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Lessons learnt from installation of field instrumentation

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Imperial College in London, UK, as part of an Engineering and Physical Sciences Research Council (EPSRC) funded research project and in collaboration with Crossrail urban railway project, is performing field monitoring research to investigate how tunnelling affects existing tunnels. Comprehensive instrumentation was installed in Hyde Park and bordering Bayswater Road, beneath which the new Crossrail tunnels were constructed in London Clay below the existing London Underground Central Line tunnels. Surface and subsurface instruments were installed around the Crossrail tunnel alignments to monitor the ground response to the tunnel construction. Monitoring systems of sufficient resolution and accuracy were adopted to achieve high-quality data for assessing the tunnelling-induced ground response and mechanisms of movement from earth-pressure-balance machine tunnelling. The installation of surface and subsurface instrumentation took place in the summer of 2011. This paper describes and discusses the installation of rod extensometers, in-place inclinometers and multi-level vibrating-wire borehole piezometers. Selection of the appropriate cement–bentonite grout mixes for backfilling these borehole instruments is discussed, as this is critical for representative measurements of ground response. Some practical challenges arising during the installation process and how they were overcome are also described. Confidence in the instrument performance is demonstrated using example monitoring results from the piezometer and extensometer installations.

1. Introduction

Research is being undertaken by Imperial College London to investigate the effects of tunnelling on existing tunnels in London Clay. The research opportunity arose with the Crossrail construction of 21 km of underground twin-bore railway tunnels through central London where the new tunnels interface with existing networks of London Underground Limited (LUL) tunnels and other utility tunnels. The Crossrail western tunnel section involves construction of twin tunnels using 7.2 m dia. earth-pressure-balance tunnel-boring machines (TBMs) mostly in London Clay. The tunnel drives below the existing LUL Central Line tunnels near Hyde Park enabled a field investigation to be made of the effect of new tunnel construction on both the ‘greenfield’ ground and the ground in the proximity of existing cast iron segmental lined tunnels. The research, involving instrumentation and monitoring fieldwork in Hyde Park and Bayswater Road, aims to provide a high-quality case study of tunnel construction in London Clay and the mutual interaction of both the new and existing tunnels.

This paper describes the instrumentation scheme and the installation works. A particular focus is given to the selection of grout mixes used for the installation of borehole instrumentation. Practical challenges faced during the installation works are discussed. Example monitoring data from the instruments are presented to illustrate their proper functioning and accuracy. The

intention is that lessons learned can be applied to future similar instrumentation installation works.

2. Borehole instruments and their installation

The field research site is located near Victoria Gate, at the north-west corner of Hyde Park, where the Crossrail west-bound and east-bound tunnels pass beneath the LUL Central Line running tunnels just east of Lancaster Gate. The actual crossing is directly beneath Bayswater Road. The location of the instrumentation site in plan and the Crossrail tunnel longitudinal section are shown in Figure 1.

2.1 Instrumentation plan

A plan of the instrumentation layout is shown in Figure 2. There are in total 38 boreholes, each accommodating one or more instruments. Extensometer boreholes and inclinometer boreholes are paired so that both vertical and horizontal ground displacements can be determined at the same locations. Piezometers and spade cells have been installed around the Crossrail tunnels to measure pore-water pressures and changes in earth pressures.

Most of these instruments are aligned in a main instrument array transverse to the Crossrail tunnels. In the main array, the instruments are positioned so that they cover, in the southwest part, the ‘greenfield’ ground response and, in the northeast part, the ground response under the influence of the existing Central Line

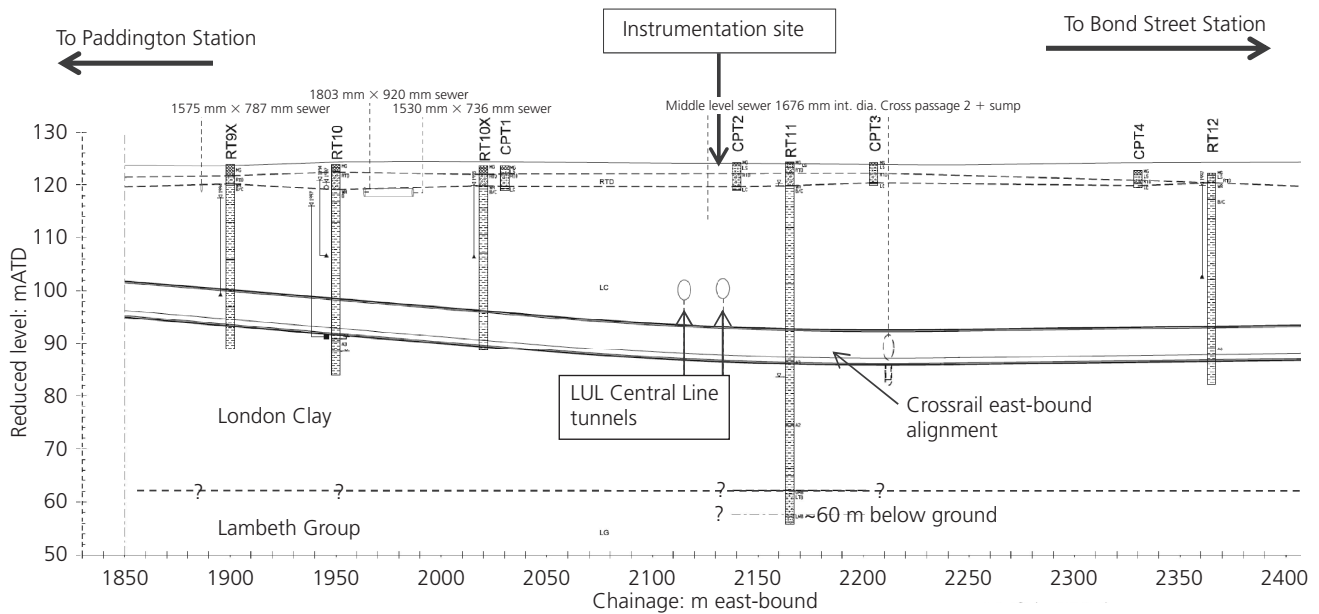
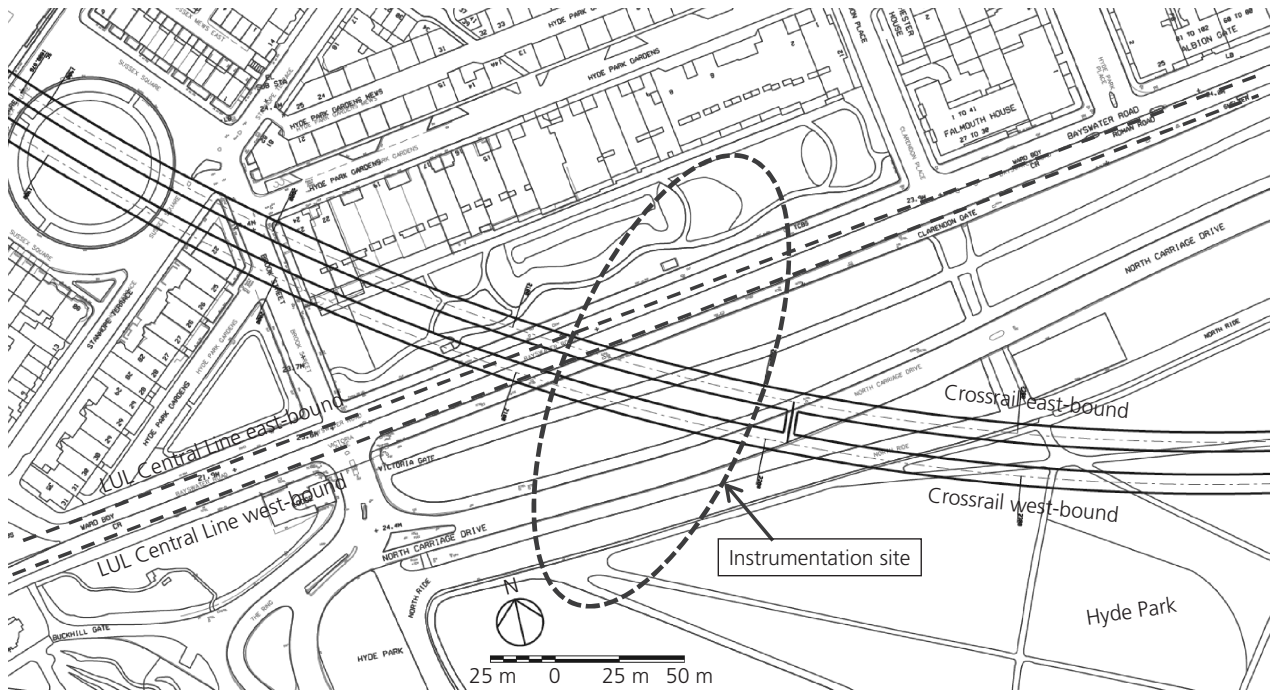


Figure 1. Location of instrumentation site in plan and Crossrail tunnel longitudinal section

tunnels. Different responses would be expected for these two areas. There is another instrument array on the pavements of Bayswater Road parallel to the LUL Central Line tunnels. These boreholes have been positioned as close as possible to the Central Line tunnel to observe how the presence of the existing tunnels affects the ground response, compared with ‘greenfield’ conditions.

The positions of the installed rod extensometers and in-place inclinometers along the main array are presented in section

schematically in Figures 3(a) and 3(b) respectively. Each cross in Figure 3(a) represents an anchor at which the subsurface vertical displacements are measured: these are largely distributed around the Crossrail tunnels. Installing them around the existing Central Line tunnels was precluded by site constraints such as heavy traffic and Thames Water pipes on Bayswater Road. Nevertheless, it was possible to install some borehole instruments under the pavement of Bayswater Road, where the Central Line tunnel will influence the ground response. As shown in Figure 3(b), there is

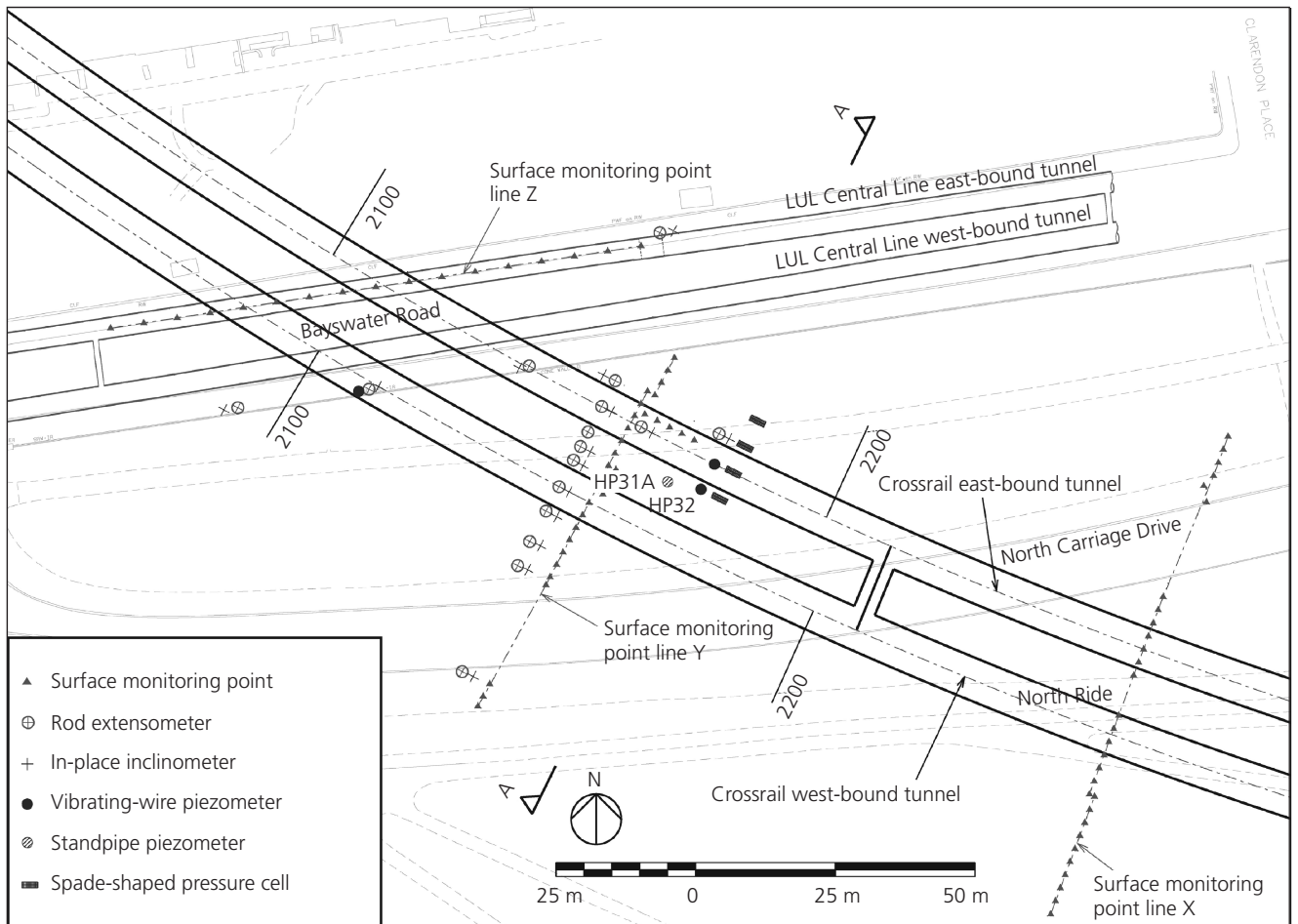


Figure 2. Plan of instrumentation in Hyde Park and Bayswater Road

a similar arrangement for the inclinometer boreholes in the main array. The inclinometer and extensometer boreholes are paired (about 2.5 m apart), to facilitate interpretation of the displacement data: this is particularly useful when determining resultant displacement vectors and ground strains.

In the main instrument array, piezometers and spade cells were installed, as shown in Figure 3(c), around the Crossrail east-bound tunnel to monitor changes in pore-water pressure and total horizontal earth pressure as the TBMs approach and pass. There are two multi-level vibrating-wire (VW) type piezometer boreholes within Hyde Park, with six piezometer sensors at different depths in each. A conventional standpipe piezometer was also installed just 2 m away from one VW piezometer borehole, at the same distance from the Crossrail tunnel, to verify the long-term pore-water pressures recorded with the VW piezometers.

Instrument types were selected for the Hyde Park site considering the magnitude of changes predicted by empirical methods. Although other instruments and techniques may be available to perform the tasks, the experience of the Imperial College field

monitoring team gained from previous research projects (e.g. Jubilee Line extension and Channel Tunnel rail link projects) played an important role in the selection process.

2.2 Rod extensometers

2.2.1 Measurement principle and performance

Subsurface vertical displacements were measured using multi-level rod extensometers. A maximum of eight anchors, positioned at different depths, attached to stainless steel rods were installed in a single borehole. The steel rods were adjusted so that the rod tips were just below the reference head at the ground surface (Figure 4). The depths of the rod tips below the reference head were measured using a dial gauge and the reduced level of the reference head itself by precise levelling. By combining the dial gauge and precise levelling measurements, the reduced level of each anchor at each survey time could be determined.

Rod extensometers generally have a much higher accuracy than magnetic extensometers. As the dial gauge to be used has an accuracy of $\pm 0.05\%$ full-scale range (100 mm), the accuracy is ± 0.05 mm, compared with the ± 1 mm accuracy typically

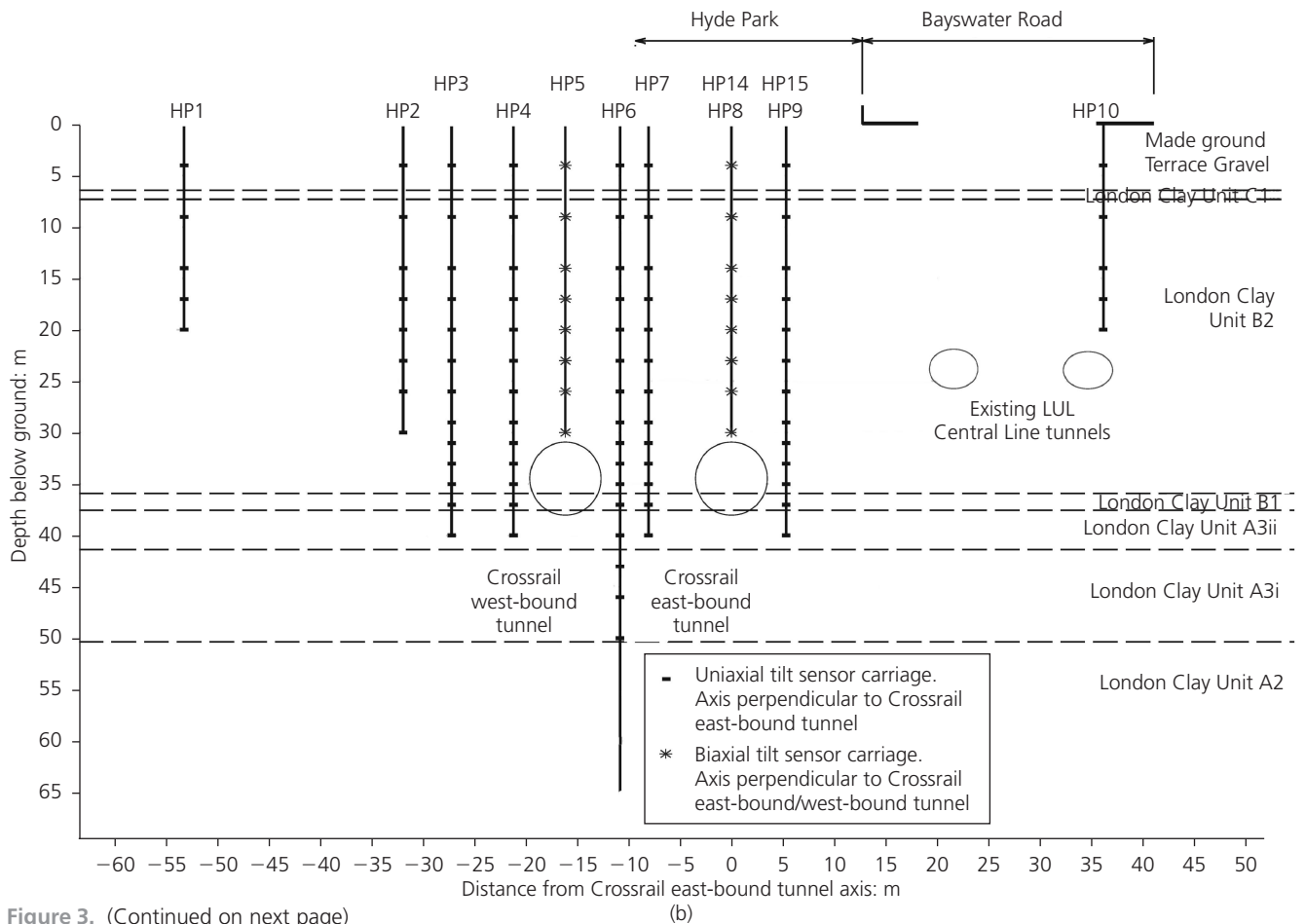
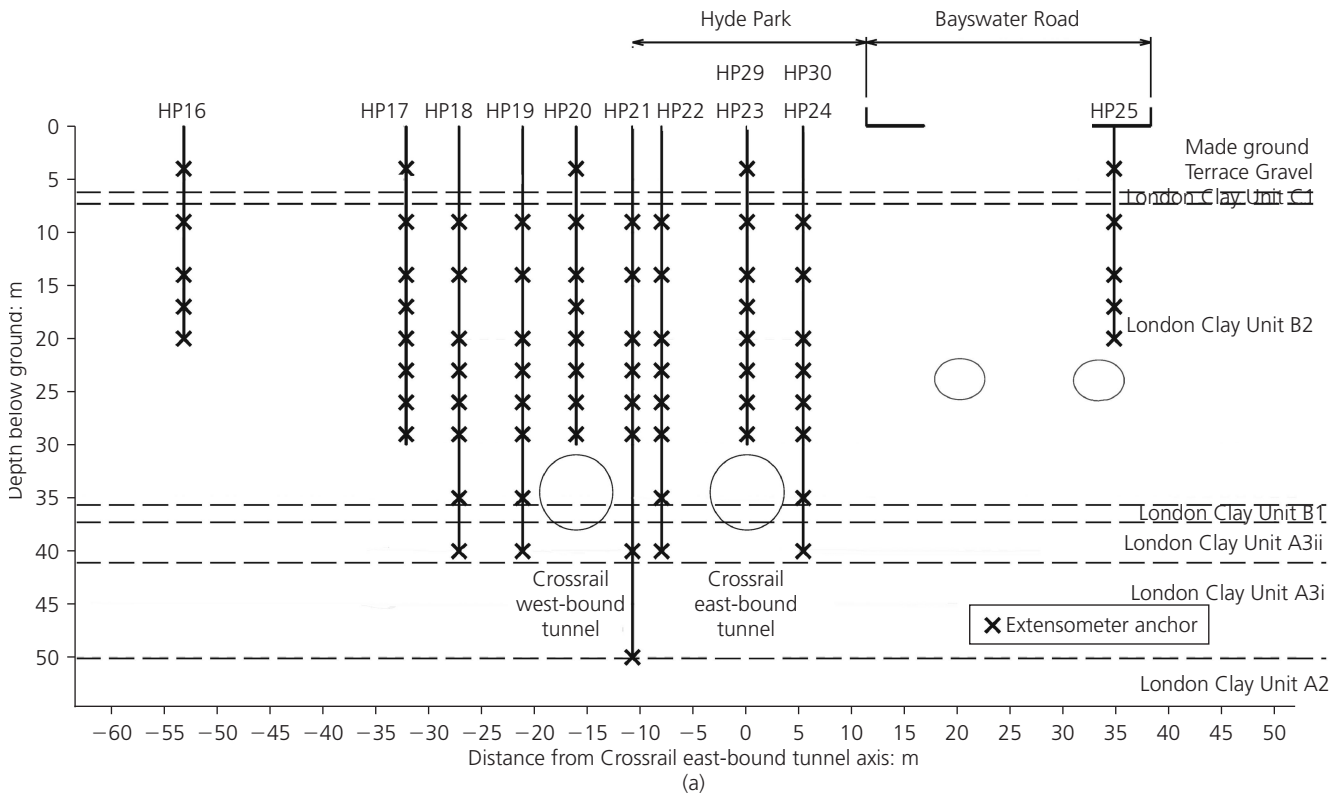


Figure 3. (Continued on next page)

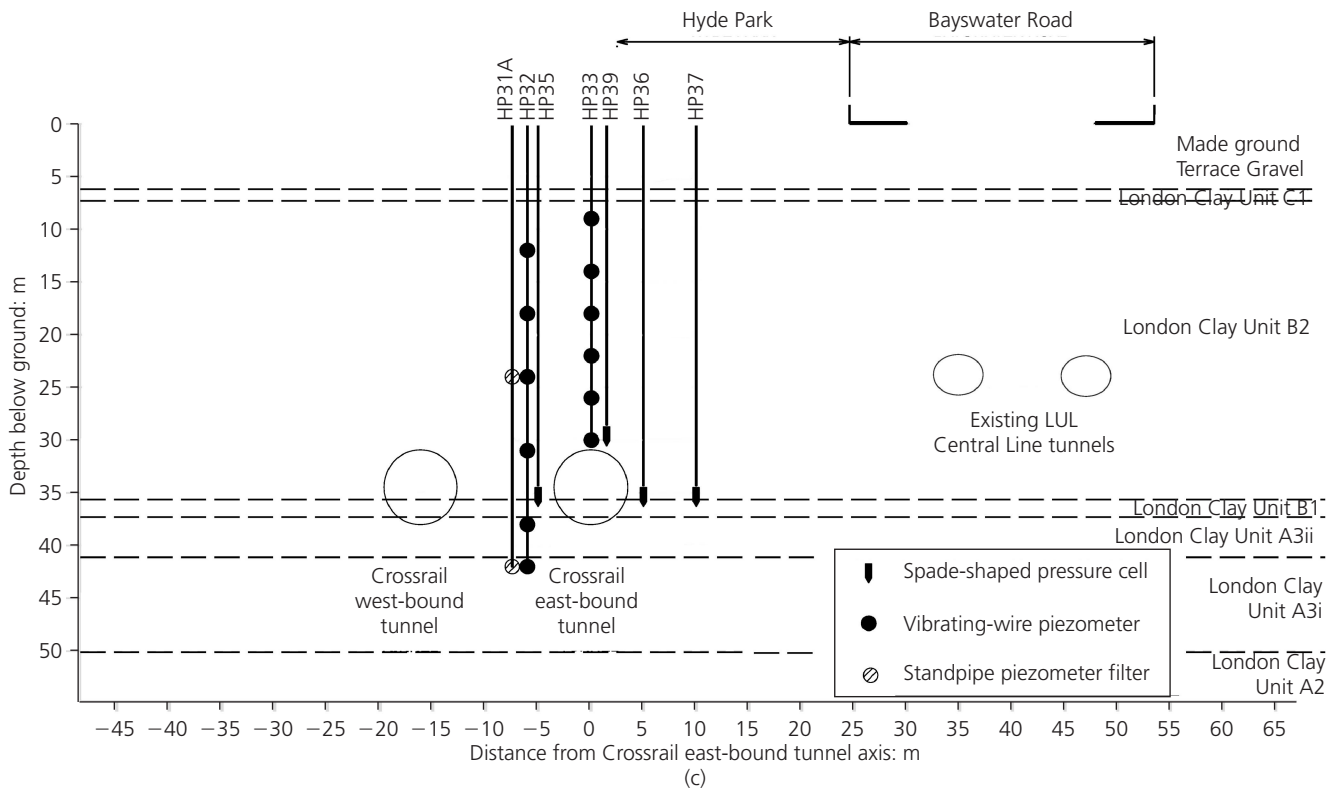


Figure 3. Cross-section A–A: (a) cross-section of rod extensometer boreholes in the main instrument array; (b) cross-section of proposed in-place inclinometer boreholes in the main instrument array; (c) cross-section of piezometer and spade cell boreholes in the main instrument array

achieved with a conventional magnetic extensometer. The overall measurement accuracy is governed by that of the precise levelling which is about ± 0.3 mm at best. Sub-millimetre accuracy was essential at the Hyde Park research site as the maximum anticipated settlement was only about 10 mm.

2.2.2 Installation

The rod extensometers were installed in boreholes formed by cable percussion drilling. During final stages of borehole drilling, the stainless steel rods, poly vinyl chloride (PVC) sleeves and hydraulic tubing for the anchors at different depths were laid out and prepared on the ground surface. On completing the borehole drilling, a tremie grout pipe was first put down the borehole. The assembly of the bottom-most anchor was then lowered down the borehole and, once at the required depth, the prongs of the anchor jacked out hydraulically by way of the tubing to secure the anchor into the wall of the borehole. This procedure was repeated for each of the remaining anchors. After these anchors were installed, the borehole was ready for backfilling with a cement–bentonite grout. Grout mix trials were conducted at the contractor’s workshop before commencement of the site work to estimate the grout properties and pumpability of different design mixes. The grout mixes used for various types of instruments are discussed in Section 3. After grouting, casing used to support the

borehole through the ground above the London Clay (in particular the Terrace Gravels) was removed, care being taken to maintain the grout level during this operation. Once the backfill grout had hardened, the reference head was installed and the headworks constructed. Finally, the upper sections of the stainless steel rods were replaced with adjustable rod heads which were set to about 50 mm below the reference head. The dial gauge has a range of ± 50 mm, allowing measurements of maximum 50 mm settlement or heave of the rods and hence anchors relative to the reference head.

2.3 In-place inclinometers

2.3.1 Measurement principle and performance

Subsurface horizontal ground displacements were determined using in-place inclinometers. Each inclinometer borehole installation consisted of a series of micro-electro-mechanical system (MEMS) accelerometer sensors mounted on short carriages positioned at selected intervals down a conventional inclinometer casing. Each accelerometer measured the capacitance change in response to tilt and transmitted the voltage change to a data-logger. The MEMS tilt sensors were left in place during and after the tunnel construction so that automated data-logging was possible. A maximum of 16 sensors were installed in any one single inclinometer casing. The horizontal displacement profile

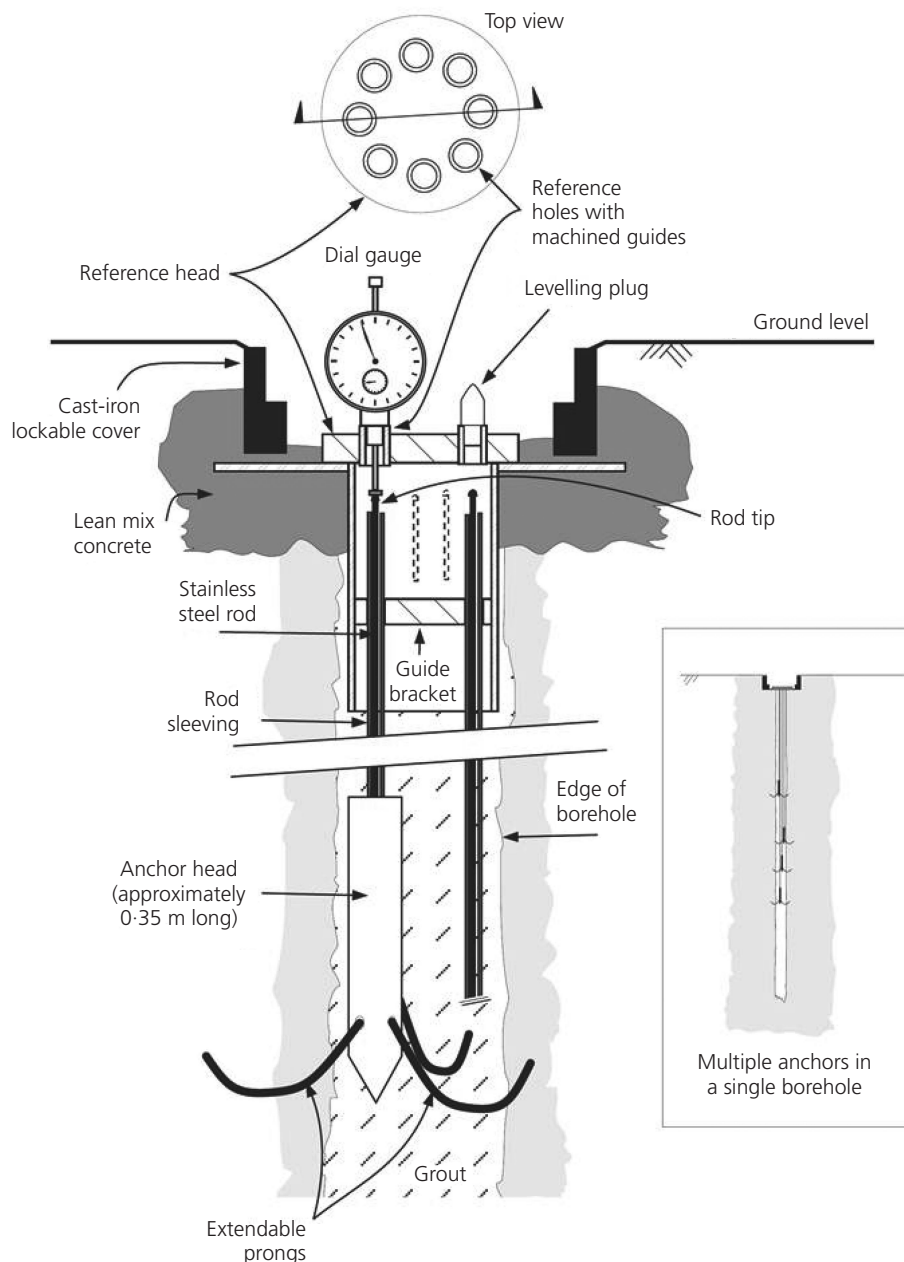


Figure 4. Details of multi-level rod extensometer (Nyren, 1998)

along the inclinometer casing could be deduced by integrating the rotations over assumed appropriate gauge lengths between the sensors (see Figure 5).

The MEMS tilt sensors have a measurement resolution of 2 arc seconds. Baseline monitoring after installation showed that repeated readings on a number of tilt sensors over a few days fell within a range of about 4 arc seconds, equivalent to about 0.08 mm for a sensor spacing of 4 m (i.e. 0.02 mm/m). Since this potential error occurs independently for each individual sensor, the overall repeatability for a maximum of 16 sensors would

equate to individual repeatability $\times \sqrt{16}$, which is 0.16 mm over 32 m (2 m spacing), or 0.32 mm over 64 m (4 m spacing).

2.3.2 Installation

Conventional inclinometer casings were installed in the boreholes formed by cable percussion drilling. Aligning the casing keyways to the desired orientation (perpendicular/parallel to the Crossrail tunnel alignments) was achieved using a string stretching across two pins on the ground, set out by surveying. The boreholes were backfilled with a cement–bentonite grout (discussed in Section 3) and the in-place tilt sensors were installed in the inclinometer

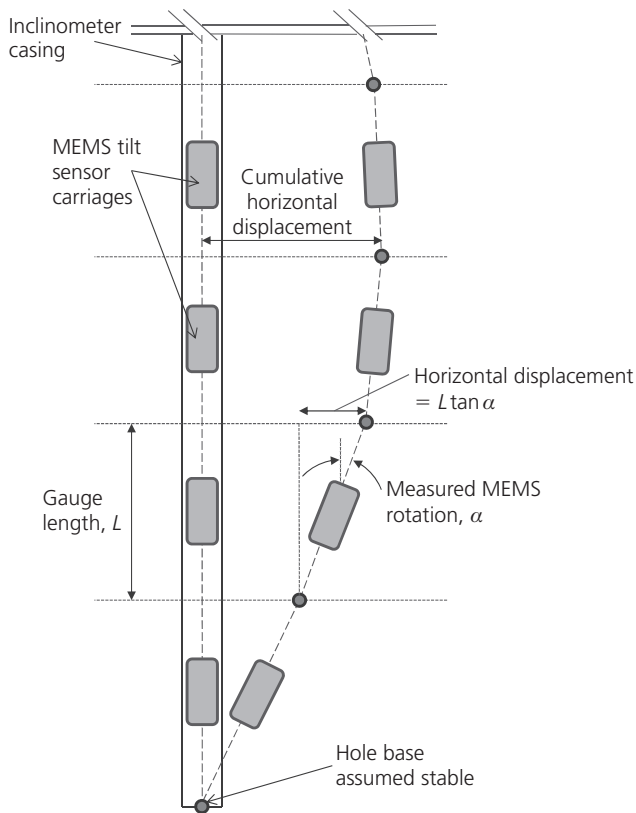


Figure 5. Schematic representation of in-place inclinometer

casings at a later stage. Each MEMS tilt sensor is fixed within a protective aluminium tube, with O-ring seals at its ends, which is mounted on a short carriage with wheels on one side and a leaf-spring on the other. Each carriage was pushed along the keyways of the inclinometer casing to the required depth using PVC rods of different lengths and held in position by the leaf-springs. Consecutive tilt sensors within a single borehole were connected in series so that all the sensors were read using just one signal cable coming out from the inclinometer casing. This was connected to a data-logger placed within the headworks. The frequency of the data logging was set and the data periodically downloaded from the data-logger to a laptop.

2.4 Vibrating-wire piezometers in fully grouted boreholes

2.4.1 Measurement principle and performance

The technology of VW type instruments has advanced in recent years, resulting in better longevity and durability than before. Historically, conventional Casagrande type standpipe piezometers, which require large groundwater inflow to or outflow from the standpipe, were installed within a borehole, surrounded by a sand filter and sealed with a bentonite plug, the remainder of the borehole being grouted (Figure 6(a)). When first developed and introduced, single VW piezometers were installed in the same way (Figure 6(b)). Recently the technique of installing them using the ‘fully grouted’ method, in which the VW piezometer units are embedded in a suitable cement–bentonite grout, has become popular (Dunnicliff, 2008).

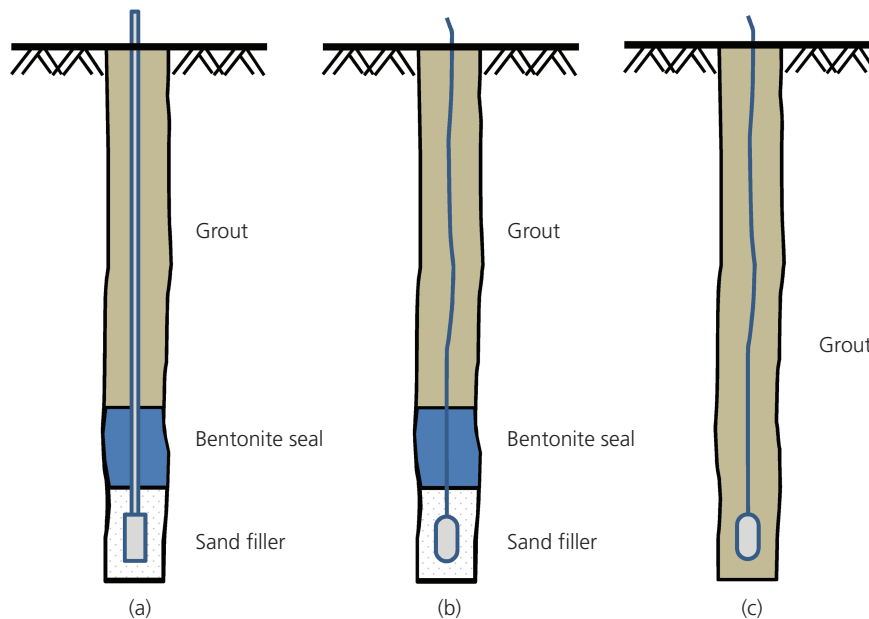


Figure 6. Schematic representation of: (a) traditional standpipe piezometer; (b) diaphragm type piezometer with traditional installation; (c) diaphragm type piezometer in fully grouted borehole (after Contreras et al., 2008)

Vaughan (1969) suggests that for diaphragm type piezometers, which require only very small amounts of water flow to operate, granular filters can be omitted and the piezometer sensors can be fully backfilled by low-permeability grout. Mikkelsen (2002) discusses the practical side of the fully grouted method and recommends good practice and appropriately proportioned cement–bentonite grout mixes. Dunicliff (2008) and Contreras *et al.* (2008) report successful installation and monitoring of piezometers installed by the fully grouted method at various sites around the world. A schematic diagram of a borehole piezometer installed using the fully grouted method is shown in Figure 6(c).

The fully grouted method has a major advantage in that the straightforward backfilling procedure allows multi-level piezometer sensors to be installed in one single borehole (very difficult to achieve reliably with sand filters). This makes it possible to obtain a pore-water pressure profile with depth at a single borehole location, significantly reducing the number of boreholes required.

2.4.2 Installation

First, the porous filters of the piezometers were de-aired by boiling in water. Prior to installation, pre-installation ‘zero’ readings were taken at ground level with the VW piezometer sensors in de-aired water. This allows future readings from these sensors to be referenced to atmospheric pressure. On site, six sensors were installed in a single borehole. A PVC grouting pipe was used for backfilling each borehole and also for holding the sensors at different depths. As the grouting pipe lengths, with sensors attached, were lowered down into the borehole, their joints were connected and sealed using duct tape. Low-permeability cement–bentonite grout (see Section 3) was then pumped by way of the grouting pipe to the base of the borehole until the piezometer assembly was fully grouted. During backfilling, the sensors were monitored by a hand-held VW readout unit to check their responses to the wet grout. Finally, the cables were connected to a data-logger housed within the headworks.

3. Selection of grout mixes for backfilling boreholes

A key requirement for successful monitoring of subsurface ground response is the appropriate selection of grout for backfilling the boreholes used for installing instrumentation. Ideally the grout provides a substitute material for the soil that has been removed from the borehole, such that the ground response is as close as possible to that of the undisturbed ground, with the instrument installed integrally within it. In practice it is not usually possible to match realistically every aspect of the ground and so attention is focused on providing grout properties most applicable for the quantity being measured.

3.1 Background considerations

Depending on the instrument types, the backfilling grouts should have different mechanical properties relative to the surrounding

ground. Mikkelsen (2002) argues that for extensometers, the grout should have stiffness equal to or smaller than the surrounding ground, while for inclinometers, a stiffer grout is desirable. For piezometers fully grouted in a borehole, Vaughan (1969) proves analytically that the error associated with pore-water pressure measurement is insignificant if the grout permeability is not larger than 20 times the ground permeability. More recently, Contreras *et al.* (2008) demonstrated using finite-element analyses that the grout permeability could be 1000 times higher than that of the ground before significant errors in pore-water pressure measurement occur. Therefore, three types of grout mixes were chosen for backfilling the extensometer, inclinometer and piezometer boreholes. In addition to the stiffness and permeability requirements, the grouts should also be readily mixable and liquid enough to be pumped through the hose and tremie pipe to the borehole.

In order to select appropriate grouts and investigate their properties, a number of trial mixes were prepared in the contractor’s workshop. Grout properties such as stiffness and permeability are affected by many factors, such as the material composition (e.g. cement type, fineness of bentonite powder, water acidity), mixing equipment, mixing sequence and even mixing temperature. On site, despite these preliminary trials and investigations, the initial trial grout mixes were found to be too thick to be pumped through the tremie pipe. Checks were made to ensure that the materials were the same for the trial mixes at the workshop and the actual mixes on site; the same sequence of the mixing was followed; the same model of mixer was used too. A thicker grout consistency could result from small temperature differences (winter to summer), but perhaps more significantly the trial grout mixes in the workshop were not pumped through the same length of grout pipe as on site to check the pumpability. This latter effect was further compounded on site as the grout pump had additionally to overcome the hydraulic head of the wet grout within the borehole.

Further grout mix trials were performed on site, using the same materials and mixer but under the site environment and using a stronger piston pump. Successful revised grout mixes were determined, but this took time and led to delays in the overall programme of work.

3.2 Properties of trial grout mixes

Ordinary Portland cement (OPC), sodium-bentonite powder and pulverised fly ash (PFA) were used for the grout. For both the initial workshop trial and the final site trial, a mixing sequence as recommended by Mikkelsen (2002) was followed: mixing bentonite at the end after cement and fly ash. The mixing procedure started by adding a measured quantity of clean water into a mixing pan. The proportioned cement was then gradually added and mixed thoroughly, followed by adding and mixing the fly ash. Finally, the bentonite powder was slowly added, stirring the mixture constantly, to break down any lumps of bentonite that formed.

The initial workshop grout trial involved 12 mixes, of which six were not pumpable using a progression cavity pump that the contractor proposed to use on site. Samples of 100 mm dia. were prepared for the other six pumpable grouts and tested in the laboratory for unconsolidated, undrained shear strength and permeability using triaxial apparatus. The details of the grout mixes and test results are summarised in Table 1 and Figures 7 and 8.

As indicated in Table 1 and Figure 7, generally the shear strengths of the grouts are insensitive to the confining cell pressures. In the cases of mixes 1, 4 and 6 there was one sample in each set of tests that had a significantly lower strength than the others. As there is no trend in the lower strength with cell pressure and as only a single sample was prepared for each cell pressure, it is assumed that these lower strengths are not representative. In general, the grout shear strength decreases with increasing water–cement ratio (note that mixes with higher water–cement ratios (and PFA) produce grouts with higher void ratios). The average secant moduli for all the mixes determined from the same triaxial tests at 0.25% strain, as presented in Table 1, range from 18 MPa to 120 MPa and follow in general the same trend as the shear strength (note that, in practice, although compatibility of stiffness of the grout and the ground is sought, often comparisons are made in terms of undrained strength for

simplicity because of the varying nature of soil stiffness with strain). Similarly, grouts with a lower water–cement ratio and hence lower void ratio generally have a lower permeability, although the bentonite content also influences permeability, as can be seen from Figure 8. For instance, mix 6, having a lower void ratio but smaller bentonite content than mix 9, has a higher permeability, suggesting that a reduction in the bentonite content would result in an increase in the permeability.

The undrained shear strengths (measured from unconsolidated, undrained triaxial tests) of soil samples taken from six nearby boreholes in Hyde Park are plotted against the sample depth below ground level in Figure 9. The representative undrained shear strengths of grout mix 2 and mix 4 are also shown. It can be seen that the strength of mix 2 is at the weaker end of the soil shear strength values, whereas over almost the full depth to 50 m, corresponding to the base of the deepest borehole at the site, mix 4 has a larger strength than the soil. In terms of stiffness, the undrained secant moduli of the London Clay samples collected from the Crossrail ground investigation site near Paddington were measured from undrained triaxial tests to be between 20 MPa and 40 MPa at large strains (> 0.25%). Again, mix 2 has an undrained stiffness on the softer side of the soil stiffness range at large strain, whereas mix 4 is considerably stiffer than the soil. In

Mix no.	1	2	4	6	9	12
Mix proportion by mass (water: cement: fly ash: bentonite)	4.0: 1.0: 0.0: 0.72	4.0: 1.0: 0.0: 1.0	2.5: 1.0: 0.0: 0.74	2.0: 1.0: 0.0: 0.5	4.0: 1.0: 0.0: 1.0	2.5: 0.8: 0.9: 0.55
UU triaxial compression test ^a						
Curing time: d	59	58	38	38	32	32
Average sample void ratio	5.14	4.93	3.50	3.03	4.16	2.76
(Confining pressure), UU triaxial shear strength: kPa	(100) 91 (200) 56 (300) 114	(100) 152 (200) 156 (300) 166	(200) 117 (400) 421 (600) 420	(200) 702 (400) 691 (600) 489	(100) 145 (200) 146 (300) 166	(200) 398 (400) 410 (600) 375
Average E_{sec} at 0.25% strain: MPa	18	22	110	80	40	120
Permeability test in triaxial apparatus ^b						
Sample void ratio	N/A	N/A	3.57	2.84	3.72	2.34
(Consolidation pressure: kPa), coefficient of permeability: m/s	N/A	N/A	(200) 6.3×10^{-9} (400) 4.4×10^{-9} (600) 3.7×10^{-9}	(200) 1.5×10^{-9} (400) 1.2×10^{-9} (600) 8.9×10^{-10}	(100) 9.4×10^{-10} (200) 6.0×10^{-10} (300) 4.2×10^{-10}	(200) 1.3×10^{-10} (400) 7.0×10^{-11} (600) 6.0×10^{-11}

^a UU: Unconsolidated undrained triaxial compression test on 38 mm specimens, in accordance with BS1377, Part 7: 1990: Test 8 (BSI, 1990b).

^b 100 mm dia. specimens were consolidated under different cell pressures before permeability was tested under a constant pressure gradient. The tests were performed in accordance with BS1377, Part 6: 1990, Clauses 5.3, 5.4, 5.5.2.2 to 5.5.2.7 and 6 (BSI, 1990a).

Table 1. Details of trial grout mixes and properties

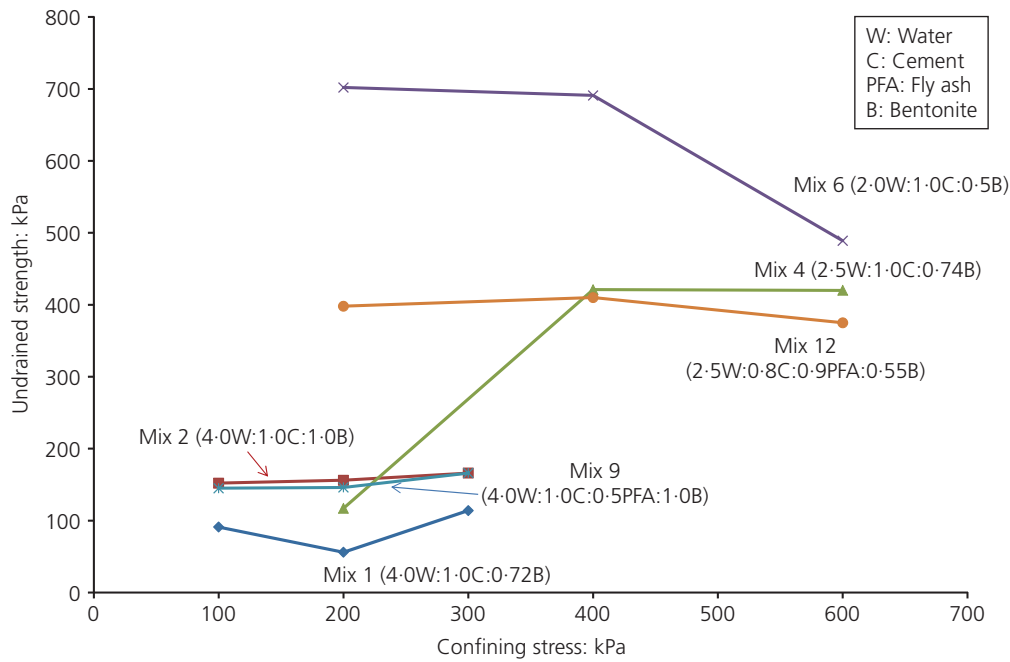


Figure 7. Unconsolidated undrained triaxial compression test on trial grouts: undrained shear strengths of 38 mm specimens of trial grout mixes

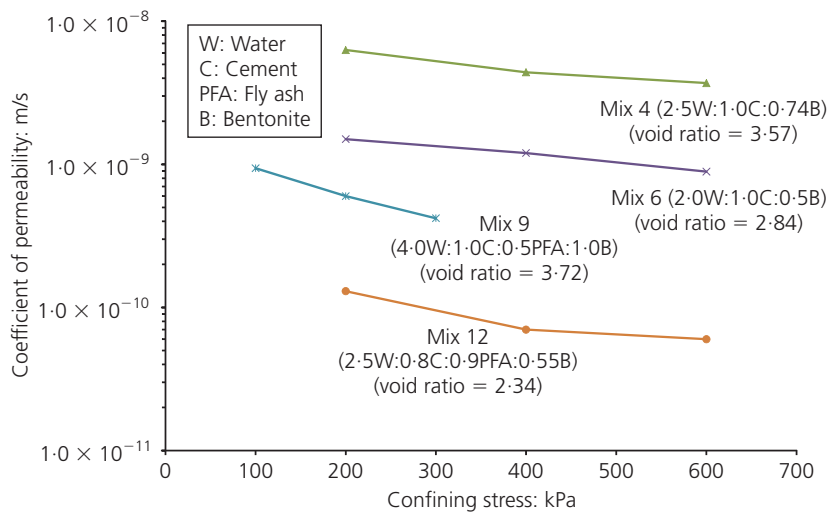


Figure 8. Permeability test on trial grouts: coefficient of permeability of trial grout mixes

view of these observations, mixes 2 and 4 were adopted as suitable design mixes for backfilling the extensometer and inclinometer boreholes respectively.

The influence of cement and bentonite content on grout strength has been investigated by Contreras *et al.* (2008). They summarise the unconfined compressive strength measured from cement–

bentonite grout samples in different projects, from both the literature and their own measurements. Figure 10 is modified after Contreras *et al.* (2008) and shows the relationship between the unconfined compressive strength and the water–cement ratio by weight, with the Hyde Park data also plotted (note that the Hyde Park data are derived from the measured, unconsolidated, undrained shear strength, S_u which theoretically equals half of the

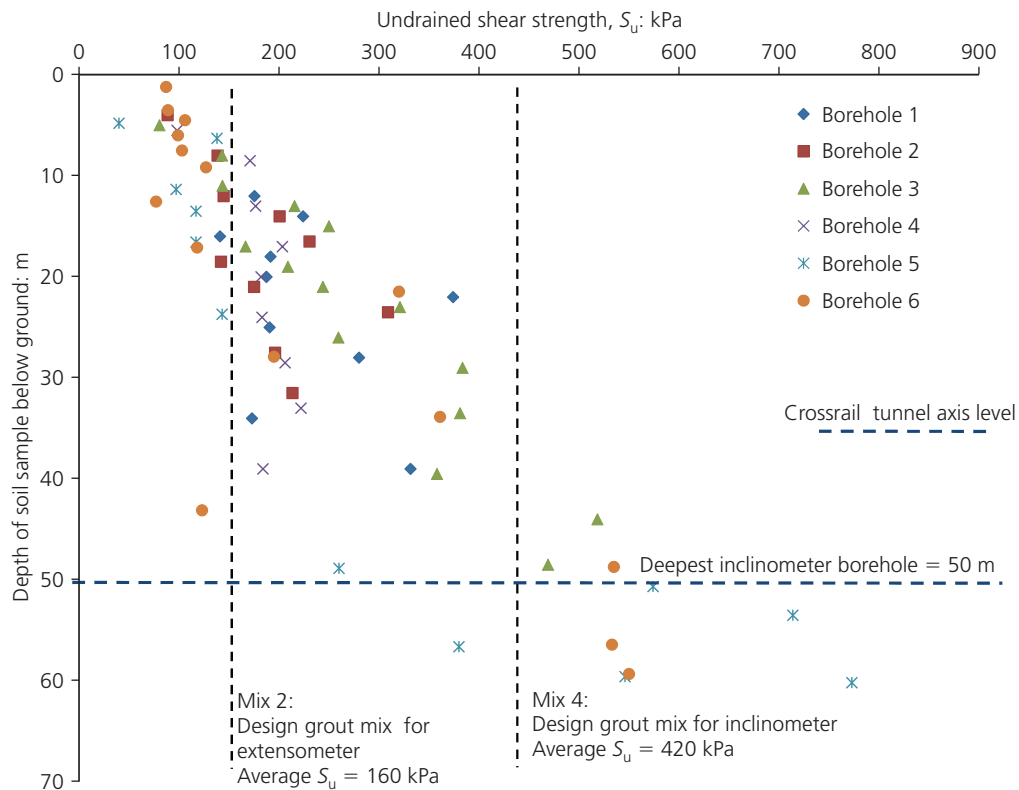


Figure 9. Undrained shear strength of London Clay borehole samples taken from Hyde Park

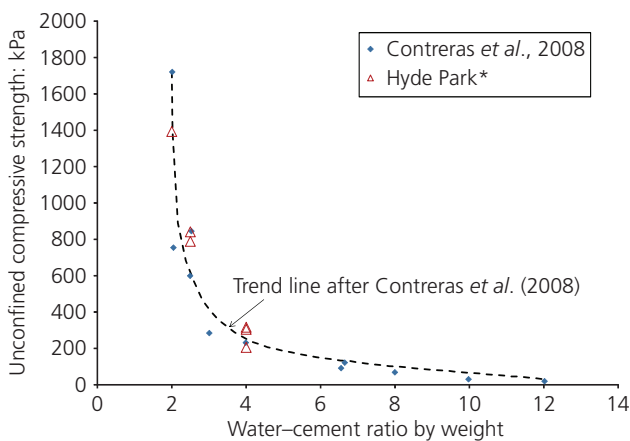


Figure 10. Measured unconfined compressive strength of cement–bentonite grout samples from different project sites (after Contreras *et al.*, 2008) (*Hyde Park data are derived from measured unconsolidated undrained shear strength)

unconfined compressive strength). All the data points lie reasonably well in one single trend line, although different types and contents of bentonite were used in these projects, suggesting that it is the water–cement ratio that is the governing parameter for the grout strength.

Mix 6, having a coefficient of permeability of 1×10^{-9} m/s, was considered to be less than 100 times more permeable than the surrounding London Clay (for which the lowest permeability was taken to be about 1×10^{-11} m/s) and therefore adopted as suitable for backfilling VW piezometer boreholes.

3.3 Grout mixes used on site

The lack of pumpability of the initial trial mixes on site using the original progression cavity pump led to modifications being made to the grout mixes for extensometer and inclinometer boreholes, primarily involving reducing the bentonite quantity and adding PFA. The pumpability of the revised mixes was tested by pumping the grouts into an abandoned 30 m borehole using a stronger piston pump. The final grout mix proportions used are shown in Table 2. Owing to the tight installation programme, no further strength tests on the modified mixes were performed. It was considered that the small additional amounts of cementitious materials used would only result in marginally increased values of strength and stiffness of the modified grout mixes which should not significantly affect instrument response and performance.

Ideally grout mix trials should be performed using the same materials and equipment on site prior to the grout backfilling. This would involve careful planning of site activities as it

Borehole backfill	Water	Cement	Fly ash	Bentonite
Extensometer ^a	4.0 (4.0)	1.0 (1.0)	0.5 (0.0)	0.9 (1.0)
Inclinometer ^a	2.5 (2.5)	0.8 (1.0)	0.9 (0.0)	0.5 (0.74)
VW piezometer	2.0	1.0	0.0	0.5

^a Bracketed values represent the initial design mix values.

Table 2. Final grout mix proportions by mass

typically takes weeks to obtain representative laboratory testing results (all grouts should be tested after a minimum 28-day period to allow full cement strength to develop). Allowance in grout pumpability should also be made to take account of the need to overcome the wet grout hydraulic head.

4. Technical challenges

Achieving research-quality instrumentation installations at times leads to additional technical and sometimes contractual challenges compared with those for more standard ground investigation and instrumentation projects. In this section, three practical challenges encountered during the works (in addition to the issues relating to grout pumpability) are discussed along with the solutions adopted.

4.1 Encountering obstructions: Thames Water pipe

Prior to drilling on Bayswater Road, several inspection trenches were excavated to expose an existing water pipe, as required by Thames Water, to determine its exact size and location. It was found to be a cast iron pipe of 600 mm diameter, with the depth of the pipe crown ranging from 0.2 m to 0.6 m below ground level in the vicinity of the boreholes. The pipe is a major water main and so Thames Water required the boreholes to be drilled in a controlled manner outside a set distance from the pipe.

A useful measure of potential vibration-induced damage can be obtained from peak ground/particle velocities (Bommer and Alarcón, 2006). In order to assess potential effects on the water main of borehole drilling, a trial was performed within Hyde Park, measuring ground vibrations within two 0.5 m deep pits excavated at 1.5 m and 3.0 m from the borehole. Measurements of peak particle velocity (ppv) were made using geophones at the base of the pits during various steps of borehole drilling. Potential mitigation measures to reduce vibration were investigated, including digging a deeper inspection pit before starting cable percussion drilling, advancing the casing by surging instead of hammering, reducing the rate of hammering and using more water to facilitate penetration of the casing close to the pipe. It was found that the maximum vibration occurred when the casing was hammered into the top of London Clay at about 5 m depth. A ppv of 4.4 mm/s was measured at 1.5 m from the borehole when the casing was rapidly struck. However, if a 2–3 s delay was allowed between each casing strike, reduced ppv values of 3.9 mm/s and 3.5 mm/s were measured at 1.5 m and 3.0 m respectively from the borehole. All of these values are far lower

than those reported in the literature for the assessment of seismically induced damage to buried pipelines; for example, the minimum value reported by O'Rourke and Ayala (1993) to have caused damage was about 100 mm/s. Nonetheless as a precautionary measure, the reduced strike rate was adopted with the borehole no closer than 3 m from the pipe. No damage to the water main was reported during the work. Clearly, when utilities pipes are expected, early engagement and close liaison with utilities companies are vitally important.

4.2 Time of completion of instrument installation

Ideally, drilling, installation of instruments and backfilling of the borehole should be completed in as short a time as possible, preferably within one day, to minimise disturbance to the ground around the borehole due to stress relief and desiccation. This helps ensure a high instrument conformance.

Finishing a 30 m or deeper borehole with instrument installation and backfilling within a day was not usually practically feasible, as there was a strict control on the working hours (8 a.m. to 6 p.m.) within Hyde Park. In such cases the installation was completed over a two-day period. A borehole would be started and left open overnight down to 30 m below ground, with the borehole being cased over the depth of Terrace Gravels, taking into account that the London Clay sub-unit B2 to this depth is relatively more stable than the underlying A3ii London Clay sub-unit (containing silt and sand partings) (Standing and Burland, 2006). The borehole drilling was completed the next morning and installation of the instruments and grout backfilling of the borehole was finished by the end of the afternoon.

Early discussions with the contractor about specific project requirements and site restraints help to achieve workable solutions.

4.3 Borehole diameters

Smaller diameter boreholes usually are more advantageous than larger ones, because faster, more effective drilling can be achieved with less disturbance to the ground. On site, 200 mm boreholes were drilled initially, although the instruments were able to fit into 150 mm boreholes. The main reason was to cover the risk of abandoning the borehole in case of borehole wall collapse: with a 200 mm cased borehole it is possible to continue drilling with 150 mm casing. If the borehole were to be started with 150 mm casing, there would be a risk of abandoning the

whole borehole if a borehole collapse occurred beyond the contractor's control.

Drilling rates are much slower for 200 mm compared with 150 mm boreholes, especially when stiffer strata or claystones are encountered. In order to minimise the ground disturbance and improve the construction progress, it was agreed that an engineer's instruction should be given to the contractor to drill boreholes of 150 mm dia. Any abandoned hole and re-drilling due to borehole collapse induced by factors uncontrollable by the contractor would be payable works. This was a re-allocation of the risk from the contractor to the client. Based on records and experience of drilling in the Hyde Park area, the chance of encountering water strike was deemed to be low. No collapse within any of boreholes occurred at the site during the works.

5. Preliminary monitoring results

In this section selected data from measurements made using the VW piezometers and extensometers are presented to provide basic examples of the monitoring data.

5.1 Initial VW piezometer measurements

Results from measurements of pore-water pressures made during the early weeks after VW piezometer installation in borehole HP32 (see Figures 2 and 3(c) for location) in Hyde Park are shown in Figure 11. Negative excess pore-water pressures around the boreholes were generated from stress relief and also shearing of the over-consolidated London Clay during the borehole drilling. The equilibration of the excess pressure was complete about 1 month after instrument installation.

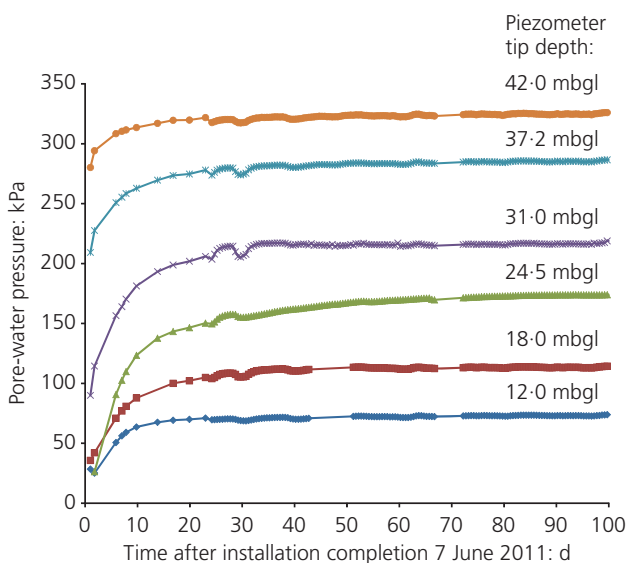


Figure 11. Initial monitoring results of VW type piezometers from borehole HP32 (mbgl: metres below ground level)

Figure 12 shows how the negative excess pressures dissipated with time at different depths for HP32, resulting eventually in the measured pore-water pressures developing a near-hydrostatic profile. Also plotted in Figure 12 are the post-equilibration pore-water pressures at the two response zones of a nearby standpipe piezometer HP31A (6 m apart), which shows that both the VW piezometer borehole and standpipe piezometer borehole are measuring the same pore-water pressure profile. The different magnitudes of negative excess pressure measured at different depths of the boreholes indicate that the grout backfill formed a good seal between the piezometer units: otherwise similar magnitudes of negative excess pressure would have been expected. Subsequent measurements of pore-water pressure change during the Crossrail TBM passage showed that their magnitudes decreased with the distance of the piezometer units from the TBM, also indicating a good functioning of the grout seal between the piezometer units within the boreholes.

5.2 Immediate surface settlement at extensometer boreholes

In the case of an extensometer borehole, the potential adverse effect of a grout backfill stiffer than the surrounding soil would be that the grout column deforms less, making the settlement measured at the extensometer reference head appear smaller than that of the ground. At the Hyde Park site, as shown in Figure 2, an array of extensometer boreholes runs parallel to a line of shallow surface monitoring points (line Y) with a separation of 5 m. Each surface monitoring point comprises a 1.5 m deep, 100 mm dia. concrete column with a monitoring point installed in the top. Thus a direct comparison can be made between the surface settlements measured at the extensometer reference heads and at the surface monitoring points, when the two TBMs passed beneath them. For each tunnel construction, the TBM control was essentially the same over the 5 m distance between the two lines of instruments and clearly ground conditions are the same. The immediate incremental settlement troughs induced by construction of the first and second TBMs measured by precise levelling along these two lines of instruments are shown in Figures 13(a) and 13(b) respectively. The measured magnitudes of surface settlement at every offset position of the instruments lying transverse to the TBM axis are essentially the same, suggesting that the grouted extensometer boreholes conformed well with the surrounding ground.

6. Summary and conclusion

As part of a collaboration field monitoring research project involving Imperial College and Crossrail, high-quality borehole instrumentation was successfully installed in Hyde Park and Bayswater Road. Reasons for instrument selection are given and discussed in the paper. An important factor for achieving good conformance of the instrumentation to the surrounding ground conditions is the choice of grout for backfilling the boreholes. Ideally for extensometers and inclinometers the grout should have a compatible stiffness to the ground. Equally grout and ground permeabilities should be compatible for piezometer

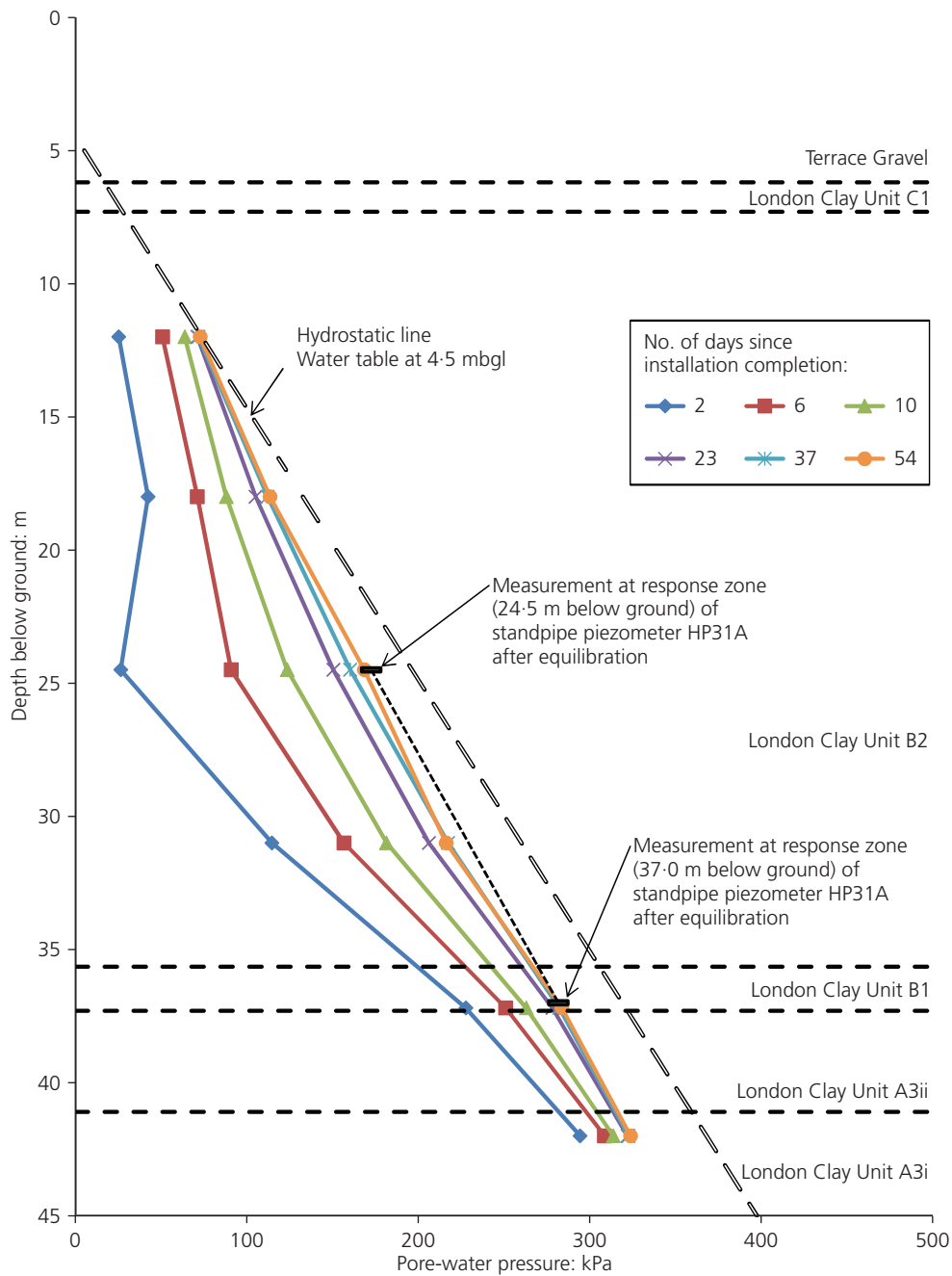


Figure 12. Measured pore-water pressure profile along the depth of borehole HP32

installations. A number of grout mixes were trialled and the final choices were made according to strength and permeability criteria, as well as practical mixing and pumping considerations. Other issues that had to be taken into account are also described (sensitivity of a cast iron water main, time constraints and optimum borehole diameter). Example data are presented to confirm good operation of instruments and selected grout for backfilling them.

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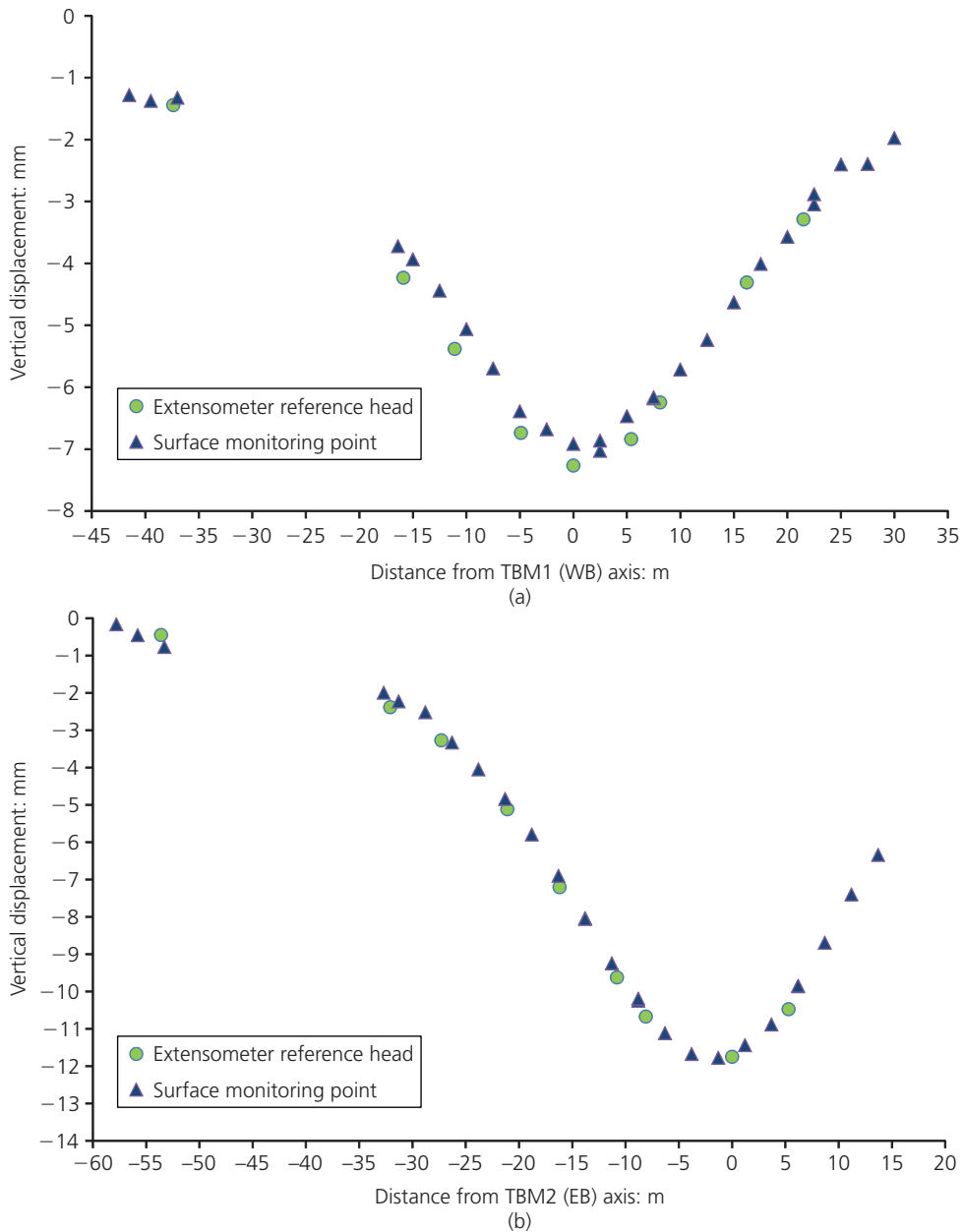


Figure 13. Measured incremental surface ground settlement at extensometer reference heads and surface monitoring points for: (a) first TBM (west-bound); (b) second TBM (east-bound) (note: negative values of vertical displacement indicate settlement)

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