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The effects of boring a new tunnel under an existing masonry tunnel

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ABSTRACT: A 3 metre diameter tunnel bored in London Clay passed under a cut and cover, brick arch tunnel constructed in 1863. The existing tunnel carries the Metropolitan Line railway under the heavily trafficked Euston Road and has an 8.7m span. The new tunnel was formed using a TBM, but some deformation of the surrounding soil was inevitable. Investigations and analysis showed that deformation would not prejudice the safety of the existing tunnel. Real-time instrumentation and surveys measured movements in close approximation to the predicted behaviour.

1 THE TUNNELS

The Metropolitan Line was built in 1863. The construction is described by Baker (1885). The section under Euston Road is a brick arch built in cut and cover. Figure 1, taken from Baker's paper, shows a cross-section through the tunnel, which has an 8.7 metre (28' 6") span.

The foundations are just in London Clay, which is covered with about 8 metres of Terrace Gravel and made ground. The line carries busy traffic for 20 hours every day. It was necessary to convince London Underground Limited (LUL) that the crossing of the cable tunnel would be without detriment to the safety of passengers or the property of LUL. The cable tunnel, constructed for London Electricity plc, is a 3.0 metre outside diameter machine bored drive supported by a wedge block lining, which was generally expanded segmental lining without bolts. A Lovat full face machine without the balanced pressure facility was used. The vertical alignment is well within the London Clay, the depth to the tunnel axis being about 18 metres. At the 75 metre radius curve at the crossing bolted wedge blocks were used for the lining. The rate of progress was slightly reduced by the bolting procedure. A plan of the crossing is shown in Figure 2. The vertical gap between the foundation of the arch and the top of the new tunnel was 7 metres.

A condition survey showed that the masonry tunnel was in reasonable condition, but suffering from some ageing effects. There was considerable loss of

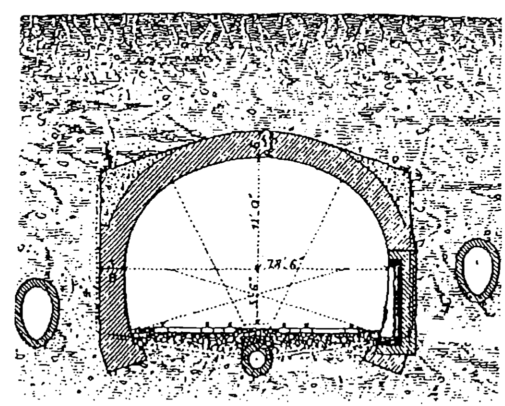


Figure 1. Metropolitan Line Tunnel.

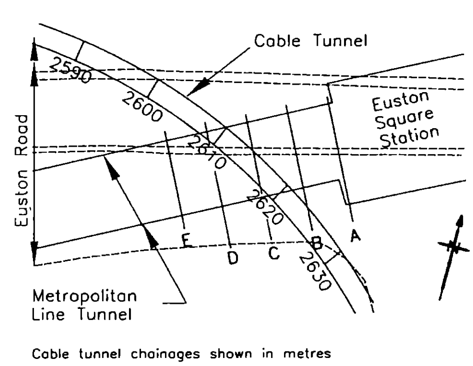


Figure 2. Plan of crossing.

mortar from the inner ring of brickwork, especially in the crown. On the walls many patches sounded hollow when struck with a hammer. A geometrical survey showed that the distance between the walls

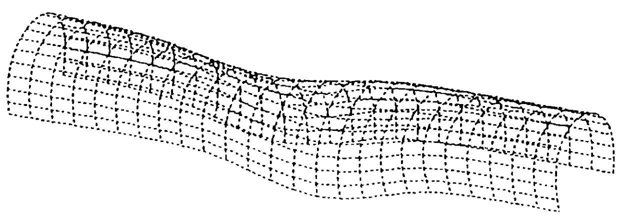


Figure 3. Deformed Superstress model.

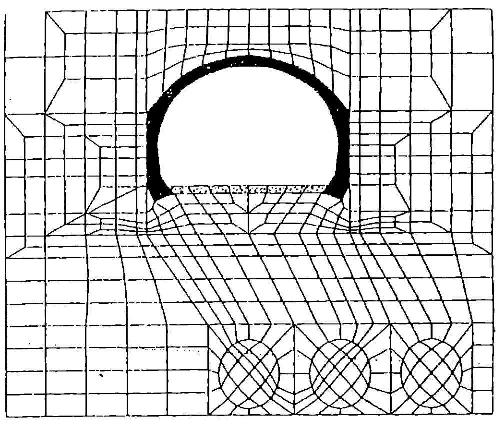


Figure 4. ICFEP model.

(P2 to P6 on Figure 6) was generally 75mm less than the dimension shown in Baker's drawings. The masonry was in places damp. The headwall, where the running tunnel opens into the station (Section A on Figure 2), was severely distorted, with cracks up to 20mm wide, probably as a result of differential stiffness during consolidation after the original construction.

Nine masonry cores were taken from the crown and walls of the tunnel. The cores were taken using a hand held barrel, but recovery was good. The jointing mortar appeared to be strong, but was absent from a few joints mainly near the intrados and the extrados. Intact core pieces up to 350mm long containing bricks and joints both parallel to and normal to the core axis were recovered. Unconfined compressive tests measured strengths between 6.9 and 22 N/mm^2. A design strength of 7.5 N/mm^2, which is consistent with the recommendation of BS 5628, was taken in the analysis.

2 INITIAL ANALYSIS

Settlement of 9mm at the railway tunnel foundations was calculated using an elasto-plastic approach, derived by Lu (1997). Consideration was given to a variety of volume losses including that associated with an unsupported face. Although data collected for the Lovat showed a volume loss of only 0.8% is achievable, it was agreed that a 1.5% volume loss would be used for design. Conditions worse than this were to be excluded by providing emergency procedures, which would ensure that the face was never unsupported. Adopting a volume loss of 1.5%, the empirical method outlined by O'Reilly and New (1982) gave a prediction of 12mm settlement at the tunnel foundations.

The effect of distortion of the masonry caused by settlement was studied using a Superstress analysis in which the expected soil deformation (with a maximum of 12mm) was imposed on the tunnel footings and the strains in the lining were calculated. The highly exaggerated shape is shown in Figure 3. The calculated strains indicated "negligible" damage to masonry according to Boscardin and Cording (1989). The analysis assumed the arch to be flexible relative to the soil, and to provide no resistance to soil movement. Having adopted this extreme assumption, the analysis gave confidence that the brick arch would not suffer damage.

The additional stiffness of the station headwall was expected to reduce deformations in the station structure. However some additional distortion was expected.

3 FINITE ELEMENT ANALYSIS

The Imperial College Finite Element Program was used (ICFEP, developed by Professor D.M. Potts of Imperial College). A section of the plane strain mesh is shown in Figure 4.

The cable tunnel was treated as running parallel to the brick tunnel, and was analysed in three positions, as shown in Figure 4. The London Clay was modelled as non-linear elastic perfectly plastic employing the model of Jardine et al (1986) pre-yield, and a Mohr-Coulomb yield surface and plastic potential post-yield. The made ground and Terrace Gravel were modelled as linear elastic perfectly plastic with Mohr-Coulomb yield surface and plastic potentials. The relevant equations and parameters are given in Appendix I. The brick arch was modelled as linear isotropic elastic with a Tresca yield surface defining compressive strength, and a limiting tensile strength model developed by Nyaoro (1989) defining tensile strength. The cable tunnel was lined with a continuous linear elastic ring. The relevant equations and parameters are given in Appendix II. The initial stresses were defined by a uniform unit weight of 20 kN/m^3 for all soil types and a water table 2 metres above the Terrace Gravel

/ London Clay interface, with hydrostatic pore water pressures below. K_o was equal to 0.5 in all strata down to the top of the London Clay, then 1.0 at the top of the London Clay increasing with depth to 1.5 over 10m, then remaining at 1.5 to the base of the London Clay.

The analysis was a coupled consolidation analysis, with the Terrace Gravel free draining and the London Clay having linear anisotropic permeability. The permeability in the vertical direction was 0.5 x 10^{-10} m/s and in the horizontal direction 1 x 10^{-10} m/s. Great care was taken to simulate the construction sequence of the brick arch, including lowering of the water table, original excavation and backfilling for the brick arch, and applying consolidation stages as appropriate. Live loads were then applied and removed at the ground surface to represent traffic on the Euston Road. Interesting outcomes of the analysis of the arch before the cable tunnel was constructed were that:

On backfilling the brick arch experienced tension at the intrados at the crown.

The ground under the tracks was in a passive condition at the end of construction.

The ground outside the walls was in an active condition and the walls moved inwards.

The distance between the walls was predicted to reduce by 80 mm at track level over the 133 year consolidation period (This compared with the 75mm reduction of distance between the walls inferred from measurements).

At the end of construction there was cracking at the extrados at the shoulders, but the compressive stress in the masonry at the intrados was permissible.

Under live traffic loads there was some cracking induced at the intrados at the crown, but the compressive stress in the masonry at the extrados was permissible.

It seems that Baker's intuitive shaping of the foundation with inward sloping footings was entirely correct without the benefit of numerical analysis.

The construction of the cable tunnel directly under one footing of the arch (see Figure 4) with a face loss of 1.5% resulted in a predicted immediate settlement of about 5mm at the footing and an inward movement of about 3mm, as shown in Figure 5. The incremental stresses induced in the arch were opposite to those occurring under dead load and caused a small reduction in the net stresses. Thus no additional cracking of the brick arch was likely. Settlement of one footing did not cause significant additional stress on the other footing and thus no foundation overstress was to be expected.

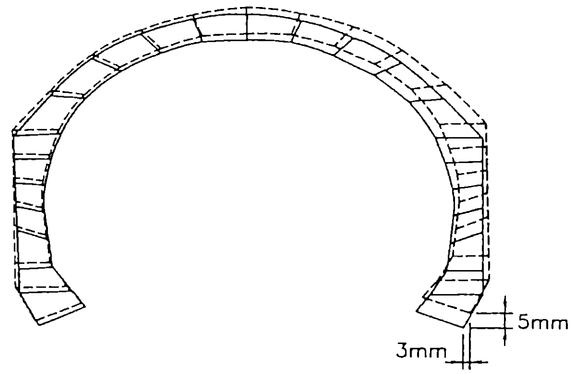


Figure 5. Predicted deformations of the arch due to tunnel construction.

The analysis was continued to the long term to assess any additional effects of consolidation associated with the new tunnel acting as a drain. The analysis predicted an additional footing settlement of 5mm. No masonry cracking was predicted.

4 INSTRUMENTATION

Figure 6 shows the locations of some of the instruments installed in the masonry tunnel.

As the crossing of the cable tunnel was expected to take about eight hours and would therefore be largely in traffic hours, instruments were installed to monitor real time deformations. 34 electrolevels were installed to measure both transverse and longitudinal angular strains at five cross sections labelled A (the station headwall), B, C (centre line of the crossing), D and E on Figure 2. Also crack meters were installed on some cracks in the brick arch and on the headwall.

It was expected that as these cracks were continuous, the strains would be concentrated on them. An alert level of 1 in 500 was set on the electrolevels at which "slight" damage is predicted by Boscardin and Cording (1989). This is at least twice the angular strain predicted by the Superstress analysis assuming 12mm maximum settlement as predicted by the empirical method of O'Reilly and New (1982).

An array of survey targets was installed at 5m intervals for a 30 metre length of the masonry tunnel. This surveying method was accurate to 1mm. The survey was backed up by reading chord lengths between lugs installed in the brickwork with a tape extensometer which is accurate to about 0.1mm after correction for temperature.

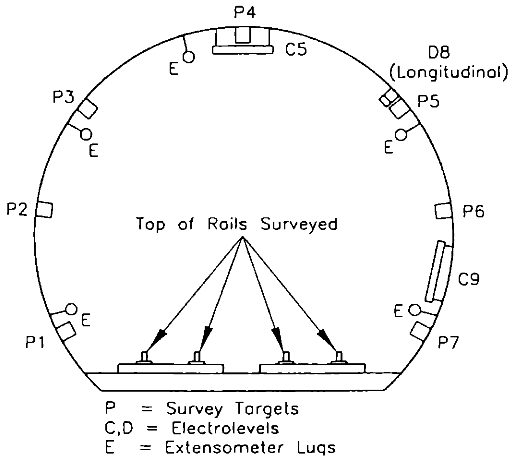


Figure 6. Schematic layout of selected instruments and survey points.

5 THE CROSSING

Before the crossing, a protective screen of aluminium mesh was constructed inside the masonry tunnel in case bricks became loosened as a result of deformations. The TBM was serviced and boarding was made ready to support the face should the TBM stop under the tunnel for any reason (10 days earlier it had intersected an unrecorded brick lined well). Working was continuous and proceeded at the planned rate of about 1.5 metres per hour. During the crossing the instruments were constantly monitored and first started to record consistent trends at midnight when the cable tunnel was 5m from the north wall of the rail tunnel as shown in Figure 7. The instruments responded in a systematic and logical manner throughout the crossing. The maximum angular strain measured was 1 in 2000 at the crown when the tunnel was half way across, as shown by instrument C5 in Figure 7. On the north wall, instrument C9 recorded outward movement towards the approaching tunnel which reversed as the face passed under the wall. Longitudinal

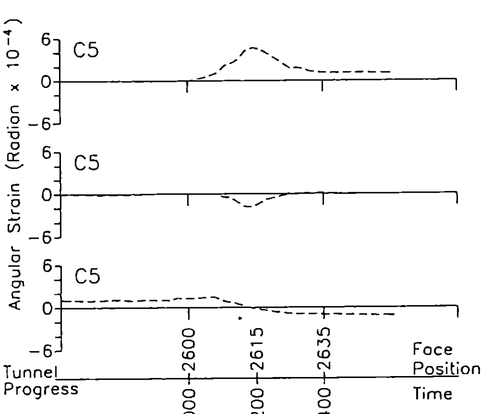


Figure 7. Real time rotation of electrolevels.

instrument D8 shows the influence of the cable tunnel approaching on a skew. It tilted clockwise as the cable tunnel approached from the west and then reversed to anticlockwise after the tunnel passed under the south wall of the arch. As shown in Figure 7 (D8), the residual rotation (1.5×10^{-4} radians) is in close agreement with the survey results shown on Figure 8. The crack meters recorded negligible movements indicating that the cracks were not of structural significance. Angular distortions on the headwall were insignificant although the trough of settlement was detectable into the station area.

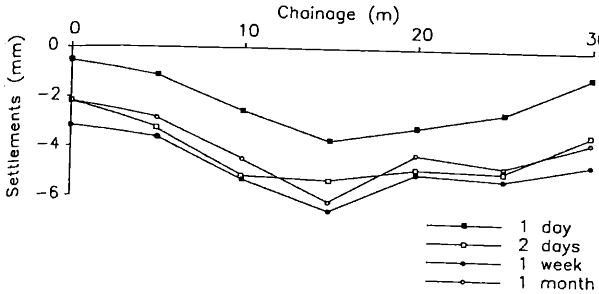
The instruments were left in place for 14 days after the crossing, but only small further rotational movement was recorded after the face of the cable tunnel had passed beyond the south wall.

Displacement surveys were carried out immediately after the crossing and at one day, 7 days, 30 days and 6 months afterwards. Unfortunately several survey targets were lost when the protective mesh was removed. The short term settlements along the length of the masonry tunnel are shown in Figure 8a. The maximum observed immediate settlement was 5mm at the crown. A further settlement of 5mm had taken place by six months (Figure 8b). These observations agree well with the ICFEP predictions. As settlement continued the points of inflection of the trough moved away from the plane of symmetry of the tunnel compared with the short term data. It is interesting that the instruments measuring rotation, which are more sensitive than the survey, did not pick up this further movement. This is because the longer term movements are more uniform and do not involve significant changes in angular strain.

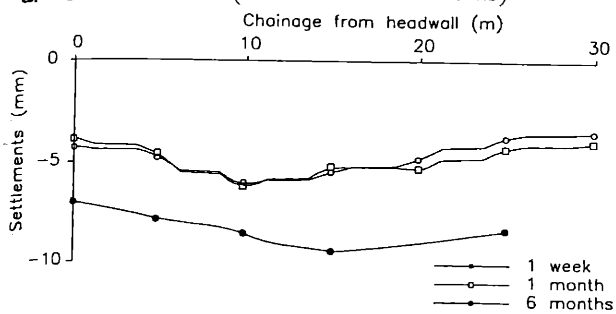
The lateral movements were all too small to be detected by the survey system. Both the survey and extensometers recorded changes in chord length (P2 to P6 on Figure 6) of less than 2mm throughout. No visible change in the appearance of the brickwork was observed in the condition survey undertaken after completion of the new tunnel.

6 CONCLUSIONS

Although, as it turned out, empirical methods of estimating settlements were found to be conservative compared with finite element model, the rigorous level of proof required by London Underground that their passengers and property would not be put at risk demanded soil-structure interaction calculations. In effect a Class A prediction was required. The ICFEP model not only



a. Crown of arch (short term settlements)



b. Along south wall (short and long term settlements)

Figure 8. Settlement in longitudinal direction.

evaluated the stresses and strains in the masonry tunnel since construction and accurately predicted the immediate deformations during the crossing, but it also accurately predicted the longer term deformations.

The analysis enabled the crossing by the new tunnel to be carried out with confidence. The instrumentation and survey showed that the predictions were met to a high level of accuracy.

The crossing took place without disruption of the railway operations and without damage to the masonry tunnel.

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APPENDIX 1. SOIL MODELS AND PARAMETERS

Non-linear elastic model:

$$\frac{3G}{p'} = C_1 + C_2 \cos\left[c_1 \left(\log_{10}\left(\frac{E_d}{\sqrt{3}C_3} \right) \right)^{c_2} \right] \qquad (1)$$

$$\frac{K}{p'} = C_4 + C_5 \cos\left[c_3 \left(\log_{10}\frac{(\varepsilon_v)}{C_6} \right)^{c_4} \right]$$

where G is the secant shear modulus, K is the secant bulk modulus, p' is the mean effective stress, E_d is the deviatoric strain invariant used in ICFEP, ε_v is the volumetric strain, and C_1, C_2, C_3, C_4, C_5, C_6, c_1, c_2, c_3 and c_4 are all coefficients. E_d is related to ε_a (the axial strain observed in undrained triaxial test) by the expression:

$$E_d = \sqrt{3}\,\varepsilon_a$$

where:

$$E_d = 2\sqrt{\frac{1}{6}\left((\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_1 - \varepsilon_3)^2 + (\varepsilon_2 - \varepsilon_3)^2 \right)}$$

ε_1, ε_2 and ε_3 being the principal strains.

The coefficients are obtained from a fit to laboratory data from stress path tests (Jardine et al. 1986). Throughout an analysis the stiffness at a particular point is continually changing. It depends on both the current strain and the current mean effective stress at the point. Until a minimum strain, $E_{d\ min}$ or $\varepsilon_{v\ min}$ is exceeded the stiffness varies only with the mean effective stress. This condition also applies once a specified upper strain limit is exceeded, $E_{d\ max}$ or $\varepsilon_{v\ max}$. $E_{d\ min}$, $\varepsilon_{v\ min}$ and $E_{d\ max}$, $\varepsilon_{v\ max}$ are required 'cut-offs' because of the trigonometric nature of Equations 1. The magnitude of the stiffness is prevented from falling below specified minimum values (G_{min} or K_{min}).

Figure 9 shows the stiffness - strain curve adopted for

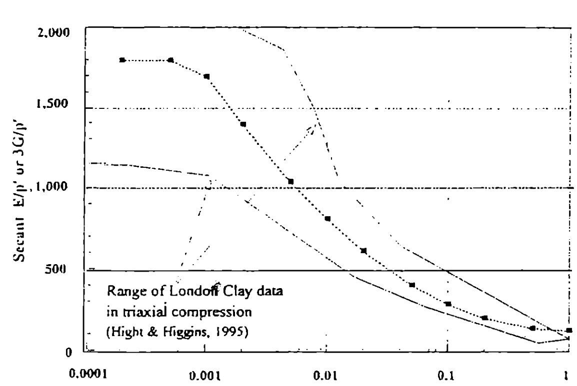

Figure 9. Stiffness against strain for London Clay.

London Clay, plotted as E_u/p' (where E_u is the undrained Young's Modulus), which is equal to $3G/p'$, against axial strain in a triaxial test. For comparison the upper and lower bounds to London Clay data from Hight and Higgins (1995) are plotted.

Table 1. Parameters employed in non-linear elastic model for the London Clay.

C_1	1400	c_3	2.069
C_2	1270	c_4	0.420
C_3 (%)	1.0×10^{-4}	$E_{d\,min}$ (%)	8.660250×10^{-4}
C_4	686	$E_{d\,max}$ (%)	0.69282
C_5 (%)	633	$\varepsilon_{v\,min}$ (%)	5.0×10^{-3}
C_6	1.0×10^{-3}	$\varepsilon_{v\,max}$ (%)	0.15
c_1	1.335	G_{min} (kPa)	2666.7
c_2	0.617	K_{min} (kPa)	5000.0

Table 2. Linear elastic parameters for made ground and Terrace Gravel.

	Made ground	Terrace Gravel
Young's Modulus (kPa)	5000	15000
Poisson's Ratio	0.2	0.2

Table 3. Mohr Coulomb yield surface and plastic potential.

	Made ground	Terrace Gravel	London Clay
Strength parameters	$c' = 0$ kPa $\varphi' = 35.0°$	$c' = 0$ kPa $\varphi' = 35.0°$	$c' = 0$ kPa $\varphi' = 23.0°$
Angle of Dilation	$\nu' = 0.0°$	$\nu' = 17.5°$	$\nu' = 11.5°$

APPENDIX II. TUNNEL MODELS AND PARAMETERS

Limiting tension yield function for masonry.

The ductile limiting tension yield surface is very simply defined by:

$$\sigma_1 = \sigma_T \qquad (2)$$

where σ_1 is the major principal stress, and σ_T the tensile strength of the material.

Metropolitan Line masonry parameters:

Young's Modulus	8.5×10^6 kPa
Poisson's Ratio	0.15
Compressive strength	7,500 kPa
Tensile strength	100 kPa

London Electricity cable tunnel parameters:

Young's Modulus	28.0×10^6 kPa
Poisson's Ratio	0.15
Cross sectional area	0.168 m^2/m
Second moment of area	3.9514×10^{-4} m^4/m

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