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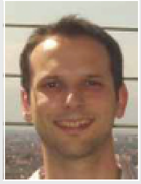

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Low-volume-loss tunnelling for London ring main extension

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The Thames Water ring main extension is a 4·5 km long tunnel from Stoke Newington, in the London borough of Hackney, to New River Head in Finsbury, in the London borough of Islington. The 2·85 m i.d. tunnel was excavated by an earth pressure balance tunnel-boring machine (TBM) at depths between 40 and 60 m below the surface. Surface settlements along the route were measured by precise levelling, and were found to be small. It was therefore even more important to measure these settlements as accurately as possible, in order to provide informed estimates of subsurface movements induced in third-party underground structures much closer to the tunnel horizon. Because of the relatively large magnitude of the background movements measured, when compared with the small tunnel-induced settlements, it was necessary to adopt a rigorous statistical method to fit a Gaussian curve to the data. This exploited the analogy of the 'error function' to define the Gaussian curve parameters i and V_l. In all, 13 tunnels were underpassed successfully by the TBM, all within the 'conservative expected value' predictions, and without incident. The predictions and structural monitoring schemes undertaken for the High Speed 1 tunnels near Corsica Street and the Northern line tunnels near Angel station are described in the paper. It was found that the surface and subsurface trough width parameter K did not vary with depth as predicted: therefore a new relationship is proposed.

NOTATION

A_f excavated face area of tunnel
i trough width, point of inflexion of Gaussian settlement curve
j point number within transverse settlement array
K trough width parameter
n number of points in transverse settlement array, or sample size
S_j settlement at point j
S_{max} maximum centreline settlement
V_l volume loss
V_{lm} measured volume loss
x offset from centreline
x_j offset from centreline at point j
z depth below ground level
z_0 depth to tunnel axis of tunnel under construction
σ standard deviation

1. INTRODUCTION

The Thames Water ring main extension (TWRM) is a 4·5 km long tunnel from Stoke Newington, in the London borough of Hackney, to New River Head in Finsbury, in the London borough of Islington. The 2·85 m i.d. tunnel was excavated by an earth pressure balance (EPB) tunnel-boring machine (TBM) at depths between 40 and 60 m below the ground surface. The excavated diameter was 3·362 m. Within the tailskin, a 180 mm thick steel-fibre-reinforced, precast concrete bolted segmental lining was erected. The alignment and geology are described in Section 2.

Surface settlements were measured by precise levelling in order to provide feedback on the tunnelling process, and to inform predictions of subsurface settlements of the 13 third-party tunnels that were going to be underpassed by the TBM. These measurements, and how they were interpreted, are described in Section 3. The trough width was consistently found to be narrower than predicted.

The monitoring of the High Speed 1 (formerly known as the Channel Tunnel Rail Link) tunnels near Corsica Street and the Northern line tunnels near Angel station is described in Section 4. The results are compared with methods of subsurface settlement prediction, and the trough width was again found to be narrower than predicted.

In the discussion in Section 5 the surface and subsurface settlements are put into context by comparing the results with previously published case history data, model tests and empirical relationships in the literature. In particular, the variation of trough width parameter K with depth was not adequately described by current empirical relationships, and surface and subsurface trough width parameter values could be smaller than predicted when a tunnel is excavated this deep. A new relationship between trough width parameter and depth is proposed in Section 5 that fits well with both the new data from this project and previously published case histories and model tests. It is formulated in a simple way, with the intention that, as new case histories are added, the best-fit line may be redrawn. This will become of more interest as the vertical alignments of new tunnels are set ever lower to avoid existing underground infrastructure.

2. LOCATION AND GEOLOGY

A plan and long section of the TWRM tunnel and the expected geology interpreted from the borehole logs are illustrated in Figure 1 (Newman *et al.*, 2010). The actual geology encountered during the tunnel drive is also shown. On the whole the agreement between predicted and encountered geology was good. Figure 1 shows the tunnel to have been driven mainly through the sandy clays and clayey sands of the Upnor Formation. The changes in geology had little impact on the tunnel construction programme, thanks to timely changes to the cutterhead teeth configuration and the redesign of the scrapers/loaders early on in the drive. TBM penetration rates were fairly consistent at around 50–100 mm/min, except through the Mottled Clay of the Lambeth Group, which slowed penetration rates to around 30–50 mm/min. None of the six head interventions required compressed air, and at no point did groundwater pressures cause problems during the tunnel drive.

3. SURFACE SETTLEMENTS

Surface settlements were monitored by precise levelling of either fixed single points or fixed points arranged in transverse arrays. This gave feedback on the performance of the tunnelling process, and provided input data for the subsurface settlement predictions required for the third-party tunnels under which the TWRM tunnel was passing. The following section describes the methods and procedures adopted for the precise levelling along the route, assesses the repeatability of the readings obtained, and discusses the possible sources of the variability encountered. Then Gaussian settlement trough parameters i and V_l are calculated from the data, allowing estimates of subsurface ground movements to be made.

3.1. Precise levelling procedure

A Leica DNA03 level was used with a bar-coded Invar staff. Over the length of a levelling loop for a typical array this should result in a repeatability of less than ±0·1 mm under controlled conditions, according to the instrument manual (Leica Geosystems, 2009). However, a range of other factors such as ambient temperature, heavy traffic, sunlight heating the road or pavement surfacing and near-surface pore pressure changes due to rain or tree root suctions will result in a worse repeatability than this. These factors result in background movements, some of which will affect each monitoring point randomly and independently, and some of which will affect an array of monitoring points in a similar manner.

Road nails were installed in the road surface or pavement for use as monitoring points. Two benchmarks were used for each single point or array, one either side of the tunnel alignment, and at least 50 m from the tunnel centreline to ensure they were outside the zone of influence. Monitoring point levels were then baselined to both benchmarks using a weighted average depending on relative distance. This reduced

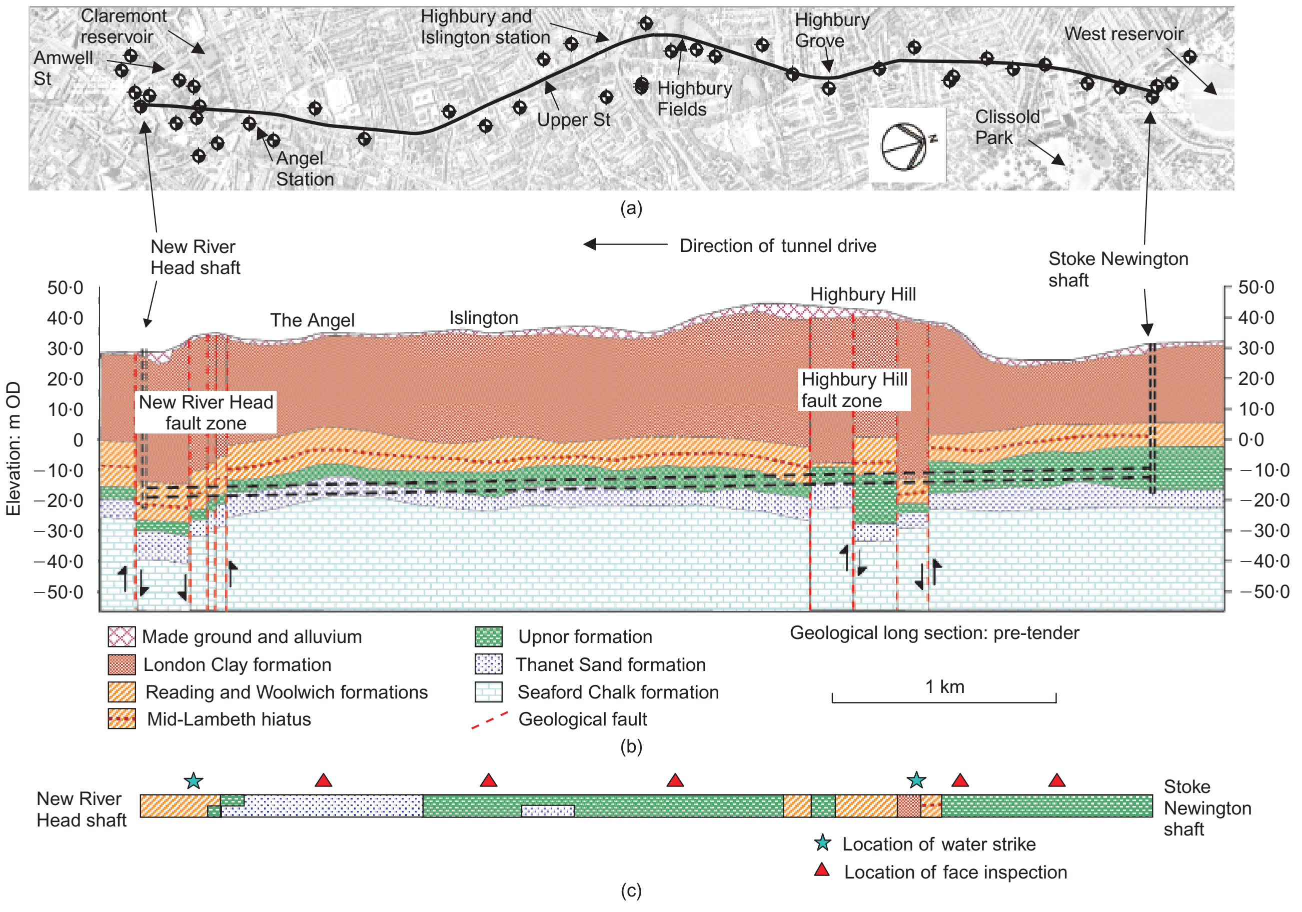

Figure 1. Plan of alignment and geological section: (a) aerial plan, showing alignment and borehole locations; (b) geological long section, pre-tender; (c) spoil encountered during tunnel drive

systematic measurement errors, and meant that if one benchmark was damaged, removed or parked over, levels could still be obtained.

3.2. Repeatability and variability in the field

To assess the true repeatability in the field, one can look at readings taken before the TBM was close enough to have an effect. The variation from the baseline readings of subsequent readings, taken before the TBM was within 50 m of the monitoring points, is shown in Figure 2. Each of these readings consists of both a measurement error and background movements caused by the aforementioned environmental effects. A normal distribution curve with the same mean and standard deviation as the data is also shown. The standard deviation of the dataset shown in Figure 2 is 0·227, for a sample size of 142. Therefore, for a confidence level of 95% (2 standard deviations either side of the mean), the repeatability of the precise levelling was approximately ±0·5 mm.

In the author's experience, usual practice is to fit a Gaussian settlement curve to the data 'by eye'. However, this method was not satisfactory in this case, since it was impossible to have confidence that the Gaussian curve parameters i and S_{max} were correct, owing to the magnitude of the variability of level data relative to the magnitude of the tunnel-induced settlements, which were typically between 0 and 2 mm. It is difficult to objectively give equal weight to all points, and there is a tendency to make the curve pass through the centreline settlement when it has just as much chance of variability as the other points. New and Bowers (1994) used a regression analysis varying maximum settlement S_{max} and trough width i. However, because a regression analysis takes the squares of the differences between the data and the hypothesised Gaussian curve, it would give undue weight to larger variances when, as in this case, the variability of the level data is not negligible relative to the magnitude of the tunnel-induced settlements.

For this project a method of Gaussian curve-fitting was used that exploited the analogy of the 'error function' to define parameters. The aim was to take advantage of the fact that the sum, mean or standard deviation of several points that are subject to random variations is likely to be more accurate than any of the points taken individually, since the variations will to some extent cancel out. If the variations are random and independent, then the mean of an array of points will follow a normal distribution, with the mean equal to the population mean and the standard deviation equal to the population standard deviation divided by the square root of the sample size n. So for an array with nine points the error in a direct calculation of volume loss is one third of the error of each point. Therefore it was important to establish, first of all, whether the variations were in fact random, so that this property could be exploited.

As a first step, a statistical normality test was performed on the data shown in Figure 2 to establish whether they are well modelled by a normal distribution. The normal distribution curve plotted in Figure 2 is based on the standard deviation and mean of the 142 readings that are represented by the histogram. The coefficient of determination r^2 is 0·94, which indicates that the data are well modelled by the normal distribution.

Although it may appear from Figure 2 that the variations must be random, because they follow a normal distribution, lumping all the data from many levelling runs together will hide systematic errors and environmental effects that affect a single levelling run in a non-random manner, for example misreading of a benchmark (a systematic error) or movement of a benchmark (an environmental effect). In order for the variations in the data presented in Figure 2 to be random, the mean of the changes from the baseline for each individual levelling run should be close to zero. In general this was the case, with the mean of the absolute values of the mean changes equal to 0·08 mm, with a standard deviation of 0·05 mm, for all the levelling runs that were within one week of the baseline readings. However, as more time elapsed between the baseline readings and subsequent readings, the

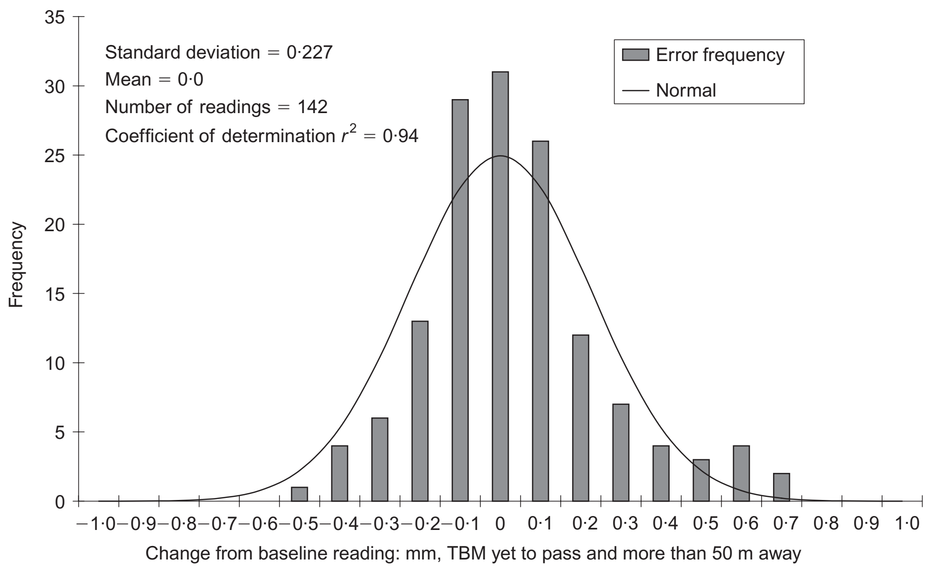

Figure 2. Repeatability of surface settlement monitoring data

background movements could in some cases grow larger, and the mean absolute change could be up to 0·5 mm for a single levelling run. These were most likely due to environmental effects, since the instrument was calibrated regularly, and the movements were not of a pattern that suggested equipment drift could be the cause. It was therefore considered important to ensure that the baseline readings were checked just before the TBM entered the zone of influence.

The surface settlement data are shown in Figures 3 to 11. The method used to fit the Gaussian curves to the data will be described in the following section. For ease of reference the arrays are numbered with the approximate chainage, and the chainage runs in the opposite direction to the TBM drive direction, that is, from New River Head to Stoke Newington. The key to the markers also gives the distance of the TBM cutterhead from the array (negative is before the array, positive is past the array), and the date and time the readings were taken. Note that the scale for the settlements is different in Figure 3 (array 4500) and Figure 9 (array 2470) compared with all the other graphs. Where baseline readings taken before the TBM is within 50 m of the array have changed, the latest values have been used and previous readings omitted from the graphs. In Figure 3 (array 4500), Figure 7 (array 3060) and Figure 9 (array 2470) the effects of environmental background variations over time on at least some of the points in the arrays can be seen. In Figure 4 (array 4300), Figure 5 (array 4100) and Figure 10 (array 2150) there is evidence that the monitoring points were stable over a time period of several weeks. In Figure 6 (array 3975) the monitoring points remained stable for over a year after construction.

3.3. Calculation of surface settlement trough parameters

When a normal distribution or 'Gaussian curve' is used to represent ground movements due to tunnelling, by analogy the standard deviation or the point of inflexion is the trough width i, and the frequencies are the settlements. Therefore the trough

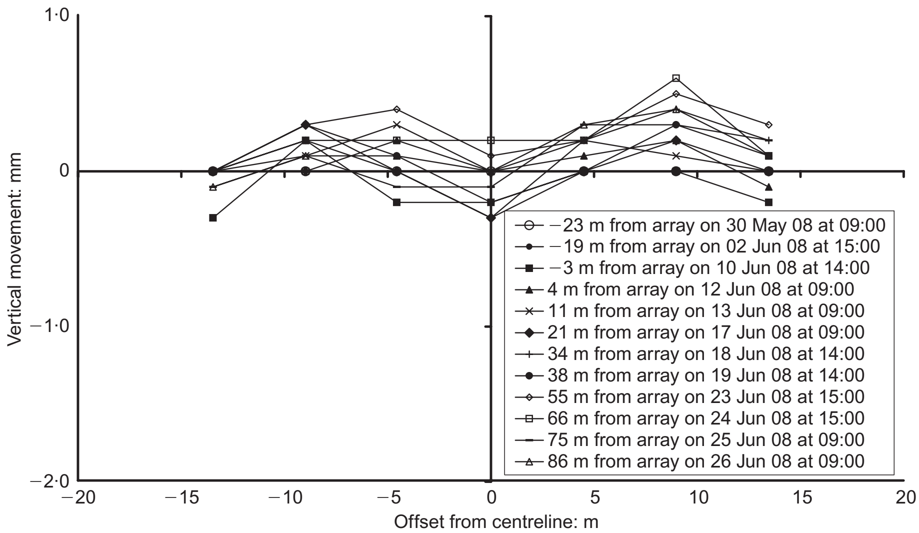

Figure 3. Array 4500 surface settlement monitoring data

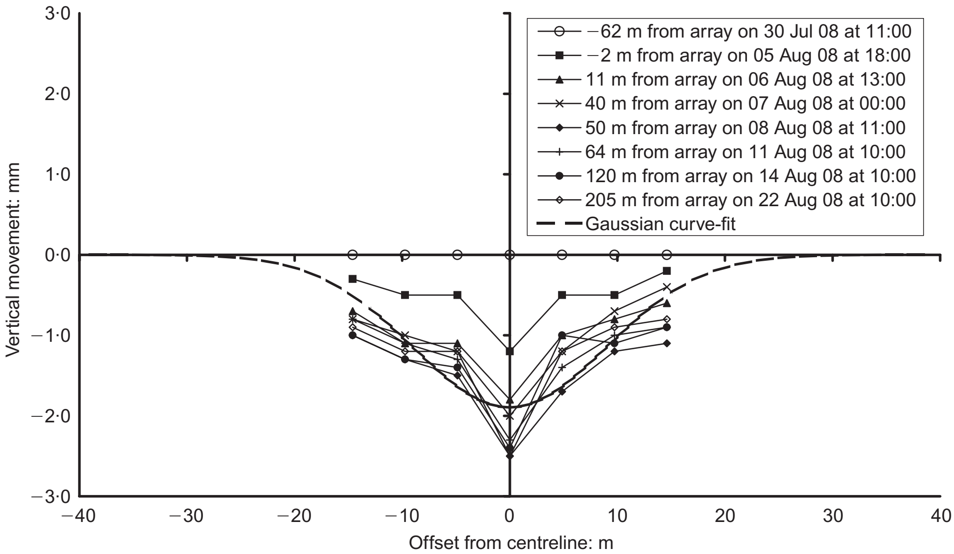

Figure 4. Array 4300 surface settlement monitoring data

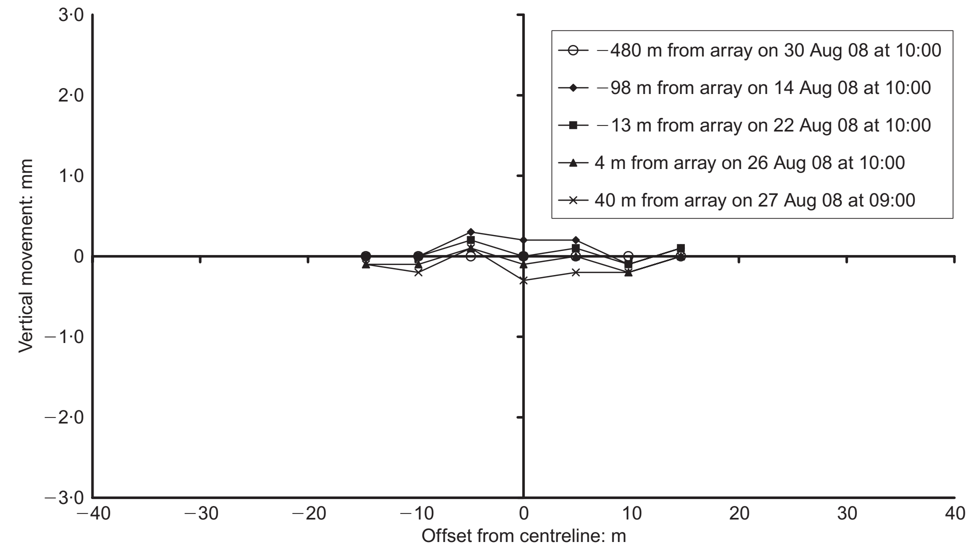

Figure 5. Array 4100 surface settlement monitoring data

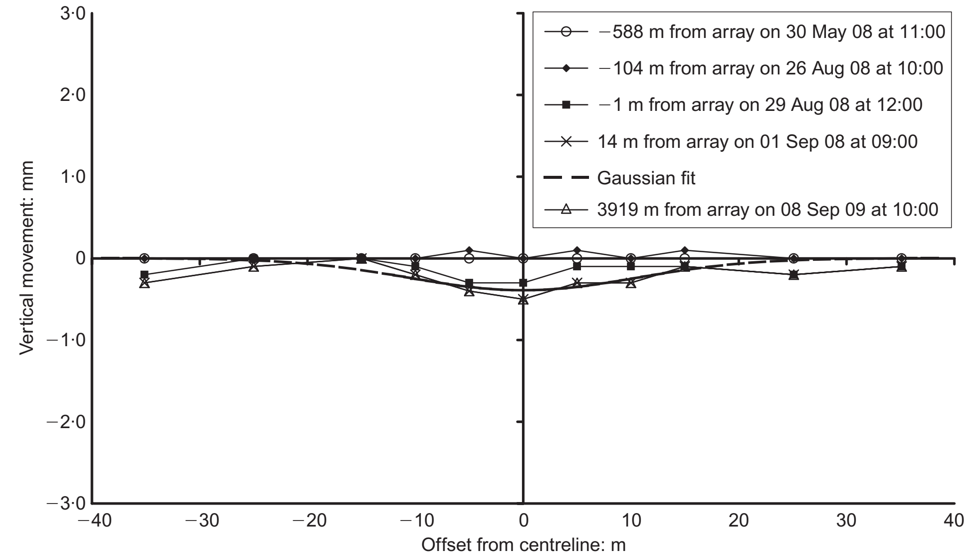

Figure 6. Array 3975 surface settlement monitoring data

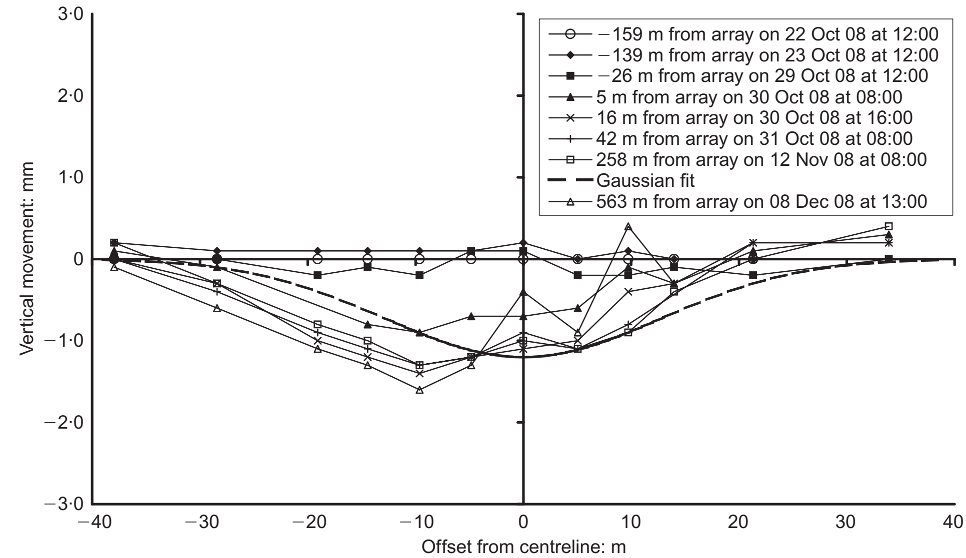

Figure 7. Array 3060 surface settlement monitoring data

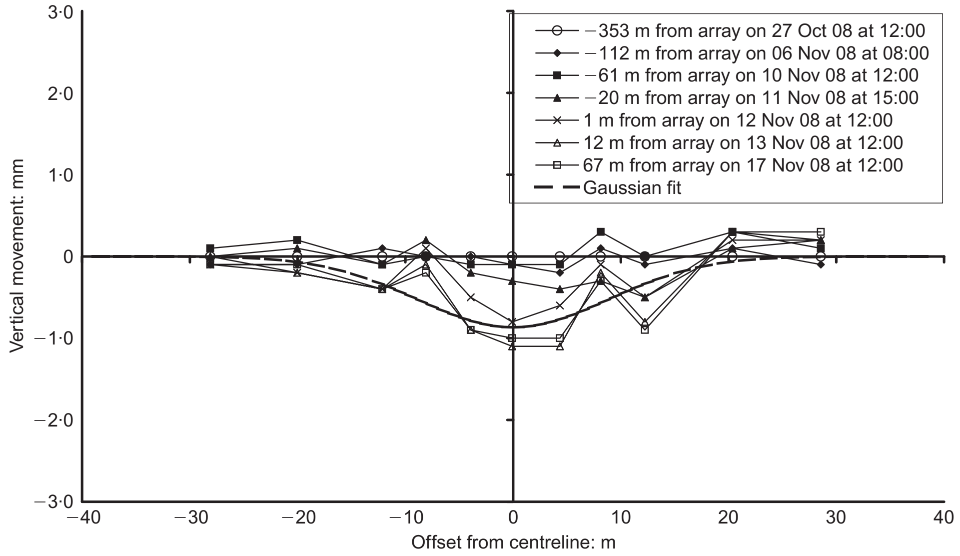

Figure 8. Array 2800 surface settlement monitoring data

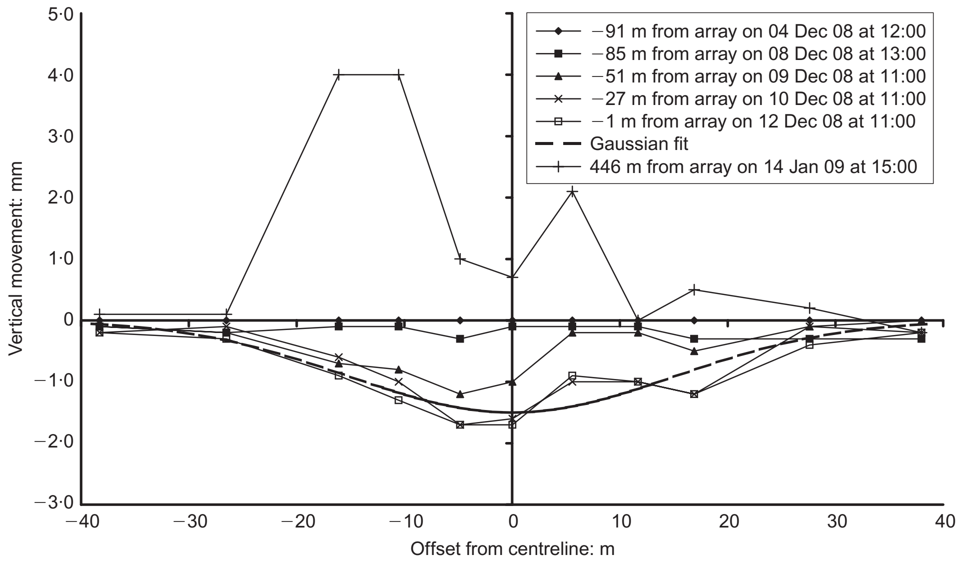

Figure 9. Array 2470 surface settlement monitoring data

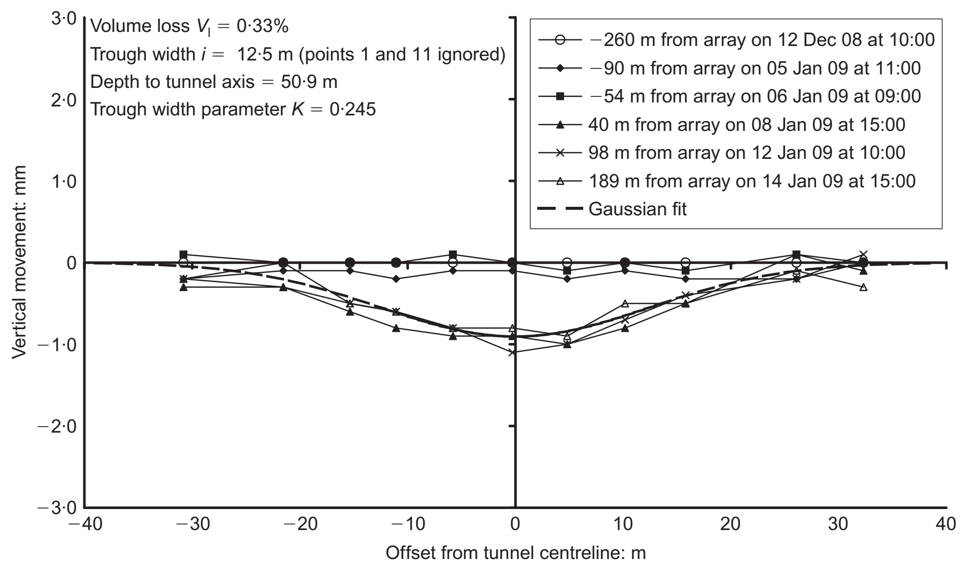

Figure 10. Array 2150 surface settlement monitoring data

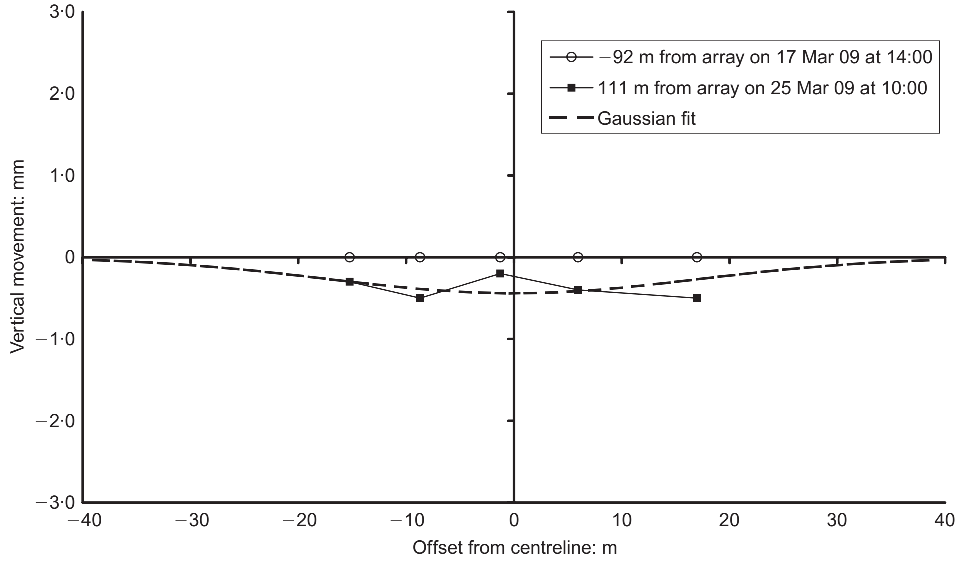

Figure 11. Array 730 surface settlement monitoring data

width may be calculated directly from the data by calculating the standard deviation about a mean assumed to be at the centreline of the tunnel. The volume loss is also required to define the curve, and this was calculated by trapezoidal integration of the settlements over the extent of the array and correcting for any missing tails.

The standard deviation σ, or the trough width i, is given by

$$\sigma = \sqrt{\frac{\sum_{j=1}^{n}\left(x_j^2 S_j\right)}{\sum_{j=1}^{n}(S_j) - 1}} \tag{1}$$

where j is the point number from 1 to n, n is the number of points in an array, x_j is the transverse distance of point j from the tunnel centreline, and S_j is the measured settlement of point j.

This equation is sensitive to errors at large offsets, so judgement should be exercised to exclude errors at large offsets, where the settlement should be negligible.

The measured volume loss V_{lm} is given by

$$V_{\mathrm{lm}} = \frac{\sum_{j=1}^{n-1}\left\{\left[(S_j + S_{j+1})/2\right](x_j - x_{j+1})\right\}}{A_{\mathrm{f}}} \tag{2}$$

where A_{f} is the excavated face area of the tunnel.

The numerator of the quotient in Equation 2 is basically a trapezoidal integration of the settlement data. If the data do not cover the whole of the settlement trough, for example because the trough width was larger than expected, or because points could not be installed owing to the presence of buildings or other obstructions, the actual volume loss V_{l} may be estimated by using the equation

$$V_{\mathrm{l}} = \frac{V_{\mathrm{lm}}}{\int_{-\infty}^{x_n}\left(1/\sqrt{2\pi\sigma}\right)\mathrm{e}^{-\left(x^2/2\sigma^2\right)} - \int_{-\infty}^{x_1}\left(1/\sqrt{2\pi\sigma}\right)\mathrm{e}^{-\left(x^2/2\sigma^2\right)}} \tag{3}$$

where the two terms in the denominator represent the unit cumulative distribution from minus infinity to x_1 and x_n: that is, the denominator is the proportion of the total volume loss that is within the limits of the array.

The settlement data for array 2150 at the northern end of Highbury Fields is shown in Table 1 for the levelling run on 14 January 2009. The calculated volume loss was 0·33%, and the trough width was 12·5 m. The outermost points 1 and 11 were not included in the calculation of trough width since, when multiplied by the square of the offset, the small variations, which were probably not tunnel-induced, contributed disproportionately to the trough width calculation. The Gaussian curve can be seen in Figure 10.

	Monitoring point No. j										
	1	2	3	4	5	6	7	8	9	10	11
Settlement: mm	−0·3	−0·1	−0·5	−0·5	−0·9	−0·8	−0·8	−0·6	−0·5	−0·3	−0·2

Table 1. Array 2150 levelling run on 14 January 2009

3.4. Surface settlement Gaussian curve parameters

Table 2 is a summary of the settlement trough parameters calculated from the surface settlement monitoring arrays using the curve-fitting method described in the previous section.

Single points were also installed between arrays, and were located on the tunnel centreline. The maximum settlements are presented in Table 3.

Tables 2 and 3 indicate that the likely range of volume loss is 0·0–0·5%. The one array that exceeds this is array 2470. It is quite probable that up to 17 m of ground beneath it was disturbed prior to TWRM tunnelling by the construction of the Network Rail Canonbury tunnel, a Victorian brick arch tunnel that to one side of the TWRM alignment was constructed by the cut and cover method and to the other side was bored. Further, visual inspections of the Canonbury tunnel indicated that it also behaved as a drain on the ground above it, with significant volumes of water dripping onto the tracks from the crown after a rainfall event. There also appeared to be significant background movements in this area, possibly caused by the large trees lining the path in which the array was installed, as evidenced by the subsequent heave shown in 14 January 2009 data in Figure 9.

Point	Maximum settlement: mm	Depth to tunnel axis: m
3800-1	0·6	41·8
3800-2	0·2	42·2
3650-1	1·9	46·9
3650-2	1·4	47·2
3540-1	2·0	53·3
3540-2	2·1	53·1
3490-1	1·2	55·6
3400-1	1·9	55·8
3300-1	1·8	56·8
3130-1	0·7	58·1
2660-1	0·6	58·6
1940-1	0·0	52·4
1540-1	1·0	52·2
1330-1	0·4	53·2

Table 3. Maximum settlements of centreline monitoring points

4. MONITORING OF THIRD-PARTY TUNNELS

Only the monitoring data from the HS1 tunnels and Northern Line running tunnels are described in detail in the following sections, since they were the only tunnels for which high-quality settlement data were obtained. Their chainages and depths are listed in Table 4. The three Network Rail tunnels and the four Victoria line tunnels that were underpassed had at least 30 m of clear ground between the crown of the TWRM tunnel and their invert levels, and had relatively generous tolerances for settlement compared with the predicted values, even at a worst case of 1·5% volume loss. And so on the basis of a thorough and detailed risk assessment, only condition surveys, visual inspections and gauging surveys (in the case of the Victoria Line) were carried out. The 200-year-old British Waterways Islington Tunnel on the Regent's Canal was monitored, but no tunnel-induced movements were detected. The Angel London underground station upper escalator, the longest escalator in Western Europe, was also monitored, but only background movements were detected. These will not be described further.

4.1. Real-time monitoring of the High Speed 1 tunnels

The High Speed 1 (HS1) rail tunnels were underpassed approximately 200 m west of Corsica Street shaft. A plan and section of their location relative to the TWRM tunnel are shown in Figure 12. The HS1 tunnels have an internal diameter of 7·15 m, and consist of a precast reinforced concrete segmental bolted lining 0·35 m thick. The HS1 tunnels were situated with the crown of the tunnel in the London Clay and the invert of the tunnel in the Lambeth group Upper Mottled Clay. The TWRM tunnel was in the Upnor formation: hence both tunnels could be considered to be in cohesive ground. The clear ground distance between the TWRM tunnel crown extrados and the HS1 tunnels invert extrados was 11·7 m. The HS1 down-line tunnel was underpassed first, and the distance between the axes of the down and up lines was 17·3 m. Both tunnels were underpassed at an angle of approximately 75°.

This was the first underpassing of a high-speed rail tunnel in the UK, and at typical TBM advance rates of 30–40 m/day the

	Date of data used	Volume loss, V_l: %	Trough width parameter, K	Maximum settlement, S_{max}: mm	Depth to tunnel axis: m
Array 4500	23–26/6/08	0	Not measurable	0·0	42·0
Array 4400		Unstable benchmark while TBM passing			39·1
Array 4300	22/8/08	0·50	0·233	1·9	38·5
Array 4100	27/8/08	0	Not measurable	0·0	37·9
Array 3975	1/9/08	0·12	0·277	0·4	38·2
Array 3060	12/11/08	0·52	0·223	1·2	58·5
Array 2800	17/11/08	0·23	0·151	0·9	58·8
Array 2470*	12/12/08	0·67	0·280	1·5	54·1
Array 2150	14/1/09	0·33	0·245	0·9	50·9
Array 730	25/3/09	0·22	0·343	0·4	50·1
Array 530		Unstable benchmark while TBM passing			52·0

*Array 2470 Gaussian curve parameters were calculated when the cutterhead of the TBM was under the array. The next set of readings, after the Christmas break, had experienced significant background movements (Figure 9). The TBM was 'parked' over Christmas approximately 200 m past this array.

Table 2. Calculated surface settlement parameters

Third-party structure	Crossing chainage: m	Depth below ground level of third-party tunnel axis, z: m	Depth below ground level of TWRM tunnel axis, z_0: m
HS1 down-line tunnel (to Paris)	2179	34·1	51·4
HS1 up-line tunnel (to St Pancras)	2162	34·1	51·4
Northern line southbound running tunnel	520	35·9	51·8
Northern line disused tunnel	514	36·3	51·8
Northern line northbound running tunnel	500	36·2	51·8

Table 4. Chainages and depths of selected third-party tunnels underpassed by the TWRM tunnel

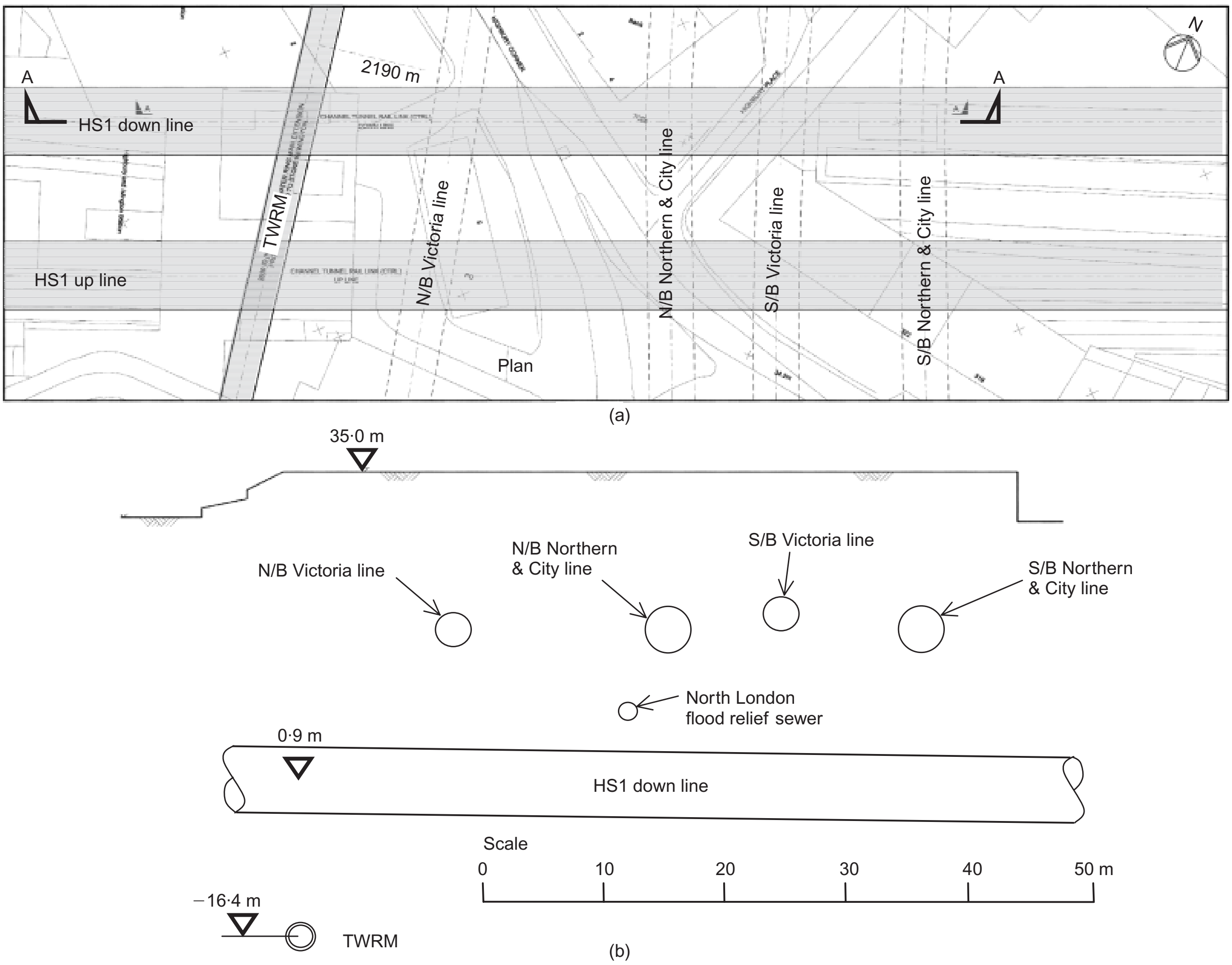

Figure 12. (a) Plan and (b) section A–A of the HS1 tunnels crossing

two tunnels would be underpassed in a matter of hours. Therefore real-time monitoring was required to identify out-of-tolerance movements of the track immediately, and if necessary impose speed restrictions until the track could be reset. Therefore trigger and alarm values with well-defined actions were agreed in advance between all parties, to ensure there was no risk to the safe operation of the railway. A designated competent person monitored and evaluated all the data in real time, and as a further level of protection automated text message alerts were sent to the TWRM site management and to Network Rail (CTRL).

The predicted subsurface movements were initially based on the method described by O'Reilly and New (1982), using a constant value of trough width parameter K of 0·45. The trough width i is related to the trough width parameter K by the expression

$$i = K(z_0 - z) \tag{4}$$

where z_0 is the depth to the tunnel axis of the tunnel under construction, and z is the depth to the existing tunnel's axis.

The method of O'Reilly and New therefore assumes constant-volume plane-strain conditions, with movement vectors directed towards the tunnel under construction's axis, and a trough width varying linearly with depth. The settlement of the existing tunnels was assumed to be governed by the subsurface ground movement at the existing tunnel's invert level, and the structural stiffness of the tunnel was not considered. This resulted in a predicted maximum settlement of 2·86 mm and trough width of 5·97 m, at the conservative expected value of volume loss of 0·5%.

Mair *et al.* (1993) found in centrifuge tests that the trough width could vary non-linearly with depth, being wider than predicted by O'Reilly and New as one approached the tunnel under construction. They used the trough width parameter K to describe this, and they found that K varies with depth according to the relationship

$$K = \frac{0{\cdot}175 + 0{\cdot}325(1 - z/z_0)}{1 - z/z_0} \quad (5)$$

Mair *et al.* (1993) compared this relationship with several case studies, and then Mair and Taylor (1997) added further data points from more recent projects. This relationship was then compared by Standing and Selman (2001) with data obtained by monitoring existing tunnels underpassed by the Jubilee Line Extension, and was found to have a reasonably good correlation. Therefore this relationship appears to describe subsurface settlements well, and these are similar to the settlements of an existing tunnel due to the construction of a new tunnel beneath. The predicted HS1 settlement trough width using this method was 14·62 m, and the maximum settlement was 1·17 mm. This wider, shallower trough was clearly less onerous in terms of structural distortions and track movements than the narrower, deeper trough predicted using the method of O'Reilly and New.

The tolerances for high-speed track depend on the speed limit through the section in question. For this location the trigger levels shown in Table 5 were set, based on a standard distance of 35 m. The action and speed restriction values in Table 5 were not very much larger than the predicted maximum vertical track movement, and if volume loss was higher than expected there was a real, though unlikely, risk of having to impose speed restrictions. The speed restriction value for maximum settlement was reduced from ±8 mm to ±7 mm to account for the accuracy of the monitoring system, and the possibility of settlement at the ends of the string that would not be captured in real time.

The real-time monitoring was achieved by using tilt sensors mounted on aluminium beams, bolted at each end into the concrete track slab. A 32 m long string of 2 m long beams running parallel to the rails was used to monitor settlement, and 1 m long beams between the rails at 3 m spacing were used to monitor twist between the rails. Readings were taken every minute and transmitted via 'Paknet', a wireless radio communications system with 99·999% availability, to a web-based graphical display. Text messages would be sent automatically to site staff and to Network Rail (CTRL) staff if trigger levels were exceeded.

Tilt sensors consist of a small circuit board mounted securely within a protective box attached to the centre-top of an aluminium square hollow section beam. A chip on the circuit board measures the angle between the orientation of the chip and the gravitational vector. If the chip rotates, a change in this angle is measured. Using this equipment, the difference in settlement between one end of the aluminium beam and the other may be deduced, with an accuracy of ±0·01 mm. In order to calculate settlements these differences in settlement need to be added together along the string from one end to the other. To minimise the effect of accumulated errors along the string, both ends of the string were assumed to be fixed.

After underpassing, no movement of the up line was discernible in the data: therefore the volume loss must have been approximately 0·0%. The down line, on the other hand, did experience some settlement. The tilt sensors at each end of the string showed some rotation, so it was not known what the settlements at the ends of the string were. Precise levelling of the ends of the string showed that no significant movements occurred, but the repeatability, as discussed previously, was probably only ±0·5 mm. A regression analysis varying both trough width and string end settlement showed that the best fit to the data was obtained with a string end settlement of 0·05 mm and a trough width of 7·0 m. The adjusted tilt sensor data are compared with the Gaussian curve fit in Figure 13. The tilt sensor data in Figure 13 have been resolved to take account of the 75° angle between the TWRM tunnel and the HS1 tunnel. The volume loss of the Gaussian curve, V_l, was 0·29%, the maximum settlement S_{max} was 1·4 mm, and the trough width parameter K was 0·407.

4.2. Monitoring of the Northern line running tunnels

The Northern line running tunnels were underpassed to the west of the Angel station platforms, under Pentonville Road near the junction with Islington High Street. The position of the Northern line tunnels relative to the TWRM alignment is shown in plan in Figure 14 and in section in Figure 15. When the station was upgraded in the early 1990s the island platform was replaced by two platform tunnels with a concourse tunnel between them. The original tunnel containing the island platform became the southbound platform tunnel (which is why it is rather large), and the northbound running tunnel had to be rerouted via a step-plate junction to the west of the crossing location to join into the new platform tunnel. So the

	Warning value: mm	Action value: mm	Speed restriction value: mm
Vertical track movement	±2	±4	±7
Twist over a 3 m length	±2	±3	±6

Table 5. Trigger values for HS1 real-time monitoring

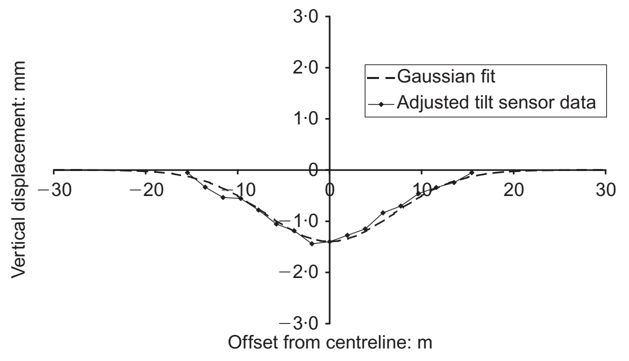

Figure 13. HS1 down line settlement monitoring data

TWRM tunnel actually passed under three tunnels, which will be referred to as the southbound, disused and northbound tunnels. The southbound and disused tunnels have older 12 ft (3·66 m) i.d. cast iron rings with $4\frac{3}{4}$ in (117 mm) flange depth, and the northbound tunnel has newer 3·85 m i.d. ductile cast iron rings with a 110 mm flange depth.

The TBM was excavating through a mixed face of Thanet Sand and Upnor formation, and the Northern line tunnels are in the Lambeth Group, with the London Clay above. Therefore, in terms of surface settlements and the movements of the Northern Line tunnels, the ground could be characterised as predominantly very stiff clay.

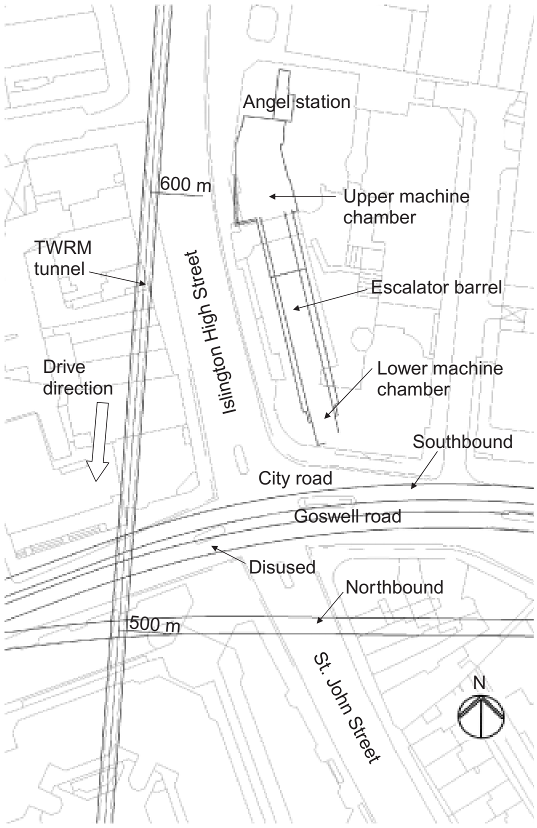

Figure 14. Plan of Northern line tunnels crossing

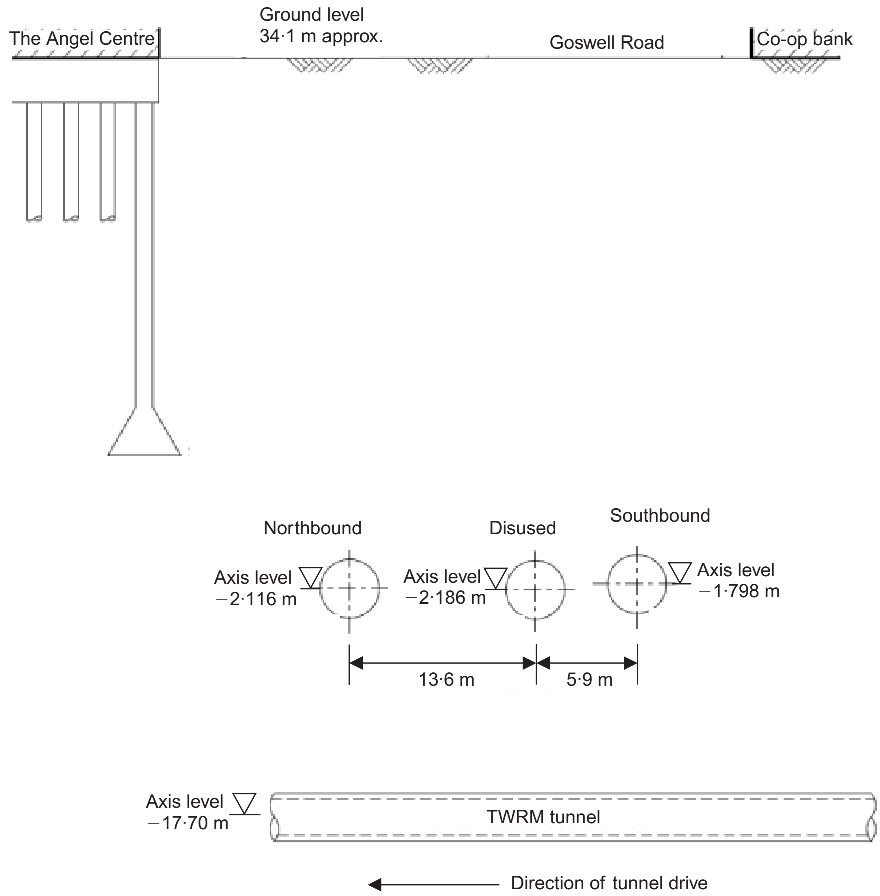

Figure 15. Section of Northern line tunnels crossing

Predicted movements based on volume losses of 0·5%, 1·0% and 1·5%, and a trough width parameter of 0·45, are shown in Table 6.

Predictions were made of structural distortions and track alignment effects, and no problems were foreseen. Daily levelling during engineering hours using a laser total station and prismatic targets was undertaken while the TBM was within the zone of influence, and then at 1 week, 2 weeks, 1 month and then at monthly intervals thereafter until movements could be proven to have stabilised.

The results of the monitoring are shown in Figures 16, 17 and 18 for the southbound, disused and northbound tunnels respectively. Maximum settlements of the six arrays in the three tunnels were very consistent at around 2 mm. Since there was little difference between levels either side of the rails in a given tunnel, and there were no increases in settlement after the TBM was more than 40 m past the tunnels, the two sets of data were averaged for each tunnel over the last four readings. A Gaussian curve was then fitted to the averaged data from each tunnel using the statistical method outlined earlier in the paper, and these are shown in Figure 19. The Gaussian curve parameters for the three Northern line tunnels are listed in Table 7.

5. DISCUSSION

The EPB TBM used on this project was able to control volume losses to below 0·5%. The ground conditions often facilitated the application of an EPB pressure well below full overburden pressure to drive the TBM forward while minimising face losses. Under the HS1 up line EPB pressures were maintained at approximately 100 kPa, approximately equal to only 10% of full overburden pressure, and this appears to have been sufficient to mitigate any settlement compared with the down line. Array 2150, shown in Figure 10, was located in Highbury Station Road, 16 m past the HS1 up line crossing. The volume loss of array 2150 was calculated to be 0·33%. This is more comparable to the volume loss on the down line. This may have been due to the fact that the dayshift TBM driver who underpassed the down line and array 2150 did not generate as high an EPB pressure as the nightshift TBM driver who underpassed the up line. The fact that it was possible to control settlements tightly with low EPB pressures, and that these could be prevented altogether by maintaining relatively low levels of face pressure compared with the in situ stress, is worthy of further investigation, particularly since the ground was in effect unsupported around the tailskin annulus until the rings were grouted approximately 6 m back from the face. This may be subject to TBM driver skill and the configuration of the cutterhead tools.

The most interesting aspect of the case study data presented in this paper is that the trough width parameter values were smaller than expected, both at the surface and in the deformations of third-party tunnels. Very little information on trough width parameter values for tunnels at depths greater than about 35 m was available to inform estimates, and so the prediction methods proposed by O'Reilly and New (1982) and Mair *et al.* (1993) were used. These methods have not been verified for a tunnel at this depth, and so sensitivity analyses of trough width parameter were performed to satisfy third parties that the operation of their assets would not be compromised at limiting values. Despite the lack of empirical data, obtaining approvals from third parties was possible largely because the volume losses measured at the surface were consistently quite small. In future situations at similar tunnel depths where larger tunnels are constructed, higher volume losses are expected, or tunnels are underpassing closer to third party tunnels, the accurate estimation of trough width parameter will become absolutely critical to a project's feasibility, and this is where a case study such as this one will be valuable.

All the trough width parameter values calculated on this project are plotted in Figure 20, which is a graph of trough width parameter K and z/z_0, a dimensionless ratio of the depth of the point in question to the depth of the tunnel under construction used by Mair *et al.* (1993). Also shown is a constant value of trough width with depth of 0·5 (O'Reilly and New, 1982), and the relationship proposed by Mair *et al.* (1993) in Equation 5 above. The surface settlement trough width parameter values varied between 0·15 and 0·35, whereas case history data in London's stiff clays suggest they should be between 0·4 and 0·6. The warning given by Lake *et al.* (1996) that trough width parameter is probably smaller than might be predicted when the tunnel depth is greater than 20 m appears to be true. As can be seen in Figure 20, the subsurface trough width parameter values were also perhaps lower than expected. This is probably due to the normalisation of the vertical axis; when the depth to the tunnel under construction, z_0, is large, as it is in this case, the effect is to make the Northern line and

Tunnel		Settlement: mm		
		Volume loss: %		
		0·5	1·0	1·5
Southbound	Crown	2·1	4·3	6·4
	Invert	2·8	5·5	8·3
Disused	Crown	2·2	4·4	6·6
	Invert	2·8	5·7	8·5
Northbound	Crown	2·2	4·4	6·6
	Invert	2·8	5·7	8·5

Table 6. Predicted maximum settlements of the Northern line running tunnels

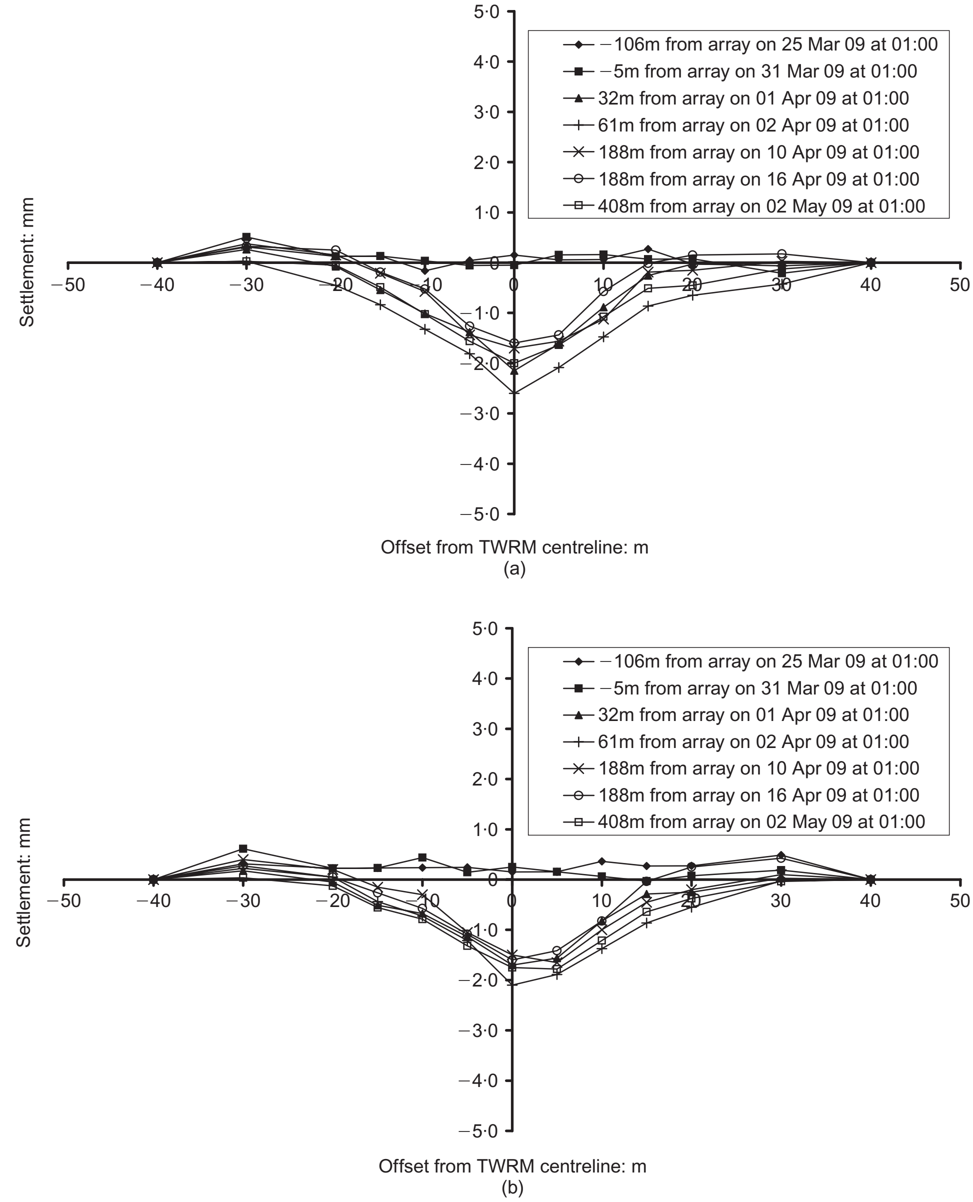

Figure 16. Northern line southbound tunnel settlement monitoring data: (a) left rail; (b) right rail

HS1 tunnels appear to be closer to the TWRM tunnel than they are, when in fact they are five or six tunnel diameters above the TWRM tunnel.

In Figure 21 the data from this project have been added to the meta-analysis of subsurface settlement data presented in Figure 20 of Mair and Taylor (1997), along with subsurface settlements reported more recently in the literature. The new data consist of measurements made in the Bakerloo and Northern Line tunnels at Waterloo during construction of the Jubilee Line Extension reported by Standing and Selman (2001), measurements made in the Piccadilly line during construction of the Heathrow CTA station by Cooper *et al.* (2002), data from extensometers installed above the tunnel centreline at Heathrow Terminal 4 station (the reader will need to refer to both Clayton *et al.*, 2006, and van der Berg *et al.*, 2003), and settlements measured in the Central line during underpassing by the City of London cable tunnel by Legge and Bloodworth (2003). In the key, the depths of the tunnels are shown in square brackets alongside the references. There appears to be a general trend for trough width parameter values for deeper tunnels to be to the left of the curve – that is, they are overpredicted – and for shallower tunnels, for example 'centrifuge model 2DP', which is at an equivalent depth of 9·8 m, to be to the right of the curve. The curve fits the 'centrifuge model 2DV' data best, which is at an equivalent depth of 16·5 m. The general picture is not cut and dried, since Equation 5 also seems to model well the data from Barratt and Tyler (1976) at 34 m depth and Standing and Selman (2001) at 30·5–31·2 m depth, whereas the points from Nyren (1998) at 31 m and Attewell and Farmer (1974) at 29 m depth are clearly overpredicted.

To illustrate this apparent dependence of K on the depth of the tunnel, Figure 22 shows the difference between measured values of K from Figure 21 and predicted values of K using Equation 5, plotted against depth. Also shown in Figure 22 are values of K calculated from surface settlements listed in Table

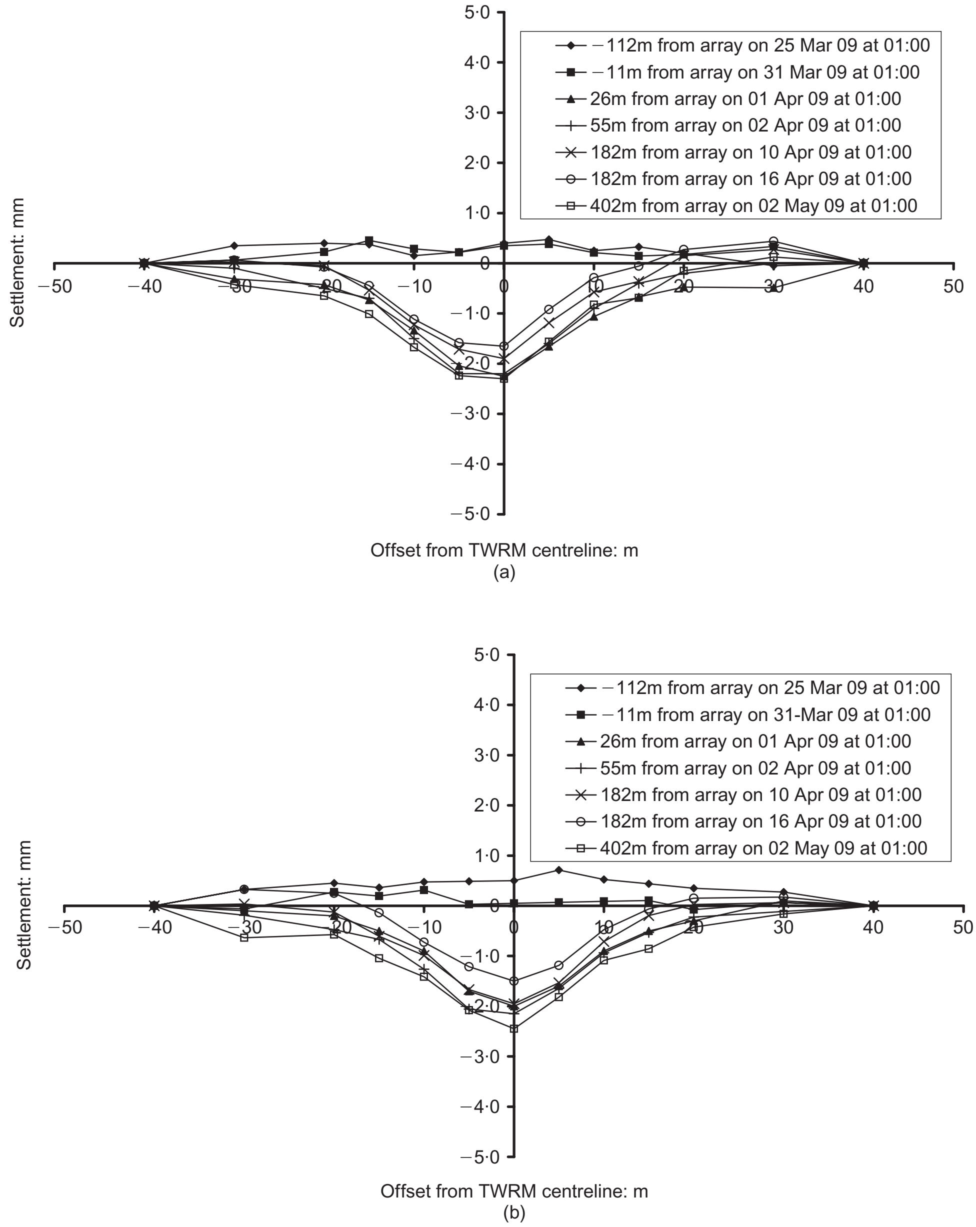

Figure 17. Northern line disused tunnel settlement monitoring data: (a) left rail; (b) right rail

1 in Mair and Taylor (1997), as well as subsurface and surface settlements from more recent projects. The new surface settlement data come from Heathrow Terminal 4 station (Clayton *et al.*, 2006) and Heathrow Terminal 5 storm water outfall tunnel frontshunt tunnel (Jones *et al.*, 2008). Those undertaking settlement predictions should take note of the variability of K as demonstrated by Figure 22, and the accuracy of its prediction, and perform appropriate sensitivity analyses. A linear regression has been performed on all the data, ignoring two outlying points of Attewell and Farmer (1974) and Nyren (1998) that were very close to the tunnel and hence should be expected to deviate from Equation 5. The linear regression line shows that at between approximately 5 and 35 m depth Equation 5 will predict K reasonably well, considering the inherent variability of the data. At depths greater than 35 m, however, K may be significantly overpredicted, although at present there are only data from this project to support this. It would be of benefit if more case histories of tunnels at depths greater than 35 m were made available to confirm these findings.

Figure 23 shows all the values of K from case histories plotted against height above the tunnel ($z_0 - z$). Equation 5 has been applied (broken lines labelled 'Mair *et al.* (1993)') for tunnel depths z_0 of 20, 40 and 60 m. Using only a tunnel depth z_0 of 20 m in Equation 5 fits the data quite well up to heights of around 35 m above the tunnel. However, when a tunnel depth z_0 of 40 or 60 m is used, the prediction of K appears to be less reliable at any height above the tunnel. It seems that K may be dependent only on height above the tunnel, and not on any kind of relative depth. This implies that the location of the ground surface may not be relevant to subsurface settlements, and the value of K at the ground surface is dependent on the depth of the tunnel, as Lake *et al.* (1996) intimated, and will not always be equal to 0·5 (as would be predicted by Equation 5). Another way of thinking of it is that K does not

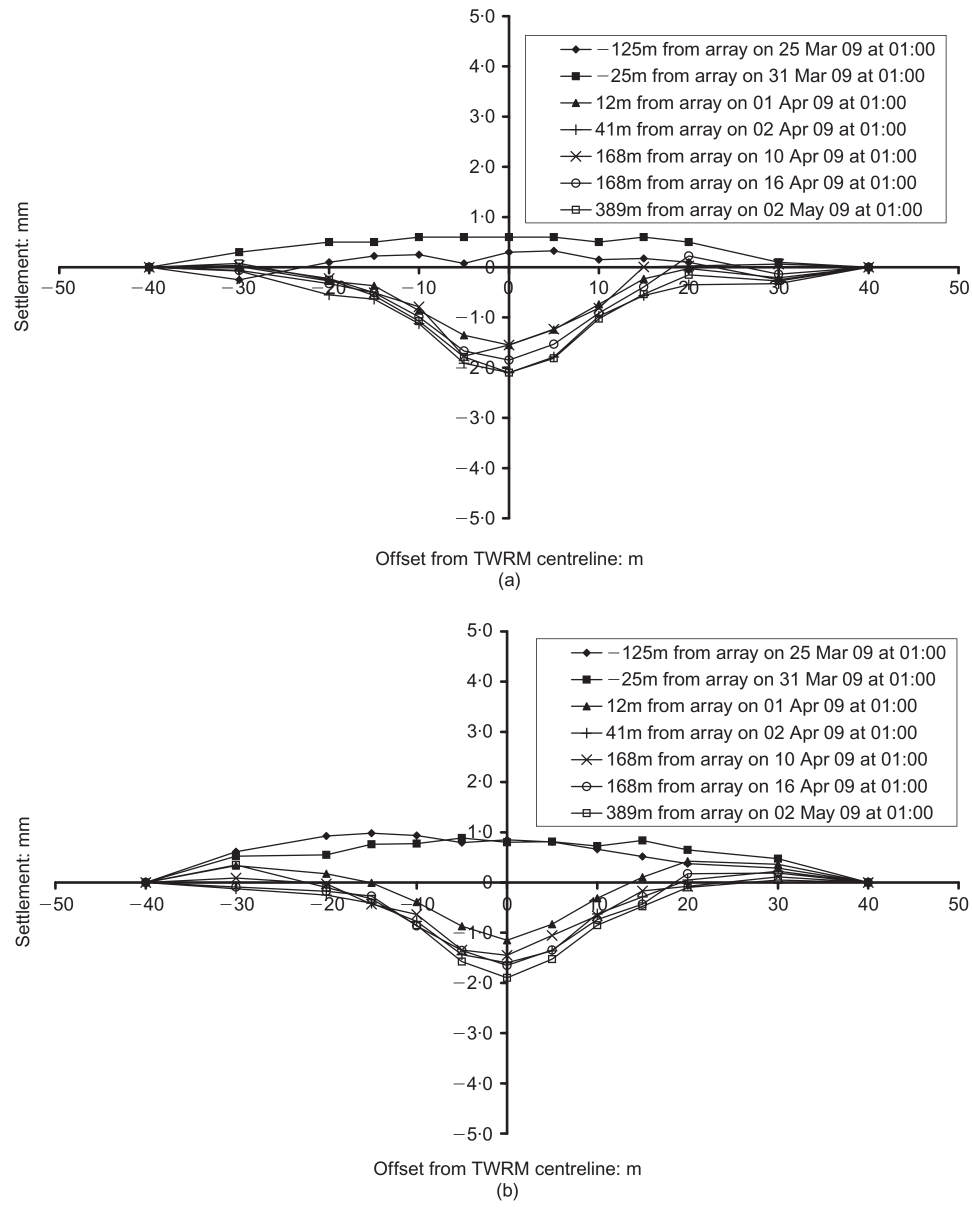

Figure 18. Northern line northbound tunnel settlement monitoring data: (a) left rail; (b) right rail

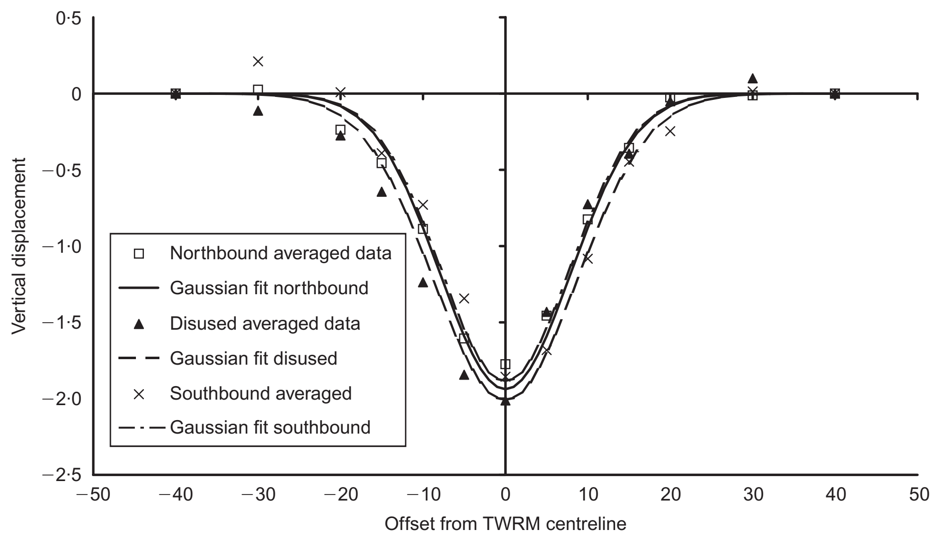

Figure 19. Gaussian curve-fit to Northern line tunnels data

Tunnel	Volume loss, V_l: %	Trough width parameter K	Maximum settlement S_{max}: mm
Southbound	0·43	0·495	1·9
Disused	0·51	0·564	2·0
Northbound	0·45	0·510	1·9

Table 7. Gaussian curve parameters for Northern Line tunnels

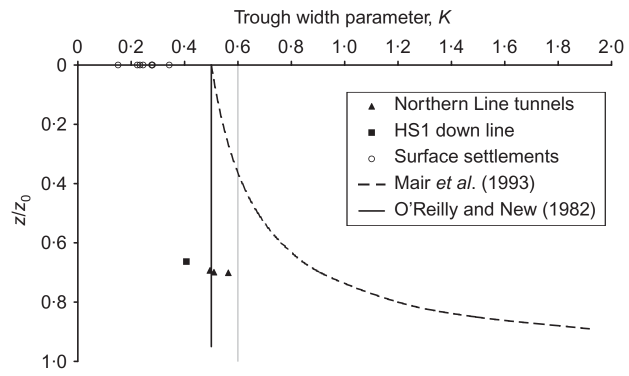

Figure 20. Variation of trough width parameter K with relative depth z/z_0 for surface and subsurface settlement profiles on TWRM extension project

increase with depth from a constant value at the surface, as Equation 5 implies: it decreases with height above the tunnel.

A curve-fitting exercise was performed on all the data shown in Figure 23. The data did not fit a hyperbolic curve similar to Equation 5, since the inverse of K was not linearly proportional to the inverse of $(z_0 - z)$. However, there did seem to be a linear relationship between K and the logarithm of height above the tunnel $(z_0 - z)$. This is shown in Figure 24. The curve is defined by the equation

$$K = -0{\cdot}25\ln(z_0 - z) + 1{\cdot}234 \tag{6}$$

Figure 24 shows a high degree of scatter as the tunnel is approached. This may be due to the variety of tunnel sizes represented by the data, since the height above the tunnel $(z_0 - z)$ is measured from the axis level. For subsurface data obtained from the monitoring of existing tunnels during underpassing, there may be variability caused by assuming the movement is determined by the axis-level (z) ground movements rather than the invert-level settlement. Therefore it would be sensible to exercise caution when using this relationship for the prediction of subsurface settlements at values of $(z_0 - z)$ smaller than 10 m. This limit may need to be higher for large-diameter tunnels.

Finally, Figure 25 shows Equation 6 with the case history data from Figure 23. Again, for values of $(z_0 - z)$ greater than 10 m

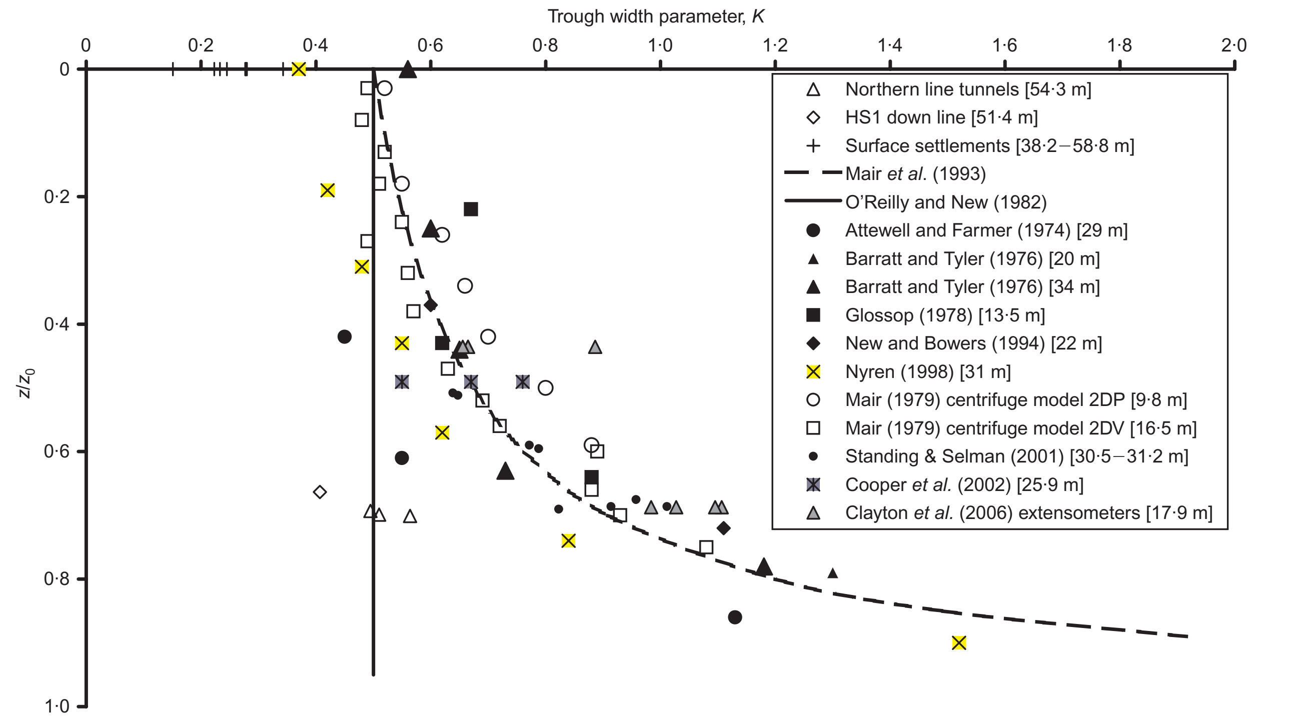

Figure 21. Variation of trough width parameter K with relative depth z/z_0 for surface and subsurface settlement profiles of tunnels in clays (based on Figure 20 in Mair and Taylor, 1997)

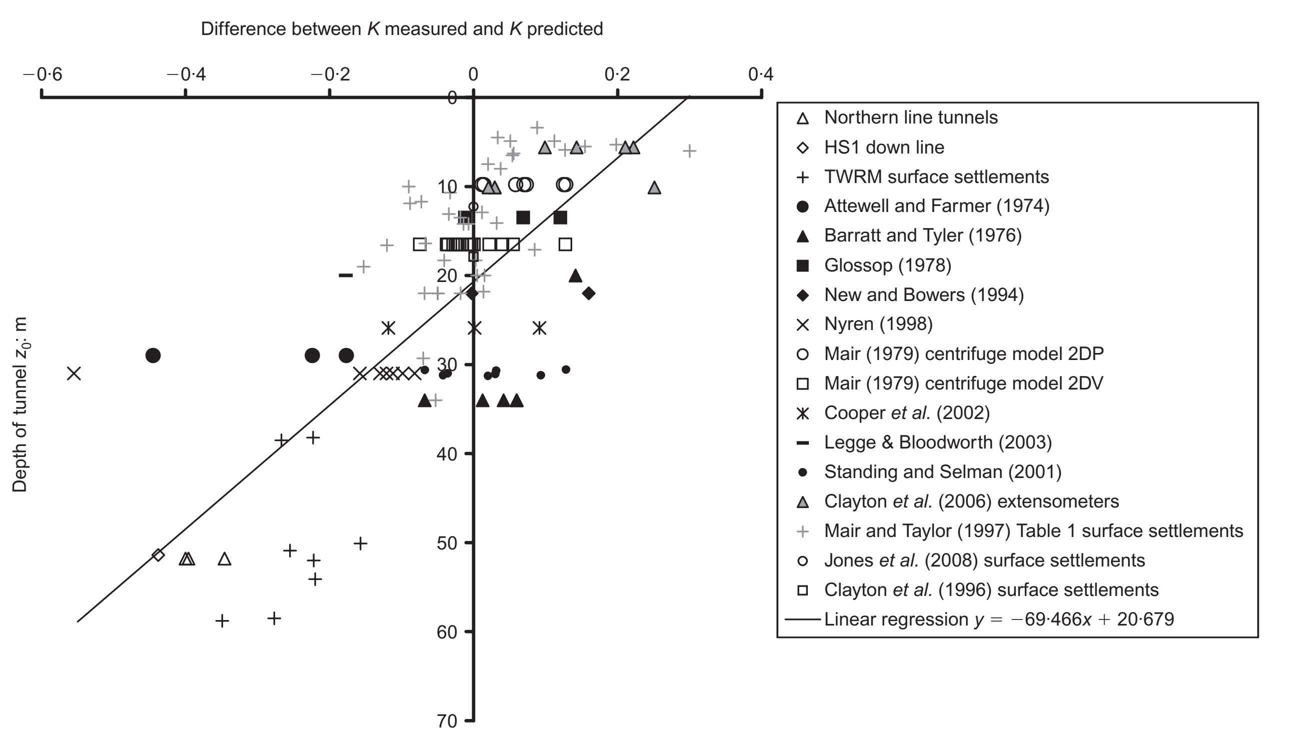

Figure 22. Variation of difference between trough width parameter K measured and K predicted using Equation 5 with tunnel depth

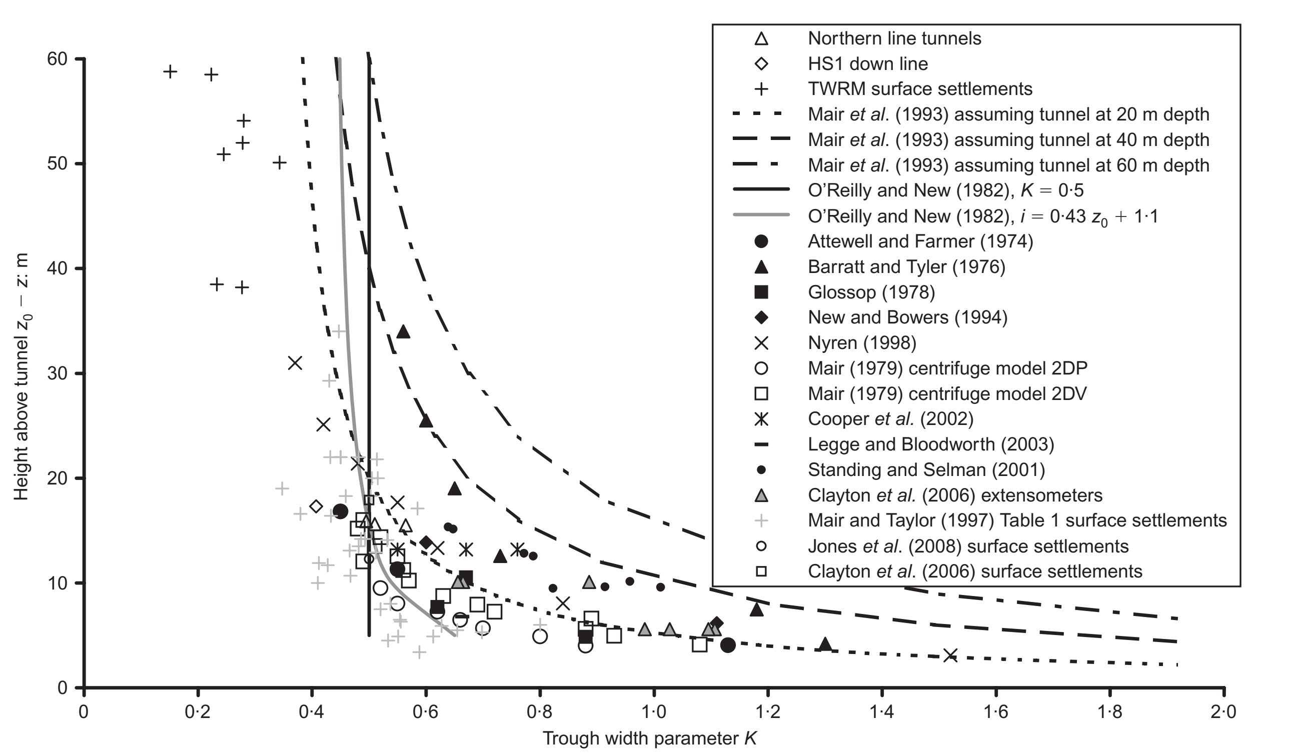

Figure 23. Variation of trough width parameter K with height above the tunnel $z_0 - z$ for surface and subsurface settlement profiles of tunnels in clays

and up to 60 m agreement is good, compared with the relationships shown in Figure 23.

6. CONCLUSION

This paper has shown how small values of surface settlements can be rigorously interpreted to obtain reliable values of volume loss and trough width parameter to aid in the prediction of, in this case, far more critical subsurface settlements.

The monitoring systems employed on the Thames Water ring main extension Stoke Newington to New River Head project were adequate mitigation for the risks, and the paper is a valuable addition to the available literature on surface and

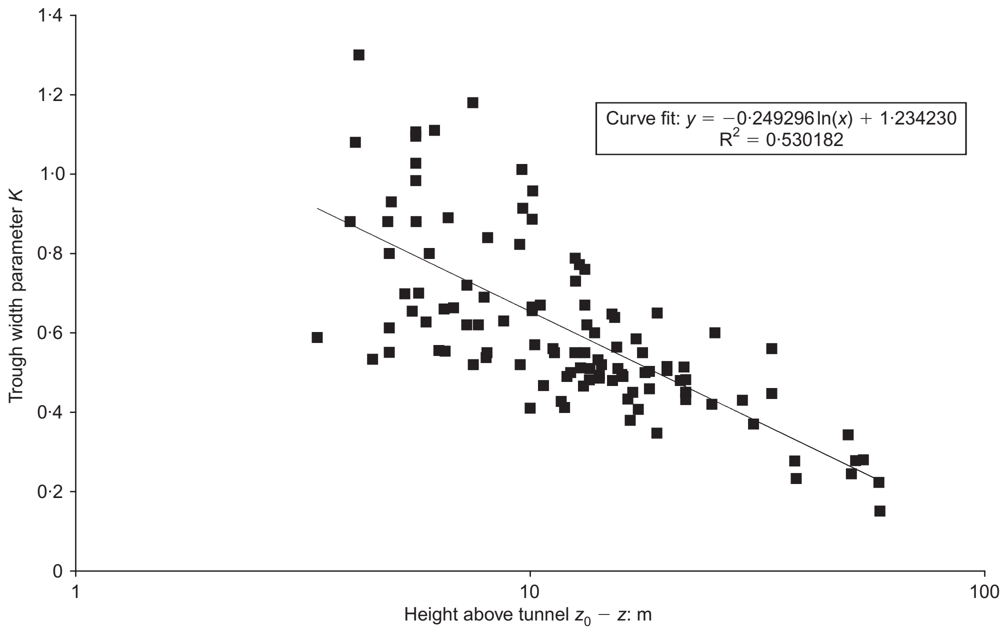

Figure 24. Logarithmic curve-fitting of trough width parameter K and height above the tunnel $z_0 - z$ for surface and subsurface settlement profiles of tunnels in clays

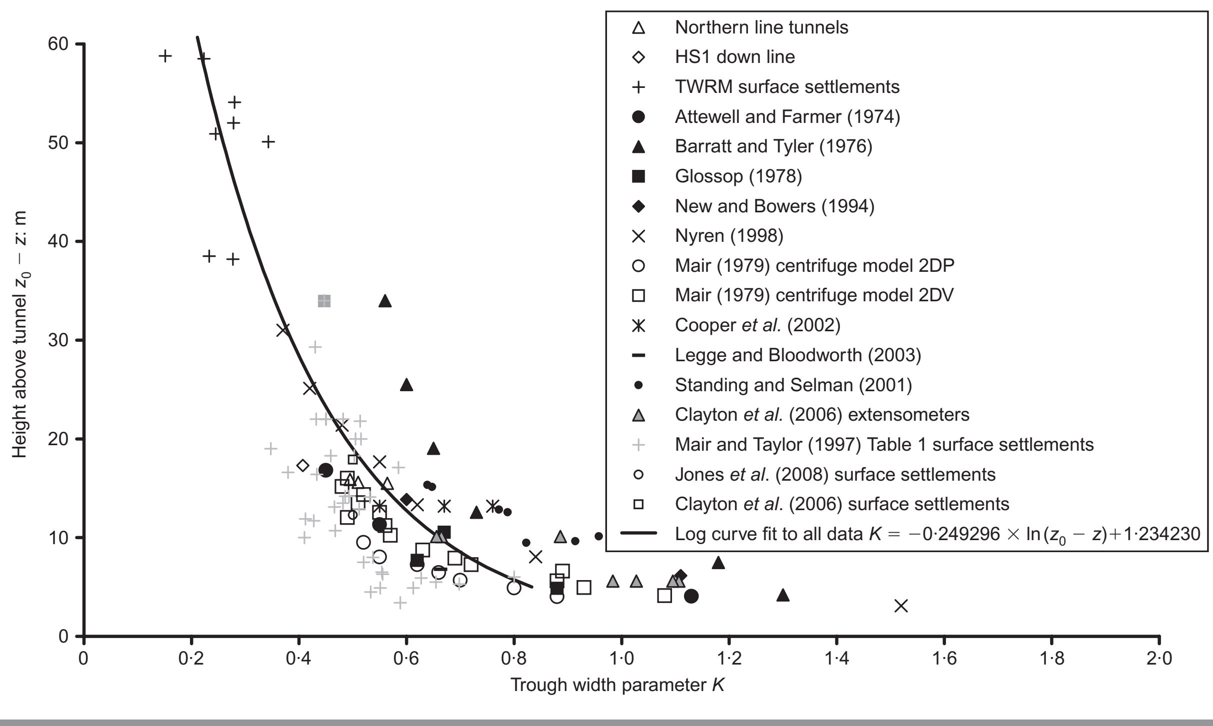

Figure 25. Logarithmic variation of trough width parameter K with height above the tunnel $z_0 - z$ for surface and subsurface settlement profiles of tunnels in clays according to Equation 6

subsurface ground movements in clays, particularly since there are so few data for tunnels at depths greater than about 35 m. The project was a success in terms of control of volume loss, and it is hoped that a case study such as this may help to alleviate the concerns of third-party stakeholders when faced with similar situations in future.

Good predictions of surface and subsurface ground movements will be critical to the feasibility of future projects, which will tend to be at greater depth because of the need to avoid existing tunnels. In particular, accurate prediction of trough width is crucial to the assessment of risk, as it determines not only the maximum settlement for a fixed value of volume loss but also the rate of change of settlement transverse to the tunnel under construction. A narrower settlement trough will have a higher maximum settlement for a given volume loss, as the area under the curve remains the same. It will also have higher gradients and curvatures, which will increase the risk of damage to buildings and rail track distortions. On the TWRM project, believed to be the first tunnel in clay below 35 m depth for which trough widths have been calculated, the trough width parameter was found to be consistently lower than predicted, and appeared to be anomalous compared with the work of Mair *et al.* (1993) when plotted against relative depth z/z_0. At the same time, it did not seem to make sense that the trough width parameter should always be equal to 0·5 at the surface and should increase with relative depth, regardless of the actual depths involved.

Based on the data from the Thames Water Ring Main Extension and a meta-analysis of previous case studies the trough width parameter K has been shown in this paper to be dependent on height above the tunnel $(z_0 - z)$ rather than relative depth z/z_0. A logarithmic equation has also been shown to provide reasonable predictions of K at heights above the tunnel of greater than 10 m and up to at least 60 m. Using this new relationship, the new TWRM data no longer appear to be anomalous. It is therefore recommended that assuming a constant value of K, as O'Reilly and New (1982) suggested, or assuming that K is always equal to 0·5 at the surface and that it varies with relative depth, as proposed by Mair *et al.* (1993), should be done with caution, particularly for tunnels at depths greater than 35 m below the ground surface.

The variability of K measured in the field over a large number of projects appears to be fairly high, and sensitivity analyses should always be performed where K is a critical parameter.

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