sonic logging. The plate is held

The test requires less site area and the majority of preparation can be carried out off site, with no need for anchor piles. Restraint at the top would prevent differentiation between resistance from the shaft or anchors and the test would only, also less effectively, determine capacity of the end bearing, leaving the test possibly ineffective in confirming design assumptions.

Figure 4: Temporary works bars for lifting being cut.

However, with no anchor piles the ability to test the base to the full extent of the desire load requires sufficient shaft capacity of the pile above the cell to the shaft and base below. Throughout the test, movement is monitored, as with standard tests. It can be stopped at any time should the overall movement or the rate exceed the desired amount. As a result, less conservative design assumptions could be adopted that may be different to the permanent works design when determining the anchor resistance.

At Farringdon the design suggested the shaft capacity was not sufficient to resist the full test load to verify the base resistance. CSL's in-house design team drew similar conclusions from a preliminary review of the relevant soil information. Fugro Loadtest, who were contracted to carry out the specialist test, suggested the design assumptions were conservative based on similar tests in the area. Through close liaison between contractor and consulting engineer, the loading cycle was altered from the ICE SPERW and approved through normal process. Acceptable movement limits at each stage were also agreed. If, at any point, limits were exceeded or unexpected the test would be stopped and if insufficient data was recovered, an alternative means of restraint would be installed for retesting.

Therefore the greatest lesson from the Osterberg cell test at Farringdon is that the shaft resistance above the cell should be considered when determining which pile should be designated as the test pile. This is to ensure there is sufficient capacity. Furthermore, it should be a fully collaborative part of any scheme with clear communication and understanding of expectations from all parties including the specialist test house, piling installation contractor as well as the consulting engineer and main contractor.

It is also worth noting that the Osterberg cell test could possibly be re-used and therefore it should be considered whether, within the industry, going forward we should be considering their installation for ongoing monitoring of as-built pile capacity over time and whether this may permit increased pile re-use as cities like London become more and more developed.

4. FIBRE OPTICS & STRAIN GUAGES

In collaboration with the University of Cambridge Centre for Smart Infrastructure and Construction (CSIC), and in support of research using innovative instrumentation, CSL installed fibre optics, as an alternative strain measurement method, to the strain gauges permitting secondary analysis and research. The detail of this has been published within the proceedings of the Edinburgh conference "The use of fibre optic instrumentation to monitor the OCell load test on a single working pile in London". Within this paper the focus is on the practicalities of their use and site considerations.

Conventional wire strain gauges can be installed completely off site with designed in junctions at the splice for connection during cage placement. Once fully connected, the data can be fed into the analyser. The limitation is that the data received is at discreet points dependant on the placement of the reinforcement cage to ensure the right level of the data capture desired. Should excavation reveal different ground conditions, resulting in a change in level for data capture, the gauges will need to be moved on site prior to installing the cage.

Figure 5: Fibre Optic Strain Cable Installation & Repair

The fibre optics can be installed partly off site by installing them within the base section of cage. However, they can as easily be installed on site completely. Provided the cage is delivered with enough time before placing the reinforcement this will have no impact on programme. There needs to be allowance within both the temperature and strain fibre optic sensor cable, at the cells location, to permit expansion without damage to the cable if using them to monitor a bi-directional load cell test. The Cable must be continuous, which requires careful installation. Repairs can be carried out on site if there are breakages.

The strain cable is fixed into tension at the top and bottom of each cage section to permit continuous analysis between these points. Once the sensor cables are installed in the base cage section the remaining sections will be installed as the cage is lowered into position. This will add a little amount of time onto the reinforcement cage installation period which should be considered in any programming of the works. This should be reviewed against the number of cables being installed and the number of cage sections the cable is to be spread across.

The main advantage of fibre optic technology is the continuous strain results that allow a more detailed analysis of the ground and its capacity along the full length of the pile. As this method of analysis can be connected to at any point in the future, this data could be captured after installation for the as-built and loaded conditions as well as for test piles, if the cables remain exposed for connection to the analyser. Going forward this would be of benefit to be used to provide long term benefits. Furthermore, If combined with an Osterberg Cell this really could be an opportunity for true Smartpiles™ with the possibility of increased long term sustainability and re-use of piles through re-testing of piles.

5. TWIN WALLED LINERS

Twin walled Liners are commonly used in London, due to the high amount of existing tunnels, as a method of preventing transfer of surcharge load that may cause damage or affect the tunnels behaviour. At Farringdon, the sequencing meant the tunnels were installed after the piles and it was understood the liners were a method to prevent the tunnels loading the piles both during the tunnel construction and use and causing additional loading which they won't have been designed for.

Figure 6: Excavation of Sand after grouting annulus.

At Farringdon, twin walled liners were used with bitumen coating on the inner liner. The purpose of the bitumen is to create a smooth surface with a reduced transfer of friction. The ground within the UK is generally at a temperature between 8-10ºC and once the bitumen temperature rises the bitumen becomes less brittle and more fluid theoretically permitting the piles behaviour to be separate from the surrounding ground. The use of two liners and grouting within the annulus between the inner and outer prevents the temporary casing from being grouted into place.

There is often significant wastage of steel when installing permanent liners as to permit installation they are often carried up to the top of casing level. At Farringdon, to reduce cost and material wastage a method was devised to permit lowering the top level of the liners to COL. This was done by installing two hollow circular lifting points on opposite points of the cylinders. Using quick release shackles in reverse we were able to lower the liners into replace and then disconnect the lifting accessory with ease.

However, there is limitation in achievable tolerances of positioning and verticality. At

Farringdon tolerances were increased through the Field Change Document request process. This change altered verticality to 1:100. The original specification verticality of 1:200 was problematic for the method of installation due to the large amount of made ground and the size of piles.

To prevent buckling of the liners, due to hydrostatic pressures of grouting the annulus, the inner liner was filled with sand to at least one meter below the grout level (Figure 6). This needs to be allowed for in the programming durations. The use of sand as opposed to backfill material was selected to ensure even compaction of a consistent material. The grout needs to cure before the sand can be removed and the pile completed. As a result the key lesson is timing of grouting the annulus within the first shift being critical to programme as delaying it to a following shift can add on a further shift to the programme. This is therefore a lesson in the importance of transparency in installation processes as well as realistic tolerance conditions for placement of liners.

6. SUMMARY

Based on near zero movement of the piles and guidance notes within the ICE SPERW it is concluded that the criteria of uplift may be based on ground conditions that don't reflect the factors of safety that are introduced into the design and thus this may not be key to determining satisfactory base grouting. Due to improvements of instrumentation and monitoring the recorded uplift is now measured and recorded by automatic digital gauges with an increased degree of accuracy. An all-encompassing assessment of the pressure, volume and uplift such as the Arup Ranking method (Patel et al 2009) allows parameters other than uplift to demonstrate success in order to provide assurance, based on more than one set of criteria that the grouting has been carried out correctly. This method could be site specific with criteria being adapted to suit ground conditions, pile sizes and lengths to determine likely uplifts and volumes and pressures.

Upon completion of the O-cell test, it was possible to reassess the movement against the known strata, which confirmed the original design capacity assumptions were conservative. Combining this realisation with minimal base movement, a ranking method such as Arup's is proposed to be adopted as standard practice going forward.

Based on the experience of base grouting within the scope of this project, and the limited as well as contradictory information available, it would benefit the industry and its' clients to invest in further research into the use of this discipline to determine the benefits and cost effectiveness of the solution.

The Osterberg Cell Test is a simple test with preparation predominantly carried out off site. There is no need for additional anchors, however there needs to be design checks to confirm shaft resistance above the cell is greater than the proposed load to be tested below. More than one cell can be used to achieve larger loads and they can be positioned to permit concrete flow and tremmie access. There is some additional time required on site to allow the bars to be cut and the top plate greased for de-bonding to permit the cell to expand freely. For the additional checking of the cage – additional space is required to lay both sections of cage down unless it is possible to do the checking off site which can be client dependant. The ability to re-use the cell was not taken advantage on the C435 contract. However, this additional benefit could create the opportunity to re-test piles It could be the future of the foundation industry to combine such load cell tests and their capacity for re-use with the fibre optics to create the ability to re-test piles to determine their capacity for re-use in the future when the building may have changed hands with an alteration to use and/or loading to reduce the life cycle costing of the building.

The use of fibre optics for strain analysis of the pile gave the soil strength profile along the entire shaft length of the pile. The cables are easily installed and can be done on site with minimal impact to piling works. Some consideration should be made for space needed to lay the cage down if they are to be installed on site. Using strain gauges relies on the cage to be installed in position and for the gauges themselves to be in the correct position requiring additional onsite checks to be carried out. This method of analysis has many benefits and could allow us to monitor the pile and ground behaviour over time if the cables can be connected to at a later date.

The installation of liners can be done with ease however, the positioning and achievable verticality should be reviewed on a project by project basis considering the liner length, level below top of casing, ground conditions as well as placement method. When using twin walled grouted liners the programme durations need to consider the need for placing a homogenous material within the liners to prevent buckling under the hydrostatic pressure of the grout. In addition, the sequence of work and how delays to individual activities can affect the overall programme should be considered and identified with a transparent programme. The use of quick release shackles in reverse permitted the liners to be placed at cut off level rather than being brought to the surface. This made a great saving of steel. However, this was only possible as a temporary casing was also used to support the ground above. In addition, using dual liners meant the grout was within the annulus and the temporary casing did not become grouted into position.

7. REFERENCES

Fleming W.G.K, May 1993, Paper 3078, Proceedings of the institution of Civil Engineers, pages 88-93, ‘The improvement of pile performance by Base Grouting’.

Institution of Civil Engineers (ICE), 2007, ICE Specification for Piling and Embedded Retaining Walls, Second Edition.

Ouyang Y et al, 2015, Proceedings of the 16th European Conference on Soil Mechanics and Geotechnical Engineering, ‘The use of fibre optic instrumentation to monitor the O-Cell load test on a single working pile in London’

Patel D, Glover S, Chew J, Austin J, Ove Arup & Partners, ‘The Pinnacle – Design & Construction of Large Diameter Deep Base Grouted Piles in London, 2009

Troughton V M, Stocker M, July 1996, Issue 119, Proceedings of the institution of Civil Engineers, pages 186-192 ‘Base and Shaft Grouted Piles’.