# Effect of TBM driving parameters on ground surface movements: Channel Tunnel Rail Link Contract 220

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ABSTRACT: The aim of this paper is to analyse the causes of ground movements induced by earth pressure balance (EPB) machine driven tunnels. An attempt to link the observed ground movements to the key driving parameters of the EPB machine, such as face pressure as well as shield and tail void grouting pressure, is presented. It was found that volume losses were successfully controlled below the 1% target with face pressures comprised between 20 and 50% of the overburden pressure.

## 1 INTRODUCTION

Channel Tunnel Rail Link's Contract 220 comprised two 7.15 m i.d. tunnels, driven from Stratford to St Pancras station in London. The 7.5 km long twin tunnels were constructed by a Nishimatsu-Cementation-Skanska JV using two 8.11 m diameter dual mode earth pressure balance (EPB) machines, which have been operated in closed mode nearly throughout the length of the drives.

The temporary tunnel lining consisted of 1.5 m long pre-cast reinforced concrete segments with a thickness of 0.35 m. Ten segments formed a ring of 7.85 m o.d., therefore leaving a 0.18 m thick annulus between the excavation boundary and the lining. In order to minimise the ground movements due to the tunnelling operations, the pressure imparted to the tunnel face was maintained above carefully predefined levels, bentonite was injected in the over-excavated annulus around the shield and the tail void was grouted simultaneously to the shield advance.

Ground and building movements were extensively monitored along the tunnelling route. In particular, 48 sections comprising arrays of 5 to 10 surface monitoring targets allowed the vertical settlements to be measured and the volume loss to be inferred at all time during construction to ensure it would remain within the contractual limit of 1%. In addition, all driving parameters of the EPB machines were monitored and logged at 0.2 Hz for the whole period of construction. These comprised in particular the position of the tunnelling machine with its pitching, and yawing angle, the earth pressure at five locations in the plenum chamber of the EPB machine, the pressure and quantity of bentonite and grout injected around the shield and lining, respectively. Such an extensive real time monitoring exercise allowed prompt reaction at the tunnel level when unusual trends of movements were measured at the surface. As a result, the tunnelling work was successfully conducted, causing volume losses generally falling between 0.2 to 0.8% (hence below the contractual limit of 1%).

The aim of this paper is to report and analyse the characteristics of the ground movement caused by the excavation of the first of the two tunnels to be excavated (the *upline*) and to establish links between the key parameters of the tunnelling process and the measured volume losses. Rather than analyses accounting for the details of each particular monitoring site, the focus has been to observe general trends, with the objective of assessing the effectiveness of face pressure, shield grouting and tail void grouting in reducing volume losses.

## 2 GROUND CONDITIONS

The ground conditions encountered along the route of contract 220 were diverse. The EPB machines have had to progress through a wide range of soil types,

Table 1. Geological conditions along the tunnel route.

| Material | $\gamma$ [kN/m$^3$] | $K_0$ |
|---|---|---|
| Made ground (MG): Firm with little fine to medium flint, brick or claystone gravel. | 18–20 | 0.5 |
| Terrace Gravel (TG): Dense sand with much gravel. | 19–20 | 0.7 |
| London Clay (LC): Very stiff, very to extremely fissured silty clay with occasional rare silt and sand partings. | 19 | 1.4 |
| Harwich Formation (HF): Can consist of sand, clay or silt. | 19 | 1.2 |
| Woolwich and Reading Beds (WRB): Complex formation varying from stiff and hard clay to dense sand and gravel. | 20 | 1.2 |
| Upnor Formation (UP): Grey or green grey silty sand. | 19 | 1.0 |
| Thanet Sand (TS): Fine and medium silty sand. | 19 | 1.0 |
| Upper Chalk (UC): Fractures closely spaced often clean and tight. | 20 | n/a |

which is often thought to be a source of problems. The initial 2 km of the drive mainly encountered the Upnor Formation, often in mixed face conditions with Thanet sands, before proceeding to full face Thanet sand for approximately 2.5 km. The tunnel then successively excavated Upnor sands, Woolwich and Reading Beds clays and finally over 1 km in full face London Clay. Around chainage 2800, thin layers of Harwich formation were often encountered, causing abrupt and recurrent changes of face conditions with London Clay. The basic characteristics of these soil strata are summarised in Table 1.

Two aquifers are found in the London Basin. The upper aquifer lies in Made Ground and Terrace Gravel and the lower aquifer lies in the Upnor Formation and Thanet Sand. On the first half of the tunnelling route, the Thanet Sand and the Upnor Formation had been dewatered prior to the tunnel construction, so that the water table in the lower aquifer was just below the tunnel invert level.

## 3 GROUND MOVEMENTS

It is well known that the transverse settlement trough caused by the construction of a tunnel may be approximated by a Gaussian distribution curve with the following equation (Peck, 1969):

$$S_T = S_{\max} \exp\left(\frac{-y^2}{2i_y^2}\right);\; S_{\max} = \frac{V_L \pi D^2}{i_y\sqrt{2\pi}} \tag{1}$$

where $S_T$ is the vertical settlement of a point lying a horizontal distance $y$ from the tunnel axis, $S_{max}$ is the maximum settlement above the tunnel axis, $D$ the excavated tunnel diameter, $V_L$ the volume loss, and $i_y$ the horizontal distance from the tunnel centre-line to the point of inflexion of the settlement trough. O'Reilly and New (1982) further showed that the trough width $i_y$ can be estimated as follows:

$$i_y = K_y z_0 \tag{2}$$

where $K_y$ is the transverse trough width parameter and $z_0$ the depth of the tunnel axis. Attewell & Woodman (1982) showed that the longitudinal settlement profile can be described by:

$$S_L(x) = S_{\max}\Phi(x)\;;\; \Phi(x) = \frac{1}{i_x\sqrt{2\pi}}\int_{-\infty}^{x} e^{-\frac{x^2}{2i_x^2}} \tag{3}$$

where $x$ is the distance ahead of the tunnel face parallel to the tunnel axis, $i_x$ is the trough length parameter and $\Phi(x)$ is the cumulative probability function. For tunnels excavated without face support, Attewell & Woodman (1982) found that the surface settlement directly above the tunnel face generally corresponds to about $0.5\,S_{max}$. However, for tunnels constructed in soft clay with face support, the surface settlement directly above the tunnel face tends to be considerably less than $0.5\,S_{max}$; this was also shown to be the case by Mair & Taylor (1997).

## 4 MACHINE PARAMETERS

The ground movements associated with shield tunnelling are commonly attributed to (1) the stress relief at the tunnel face, (2) the over-cutting edge around the shield, (3) the closure of the tail void behind the shield, (4) the deflection of the lining and (5) the consolidation of the ground around the tunnel (Cording, 1991; Mair & Taylor, 1997; Dimmock, 2003). To examine how the tunnelling process influences the different components of volume loss, dimensionless groups of machine parameters have been defined. As depicted in Figure 1, zones of influence were chosen on the simplifying assumption that the face pressure $p_f$ only influences the measured ground movements as the face approaches the monitoring section, while subsequently occurring volume losses are marginally influenced by the face pressure and rather more by the grouting processes around the shield and behind the tail void. The shield and tail grouting are quantified by the grouting pressures ($p_b$ and $p_g$, respectively), and the volume of injected bentonite and grout ($V_b$ and $V_g$, respectively.

Dimensionless groups of TBM parameters can be expressed as shown in Table 2, where $D_e$, $D_s$ and $D_{tv}$ are the diameters of the excavation, the shield and the

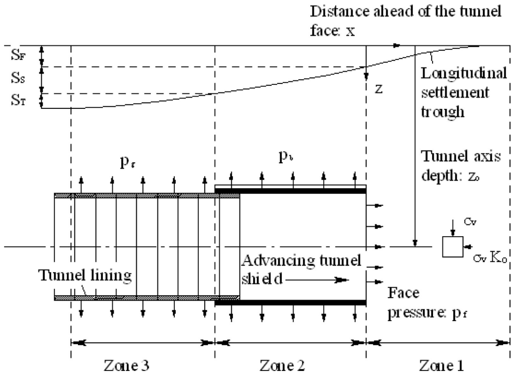


Figure 1. Definition of parameters.

Table 2. Dimensionless groups of TBM parameters.

| Machine parameter | Dimensionless group | Zone of influence |
|---|---|---|
| Face pressure: $p_f$ | Face Pressure Ratio $FPR = p_f/\gamma \cdot z_o$ | Zone 1 |
| Bentonite injection pressure: $p_b$ | Bentonite Pressure Ratio $BPR = p_b/\gamma \cdot z_o$ | Zone 2 |
| Bentonite injection volume: $V_b$ | Overcut Filling Ratio $OFR = 4 \cdot V_b/\pi \cdot (D_e^2 - D_s^2)$ | Zone 2 |
| Grouting pressure: $p_g$ | Grouting pressure ratio $GPR = p_b/\gamma \cdot z_o$ | Zone 3 |
| Grout injection volume: $V_g$ | Tail void Filling Ratio $TFR = 4 \cdot V_g/\pi \cdot (D_{tv}^2 - D^2)$ | Zone 3 |

lining, respectively, and where $V_b$, $V_g$, $p_f$, $p_b$ and $p_g$ have been averaged over the zones of influence defined in Figure 1. Note also that the face pressure $p_f$ has been assumed to equate to the average earth pressure measured at five different locations on the bulkhead at the rear of the plenum chamber.

## 5 GROUND MOVEMENT CAUSED BY THE EPB MACHINE

The volume losses measured at each of the 45 monitoring sections are shown in Figure 2.

### 5.1 *Transverse settlement*

For each monitoring section, $V_L$ and $K_y$ were estimated by best fitting the Gaussian curve to the measured settlement of the levelling points transverse to the tunnel axis. It was found that the majority of the $V_L$ is independent of tunnel depth and lies between 0.2 to 0.8%, which is lower than previous experience of tunnelling in London (using open face techniques) where reported volume loss was generally between 1.0 and 3.5% (Mair, 2003).

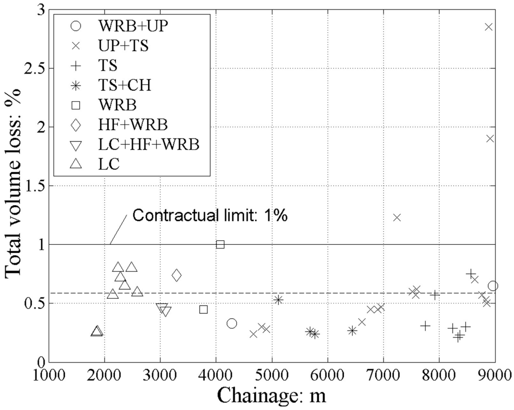


Figure 2. Measured volume loss along the tunnel route.

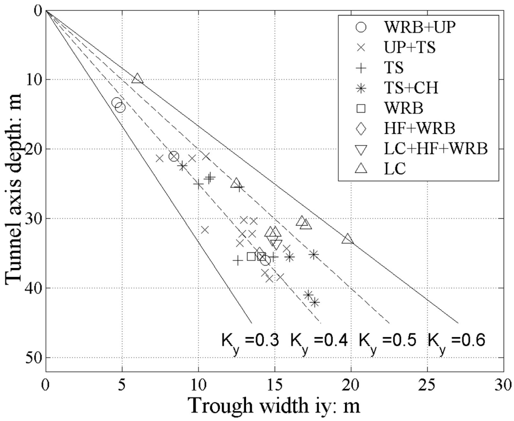


Figure 3. Transverse trough width versus tunnel depth.

Figure 3, which shows the variation of trough width with tunnel depth, suggests that $K_y$ ranges from 0.3 and 0.6. The observed trough width parameter, $K_y$, for the excavation in WRB mixed face materials in this study was 0.35–0.45, which was similar to the value reported by Dimmock (2003), for excavation also with an EPB machine. Furthermore, $K_y$ of 0.46–0.60 for the excavation in London Clay shows consistency with other data presented by Mair and Taylor (1997), despite different the excavation methods. This confirms that value of $K_y$ is independent of tunnel construction method but depends on the material in which the tunnel is excavated.

### 5.2 *Longitudinal settlement*

Longitudinal surface settlements from various monitoring points were best fitted with a cumulative probability function. Equation 3 was found to describe the data sufficiently well, despite the excavation being carried out using an EPB machine. Furthermore, it was found that, if the settlement process was divided into three phases, as shown in Figure 1, then 40 to 50% of the settlement occurred over the tail void. The proportion of the settlement directly above the tunnel face $S_F$ was in the range 0.1–0.4 times $S_{max}$. This compares with published data as shown in Table 3.

The value of trough length, $i_x$, from each monitoring section was estimated by fitting a cumulative probability function to all available longitudinal troughs, and plotting this against $i_y$ as shown in Figure 4, which shows that $i_x$ is approximately 0.6–1.7$i_y$. For open face excavation in London Clay, Nyren (1998) reported values of the trough length parameter ranging from 0.7 to 0.8 times $i_y$. The data in Figure 4 yields an average value of $i_x = i_y$ for excavation in London Clay with EPB machine. For other materials the data show that the majority of values of $i_x$ seems to lie between 1.0 to 1.7 times $i_y$. Larger trough length parameters observed in this study may be due to the larger length of the TBM (the distance between the face and the tail void is greater). Table 4 summarises volume loss, trough width and longitudinal trough parameters derived from the measured settlements when tunnelling in each material.

Table 3. Published data for the proportion of the settlement directly above the EPB face to the maximum settlement.

| Reference | $S_F/S_{max}$ |
|---|---|
| Dimmock (2003) | 0.25 |
| Sugiyama *et al.* (1999) (slurry shield) | 0.14 |

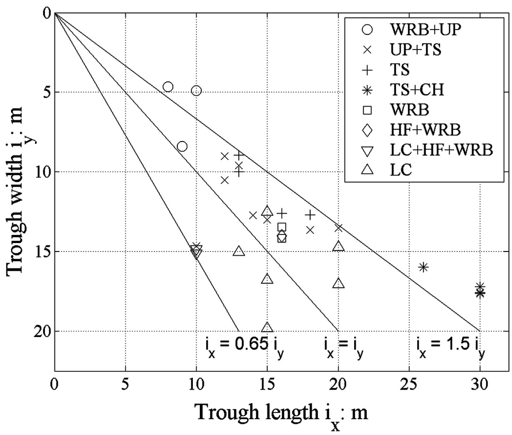


Figure 4. Relationship between trough length and width.

## 6 INFLUENCE OF TBM DRIVING PARAMETERS ON VOLUME LOSS

By analogy to the settlement $S_F$, $S_S$, and $S_T$ depicted in Figure 1, three components of volume loss have been defined. $V_{LF}$, $V_{LS}$ and $V_{LT}$ are therefore the volume losses measured as the tunnel face is within the monitoring section, when the rear of the shield passes the monitoring section, and when the shield tail has advanced 10.5 m (7 rings) beyond the monitoring section, respectively.

### 6.1 *Influence of face pressure*

Figure 5 shows the volume loss $V_{LF}$ measured as the tunnel face passes through the monitoring section versus the normalised pressure $FPR$ imparted to the tunnel face. While $V_{LF}$ may be expected to reduce as the face pressure ratio is increased, no such clear correlation can be seen. One may however conclude from Figure 5 that for tunnelling in Thanet Sand, face pressures comprising between 20–30% of the overburden pressure were sufficient to maintain the volume loss component $V_{LF}$ below 0.2%, which for $K_y = 0.5$ and $z_o = 25$ m corresponds to a maximum surface settlement of just over 3mm. When the average face pressure ratio dropped below 0.2, larger volume losses were measured on a few occasions. Interestingly, when unusually large face volume loss occurred, proportionally larger volume

Table 4. Summary of volume loss and trough width parameter for different material excavated.

| Material excavated | Volume loss: % Average | Volume loss: % Range | Trough width parameter, $K_y$ Average | Trough width parameter, $K_y$ Range | $I_x/i_y$ | $S_F/S_{max}$: % |
|---|---|---|---|---|---|---|
| WRB + UP | 0.61 | 0.33–1.0 | 0.38 | 0.35–0.4 | 1.0–2.0 | 14–30 |
| UP + TS | 0.53 | 0.24–1.23 | 0.41 | 0.33–0.5 | 0.9–1.5 | 10–40 |
| TS | 0.36 | 0.21–0.75 | 0.42 | 0.35–0.5 | 1.2–1.4 | 5–40 |
| TS + CH | 0.28 | 0.18–0.53 | 0.43 | 0.4–0.5 | 1.6–1.8 | 10–40 |
| WRB | 0.65 | 0.45–0.75 | 0.4 | – | 0.9 | 26–30 |
| HF + WRB | 0.26 | 0.1–0.45 | 0.4 | – | – | – |
| LC + HF + WRB | 0.49 | 0.44–0.47 | 0.45 | – | 0.6 | 12–18 |
| LC | 0.58 | 0.25–0.8 | 0.53 | 0.46–0.6 | 0.8–1.4 | 20–50 |

losses $V_{LS}$ and $V_{LT}$ were measured at the rear of the shield and the tail, as can be seen in Figure 7. This suggests that the low face pressures resulted in over-excavation, causing in turn further ground movements at some distance behind the tunnel shield and that the control of face pressure is crucial to the reduction of the overall volume loss.

Figure 5 also shows that slightly larger volume losses were caused when tunnelling through the London Clay/Harwich strata, even though larger face pressure ratios were applied. This can be partially explained by the fact that while the tunnel axis lied only at slightly shallower depth in the London Clay, $K_o$ was significantly larger. For a given face pressure ratio, more stress relief occurred in the horizontal direction towards the tunnelling machine, thus giving rise to more significant face volume losses. Also, the strength and stiffness of London Clay are generally lower than those of the other strata.

Shirlaw *et al.* (2003) advocated that occasional low face pressures may have a significant influence on face volume loss. This is because small face pressures may cause over-excavation or stress relief, two processes that can hardly be reversed by subsequently increasing the face pressure.

Figure 6 shows the total volume loss versus the 10% percentile of the face pressure time series, normalised by the overburden stress. The data seems to support the assumption by which low face pressures adversely affect the total volume loss.

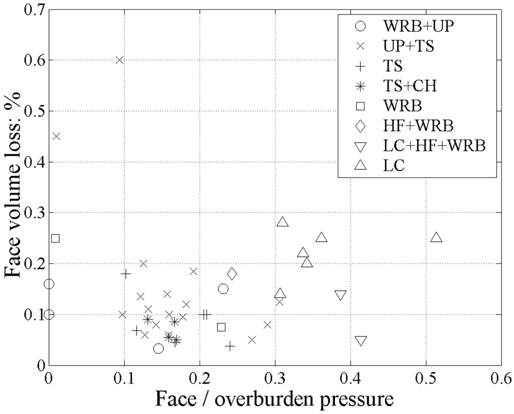


Figure 5. Relationship between face pressure ratio and face volume loss.

### 6.2 *Effect of bentonite injection around the shield*

The pressure of the bentonite injection around the shield was not found to correlate with the volume loss developing between the face and the shield. The injected volume did not offer a better measure of the effectiveness of shield grouting. Varying copy cutter stroke and loss of bentonite into the ground and into the plenum chamber partly explain the lack of correlation between the shield overcut filling ratio and the volume loss component $V_{LS}$.

In addition, the volume loss $V_{LS}$ has been found to increase as the volume loss component at the tunnel face $V_{LF}$ rises, as can be seen in Figure 7. This suggests that some proportion of the volume loss created occurring over the shield and behind is still due to the stress relief caused at the tunnel face.

### 6.3 *Effect of tail void grouting*

The effect of the normalised grouting pressure on the tail volume loss is shown in Figure 8. The grouting pressure was thought to be the variable governing

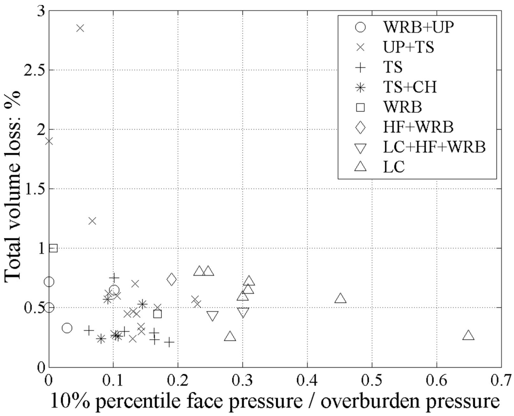


Figure 6. Influence of low face pressures on volume loss.

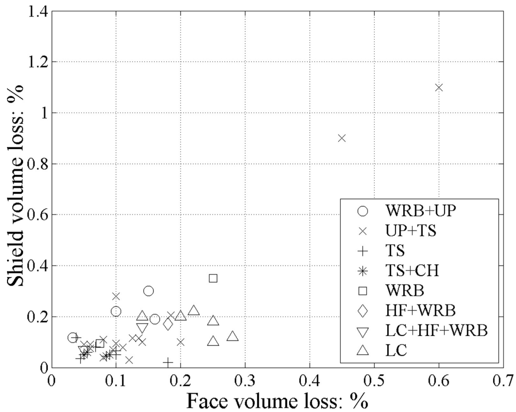


Figure 7. Relationship between face and shield volume loss.

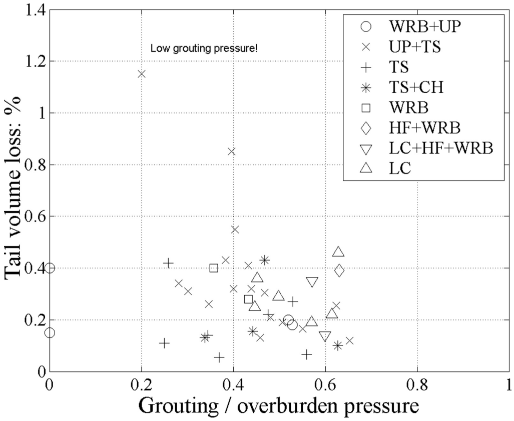


Figure 8. Influence of grouting pressure on tail volume loss.

$V_{LT}$, rather than the tail void filling ratio *TFR*. This is because the injected grout may be lost either through the tail seal or into the ground. The scatter in the data does not indicate a link between grouting pressure and tail void closure. It is believed that the scatter is due to the fact that some proportion of the volume loss occurring at the tail is directly affected by how the face pressure is controlled as the TBM passes the monitoring section and that, therefore, the influence of $p_f$ and $p_g$ cannot be fully decoupled as assumed above. It also has to be noted that the grouting pressure was not varied over a wide range throughout the drive and that the resulting volume losses are relatively small. Clearer correlations are therefore hardly to be expected from such field data. From the practical point of view, we may however conclude that grouting pressures comprised between 0.4 and 0.6 times the overburden pressure were effective in keeping the component of volume loss at the tail to approximately 0.5%. Considering that full closure of the tail void would result in a volume loss of 6%, the grouting can be considered as a successful and necessary operation. It has to be emphasised, however, that the grouting pressures quoted above are average pressures over seven excavation-ring built cycles and are therefore not to be mistaken for the true "peak" injection pressures.

## 7 DISCUSSION

It has been found to be difficult to establish simple correlations linking the settlement data to their cause, i.e. the driving parameters of the tunnelling machine. Meaningful trends were sometimes observed between face, grouting pressures and the corresponding component of volume loss, but when so, then often with a considerable degree of scatter. Three comments should, however, be made.

Firstly, the remaining scatter suggests that other parameters ought to be accounted for and the data expressed in terms of other, perhaps more elaborate, non-dimensional groups. The tendency of the tunnelling machine to pitch and yaw, as well as the overcut sometimes varying by changing the position of the copy cutters have not been taken into account in the analyses presented herein.

Secondly, no detailed accounts were made for the stratigraphy and the geotechnical parameters in each section. Neither the earth pressure coefficient $K_o$ nor the strength and deformation properties of the ground at tunnel axis and above have – for simplicity – been included in these analyses. Also, specific features about the monitoring sections have been disregarded: surface surcharge pressure and the presence of structures in or on the ground adjacent to the monitoring section may have influenced the volume loss.

Thirdly, the volume loss data presented above were compared with machine driving parameters *averaged* over the zones defined in Figure 1. Considering the back-calculated values of $i_x$, the distance ahead of the tunnel at which ground movements are affected by the face pressure appears to have been underestimated.

## 8 CONCLUSIONS

The following practical conclusions may be drawn from the data reported herein:

- The shape and parameters of the transverse and longitudinal settlement troughs were found to agree with previously published data.
- Volume losses below 1% may consistently be achieved over long lengths of mixed ground conditions. EPB machines have been successfully used to excavate two 8.1 m diameter tunnels in a wide range of ground conditions while maintaining volume losses below 1%. Approximately a third of the total volume loss occurs above the tunnel face.
- The level of face and grouting pressure required to achieve this performance where around 20 and 50% of the overburden pressure, respectively.

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## REFERENCES

Attewell, P.B. & Woodman, J.P. (1982). Predicting the dynamic of ground settlement and its derivatives caused by tunnelling in soil. Ground Engineering, Vol. 15, No. 8, pp. 13–36.

Cording, E.J. (1991). Control of ground movements around tunnels in soil. General Report, 9th Pan-American Conference on Soil Mechanics and Foundation Engineering, Chile.

Dimmock, P.S. (2003). Tunnelling induced ground and building movement on the Jubilee Line Extension. PhD Thesis, University of Cambridge.

Mair, R.J. (2003). Theme Lecture, CIRIA conference.

Mair, R.J. & Taylor, R.N. (1997). Theme lecture: Bored tunnelling in the urban environment. Proc. 14th international conference on soil mechanics and foundation engineering, Hamburg, Balkema, Vol. 4, pp. 2353–2385.

Nyren, R. (1998). Field measurements above twin tunnels in London Clay. PhD thesis, Imperial College, London.

Peck, R.B. (1969). Deep excavations and tunnelling in soft ground. *Proceeding of the seventh international conference on soil mechanics and foundation engineering*, Hamburg, State of the Art Volume, pp. 225–290.

Shirlaw, J.N., Ong, J.C.W., Rosser, H.B., Tan, C.G., Osborne, N.H. & Heslop, P.E. (2003). Local Settlements and Sinkholes due to EPB Tunnelling. Proceedings Institution of Civil Engineering, Geotechnical Engineering, October, issue 4, pp. 193–211.

Sugiyama, T., Hagiwara, T., Nomoto, T., Nomoto, M., Ano, Y., Mair, R.J., Bolton, M.D. & Soga, K. (1999). Observations of ground movements during tunnel construction by slurry shield method at the Dockland Light Railway Lewisham Extension – East London. Soils and Foundations, Vol. 39, No. 3, pp. 99–112.