Distributed fibre optic monitoring of the time-dependent behaviour of tunnel segmental linings in London clay

*Hyungjoon Seo 1) Matthew John Wilcock 2) Kenichi Soga 3) Mohammed Elshafie 4) Robert James Mair 5)

1) Department of Civil Engineering, Xi'an Jiaotong Liverpool University, Suzhou 215123, China
2), 4) Engineering Department, University of Cambridge, Cambridge CB2 1PZ, United Kingdom
3) Department of Civil and Environmental Engineering, University of California, Berkeley, 94720-1710, United States

ABSTRACT

In order to meet increasing electricity demand and help the city of London to access the renewable energy of the future, National Grid plc, the operator of the United Kingdom’s high-voltage electricity transmission embarked, in February 2011, upon a seven-year project to rewire the capital via new deep underground tunnels. The National Grid London Power tunnels were constructed mostly in London clay using an Earth Pressure Balance (EPB) shield tunnel boring machine (TBM). This paper discusses the monitoring of National Grid Power Tunnel’s new segmental concrete tunnel linings using embedded distributed optical fibre strain sensing. Fibre optic cables were installed within 36 segments (six complete rings) during manufacture in order to monitor distributed strain in the segments following their installation into the ground. In this paper, the development of the hoop and bending strain results from six segments (together making one whole ring) are presented during the first 12 months following construction; the bending moments of the segments (calculated from the fibre optic strain data) were compared to the bending moment capacity calculations and the axial stresses (calculated from the fibre optic strain data) were compared to the overburden pressure. The results show increasing bending moments and circumferential load in the segments with time as would be expected due to the time dependent behaviour of London Clay. However, the results also showed that the circumferential stresses reached 80% of the total overburden after one year; this value is relatively higher than the equivalent data obtained from open face tunnelling in London Clay. The results

---

1) Lecturer, Email: hyungjoonseo@xjtlu.edu.cn
2) PhD student, Email: mjw207@cam.ac.uk
3) Professor, Email: soga@berkeley.edu
4) Lecturer, Email: me254@eng.cam.ac.uk
5) Professor, Email: rjm50@cam.ac.uk
highlight the different response of segment tunnel linings when subjected to different tunnelling techniques.

1. INTRODUCTION

The National Grid Plc., the operator of the United Kingdom’s high-voltage electricity transmission system, embarked upon a seven-year project to rewire the capital via new deep underground tunnels in order to meet increasing electricity demand in London. The London Power Tunnels programme, upon completion, will create ten new 400kV circuits at the heart of the capital’s transmission system (National Grid, 2015). The tunnelling scheme consists some 32km of new deep level tunnel, 15 large diameter shafts and 14 head house buildings that will sit on top of the shafts. The tunnel’s alignment follows, to a large extent, a major road route through the City of London from Willesden in the west to Hackney in the east, passing through Kensal Green. From Kensal Green, a further tunnel navigates south through Wandsworth, terminating at Wimbledon in the south as shown in Figure 1. Construction work started in 2011 with the scheme planned to be fully operational by 2018.

The tunnels between Willesden and St Johns Wood (2km east of Kensal Green) are constructed with a 3.16m internal diameter expanded concrete segmental lining. From Kensal Green to Wandsworth within water bearing variable ground conditions, tunnel construction is a 3m bolted trapezoidal lining while from Wandsworth to Wimbledon the tunnel lining reverts to 3.16m expanded type. Between St Johns Wood and Hackney, the tunnel is a much larger 4m diameter, as this tunnel will carry an additional two circuits of 132kV cabling. The 4m tunnel lining is trapezoidal type, 1.3m wide, with six segments forming each ring. All tunnelling is conducted using earth pressure balance (EPB) tunnel boring machines (TBM).

There has been significant research and compilation of field monitoring data on geotechnical aspects of tunnelling such as tunnel stability (Broms and Bennermark, 1967; Mair and Taylor, 1993; Davis et al., 1980; Mair et al., 1981) tunnelling-induced
ground movements (O’Reilly and New, 1982; New and O’Reilly, 1997; Standing et al., 1996; Rankin, 1988; Mair and Taylor, 1993; Lake et al., 1992; Bowers et al., 1996) and the effect of these movements on adjacent structures (Burland and Wroth, 1974; Mair et al., 1996; Potts and Addenbrooke, 1997; Standing, 2001; Franzius, 2004). However, only a limited number of studies focused on the field measurement of the performance of concrete segmental lining during and following construction. The next section summarises some of these studies and explains the rationale behind the work presented in this paper.

2. NATIONAL GRID LONDON POWER TUNNEL – SITE DESCRIPTION AND INSTRUMENTATION DETAILS

The sector of the London Power Tunnels between St Johns Wood and Hackney was constructed approximately 30 m below ground surface with the tunnel totally embedded in London clay as shown in Figure 2; the external diameter of tunnel is 4.5 m and internal diameter is 4 m.

![Figure 2. Ground condition](image)

If any standard single mode fibre optic cable, as is used in telecommunications, is strained (stretched), the change in strain (relative to a reference reading) can be determined by a Brillouin Optical Time Domain Reflectometry (BOTDR) analyser, from which the strain can be interrogated at every point along the cable. The fibre optic cable may extend more than 10km in length. This is akin to having a continuous string of conventional foil, vibrating wire or fibre Bragg grating strain gauges mounted in series.
As a very large number of conventional gauges would be impractical, their implementation will only ever measure discrete points chosen through engineering judgement. A BOTDR-based system can have cables routed continually around the structural element in question; the cables can be routed in different ways to measure strain in different directions in the element. Particularly in the context of geotechnical structures, the true state of the structure may not easily be understood unless the full strain profile is known. Therefore, by utilising BOTDR the performance of a structure can be quantified in a more detailed and much more useful manner (e.g. Klar et al. 2006; Cheung et al., 2010; Mohamad et al., 2010, 2011, 2012, 2014; Soga, 2014; Gue et al., 2015).

Distributed fibre optic sensors were installed along with the reinforced cages as shown in Figure 3. Strain and temperature cables were installed on same location for the temperature compensation in longitudinal and hoop section of reinforced cage (from point ‘A’ to ‘S’ as shown in Figure 3). The intradoses of reinforced cage are located on section D to E and Q to R and the extradoses of reinforced cage are located on section L to M and H to I respectively. In order to distinguish the data between longitudinal and hoop sections easily, a pre-strain was applied to only longitudinal sections. In this paper, the data obtained at hoop directions were selected to estimate the circumferential load variations with time.

An EPB TBM excavated the ground having a balance with respect to overburden load as shown in Figure 4a. The segments that fibre optic sensors were embedded were installed on site with the same way as other segment installations. All stain and temperature fibre optic cables embedded in each segment have to be connected each
other to make a single cable (see Figure 4c). Eleven splicing were performed to connect these fibre optic cables and there was few power loss from this slicing. A spliced cable can be plugged into the analyser and then every data of temperature and strain cables can be taken as shown in Figure 4d. The schematic diagram of the spliced cable is shown in Figure 4e. The data started to be taken on 14th January 2014 and monitored after 37 days, 93 days, 157 days, 190 days, and 379 days respectively. The strain profile along the cable can be estimated by monitored central frequency data and then the variations of circumferential load and moment can be estimated by this strain profile.
BOTDR is a method for monitoring the reflected laser coming from each sampling point so that only one cable end has to be selected to plug into the analyser. In this study, temperature cable end was selected to take data but strain cable was also used in a certain period of time because the strain data were not possible to be taken between segment 3 and 4 due to a breakage of cable. Therefore, it was not possible to obtain all strain data of segments 4, 5, and 6 in cases of 37 days and 93 days. The detail monitoring schedule is shown in Table 1.

Table 1. Cable types connecting analyser at each monitoring date

<table>
<thead>
<tr>
<th>Date</th>
<th>1 day</th>
<th>37 days</th>
<th>93 days</th>
<th>157 days</th>
<th>190 days</th>
<th>379 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable types</td>
<td>Strain</td>
<td>Temperature</td>
<td>Strain</td>
<td>Temperature</td>
<td>Strain</td>
<td>Temperature</td>
</tr>
</tbody>
</table>

3. DISTRIBUTED FIBRE OPTIC SENSOR

The distributed fibre optic strain measurement is based on the Brillouin scattering; when an optical pulse travels along a fibre, a frequency-shifted component, the Brillouin component, is back reflected. The Brillouin scatted light has roughly 25 to 30 MHz bandwidth and the central peak frequency is around 11 GHz for standard single mode fibres when no strain is applied. The back-scattered Brillouin central frequency \( v_s \) is related to the input light based on the following equation.

\[
v_s = \frac{2n_f v_a}{\lambda}
\]

Eq. (1)

where \( n_f \) is the fibre core refractive index, \( v_a \) is the acoustic velocity in the fibre and \( \lambda \) is the wave length of the input light.
If a specific load applies to a certain location of strain sensor, the frequency of the Brillouin scattering is changed at the applied point (see Figure 6). The shift amounts of central frequency can be converted to the strain and the behaviour of the structure can be estimated by the compensated strain via the following equation.

$$\Delta \varepsilon_s = \frac{\Delta v_a - (C_s \alpha + C_T) \Delta T}{C_s}$$

Eq. (2)

where, $\Delta \varepsilon_s$ is differential strain due to deformation of the structure, $\Delta v_b$ is Brillouin frequency shift, $C_s$ is strain coefficient (480 MHz / % for strain cable), $\alpha$ is thermal expansion coefficient of structure (10 με /ºC for concrete structure), $C_T$ temperature coefficient (0.96 MHz / ºC for temperature cable), $\Delta T$ is differential temperature.

If only mechanical strain is of interest, as in this application, the temperature effect has to be compensated for. This is usually done by installing a loose-tube FO cable (referred to as the temperature cable) in parallel to the strain-sensing fibre. The fibres in the loose-tube cable swim in a liquid gel as shown in Figure 5b, and are therefore not affected by structural movement. In this study, the temperature cable manufactured by @@ was used. For the strain-sensing fibre, the reinforced FO cable manufactured by Fujikura Ltd is used; to resist the harsh environment, this cable has four glass cores protected by a thick nylon sheath, and is strengthened by two steel wires as shown in Figure 5a. The fibre optic cables are not less likely to be affected by environmental conditions and they can be potentially used as long-life sensor because it is an inert glass material.

(a) Strain cable
(b) Temperature cable

Fig. 5 Distributed fibre optic sensors
As the speed of light is constant, the location can be evaluated by measuring the time elapsed between when the light was pumped into the fibre and the time at which it comes back to the analyser. By re-solving both time and frequency, it is therefore possible to generate a continuous strain or temperature change profile along the fibre for distances of a few kilometres or more.

The current state of the art distributed fibre optic strain measurement systems provide data in the micro-strain range with a spatial resolution (strain is averaged over a specified gauge length) of 0.2 m or less. This means that it is possible to have thousands of ‘strain gauges’ along a single cable connected to structures, or embedded in civil engineering infra-structure. In this study, we use the Yokogawa analyser to acquire the data with 0.1 m sampling resolution, 1.0 m special resolution and 30 με resolution.

4. RESULTS OF FIBRE OPTIC MONITORING

This paper discusses the one year monitoring of the fibre optic embedded tunnel segments installed within London clay. Figure 7 shows the central frequency variation of strain and temperature. After taking a baseline, the strain cable was broken between segments 3 and 4 so that we started to take readings using both strain and temperature ends. In the raw data of strain at each segment, four peaks can be found because the pre-strain applied at the longitudinal direction of the segment lining as mentioned in Figure 3. In order to determine the circumferential load variations of segment lining, the data between peaks were taken.
The differential strain profile can be obtained by subtracting the all obtained data from the baseline. The baseline was monitoring data obtained on the first day. The differential strains were also temperature compensated. As shown by the first segment of the distributed data in Figure 7a, temperature does not change with the time because this tunnel was constructed 30 m below the ground surface. Figure 8 shows the development of strain with time. Number (2) and (3) mean the fibre optic cables installed within the extrados of segment and number (1) and (4) mean the fibre optic cables installed within the intrados of segment. As shown in Figure 4c, Segments 1, 3, 4, and 6 are located at the tunnel shoulder whereas segments 2 and 5 are located at the tunnel crown and invert. The data show that large compressive strains are developing with time within the extrados of Segments 1, 3, 4, and 6 and
within the intrados of Segments 2 and 5, respectively. In the other hand, tensile strains are developing with time with the intrados of Segments 1, 3, 4, and 6 and with the extrados of Segments 2 and 5. This means that bending moments are developing with time.

Mair and Taylor (1997) introduced that most tunnel excavated by open face TBM within London clay have a squatting shape with the time increasing even though the coefficient of earth pressure may be greater than 1. However, according to the results of differential strain, this tunnel is exhibiting a long egg ing shape as shown Figure 9. The tunnel was constructed by an earth pressure balanced machine with face pressure...
of almost 35 % of the overburden pressure. The tunnelling induced ground movements were very small and hence the stress release by excavation was minimised.

![Fig. 9 Expected deformation of tunnel](image)

In order to calculate the bending moment and the axial force, a single data point was selected in the middle of the circumferential strain cable as shown in Figure 10. The results of Segments 1, 3, 4, and 6 show that compressive behaviour is dominant and the compressive strains of the extrados are significantly greater than these of the intrados. The horizontal stress of the ground strongly applies to those segments than the vertical stress. The relative strain difference between the extrados and the intrados is 159.34 με in average. The extrados of Segments 2 and 5 show tensile strains and small compressive strain, respectively. The intrados show compressive strains which means that horizontal stress of the ground strongly applies to those segments as same results as segments 1, 3, 4, and 6 and hence directions of moments are expected as expressed in Figure 9. The relative strain difference between the extrados and the intrados is 134.82 με in average. Based on the results of relative strain difference at each segment, the bending moment can be calculated as shown in Table 2. The results of bending moment show that Segments 1 and 4 have relatively larger bending moments than the others.
Table 2. Moment variations of each segment with time

<table>
<thead>
<tr>
<th>Day</th>
<th>Moment (kN·m)</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
<th>Segment 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>37</td>
<td>-13.64</td>
<td>2.66</td>
<td>-13.64</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>93</td>
<td>-25.25</td>
<td>12.48</td>
<td>-18.82</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>157</td>
<td>-33.51</td>
<td>19.69</td>
<td>-15.67</td>
<td>-45.93</td>
<td>31.50</td>
<td>-20.95</td>
<td></td>
</tr>
<tr>
<td>190</td>
<td>-28.59</td>
<td>21.43</td>
<td>-18.90</td>
<td>-47.13</td>
<td>28.21</td>
<td>-23.52</td>
<td></td>
</tr>
<tr>
<td>379</td>
<td>-33.27</td>
<td>28.22</td>
<td>-22.92</td>
<td>-53.31</td>
<td>34.69</td>
<td>-28.79</td>
<td></td>
</tr>
</tbody>
</table>

Figure 11 shows the bending moment / axial force (M/N) envelope of the ULS (Ultimate Limit State) load cases for Muir Wood (1975) and Duddeck and Erdmann (1985) for zero shear and full shear interaction with the ground. Concrete Society Technical Report (2004) suggested the design of Steel Fibre Reinforced Concrete (SFRC). The stiffness used for the calculation is shown in Table 3. The axial loading on every segments are within the envelope and are smaller than those calculated within design during 1 year. In short term (one to three months), the bending moment reaches up to almost 50 % of the critical state in average for a given axial load. In the long term (six to twelve months), the axial loads increase with less increase in bending moments and the stress state reduces to 25 % of the critical state in average. From these results, the increase of axial load due to the overburden pressure is more dominant than the increase of the moment over the time. This implies that the ground gives more isotropic load increments with time.
Fig. 11 Bending moment variation with time

Table 3. Stiffness of Steel Fibre Reinforced Concrete (SFRC)

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>$f_{cd}$ (MPa)</th>
<th>$f_{R1}$ (MPa)</th>
<th>$f_{R4}$ (MPa)</th>
<th>$f_{ck}$ (MPa)</th>
<th>Factor of safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>31.5</td>
<td>2.5</td>
<td>2.0</td>
<td>50</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Where, $f_{cd}$ is design compressive stress of concrete, $f_{R1}$ and $f_{R4}$ are the characteristic residual strengths at CMOD (crack mouth opening displacement) of 0.5 and 3.5mm respectively derived in the BS EN 14651 beam test, and $f_{ck}$ is characteristic compressive cylinder strength.

Figure 12 shows that the development of axial stress on the segment linings with the time. The vertical stresses acting on Segment 1, 3, 4, and 6 are larger than the horizontal stresses acting on Segment 2 and 5 (see Figure 12a). This may be due to the egging effect, in which horizontal stress reduces and vertical stress increases in relative terms.

The tunnel was installed almost 30 m below the ground surface. The measured axial stresses can be converted to the equivalent soil pressures. Figure 12b shows the normalized soil pressures against the vertical total overburden stress at the axis level. The equivalent soil pressures acting on the segment linings reach up to 80 % of the overburden pressure after 1 year.
There are several field monitoring data that show an increase in axial load with time within London clay. Most of the cases are from open face tunnel excavation with instruments using load cells or strain gauges. Barratt et al. (1994) monitored a segment lining with vibrating wire load cells installed between the segments. The measurement was made on a 4.15 m outer diameter tunnel lined with concrete segments and the tunnel was constructed by an open face TBM at a depth of 20 m in London clay at Regent’s Park.
The measurements were made for a period of 20 years and the applied load reaches up to almost 50% of the overburden in average as shown in Figure 13a. In this tunnel, the volume loss was 1.3% and K0 was estimated to be 1.5 to 2 prior to tunnel construction. Bowers and Redgers (1997) also used load cells installed between segments. Their monitored tunnel was constructed by an open face TBM at a depth of 21 m in London clay for the Jubilee Line Extension project. The outer diameter tunnel was 4.85 m and the volume loss was 2.9 to 3.3% after tunnel construction.

The measured soil pressures in this study increase faster than the previously monitored cases in London clay. They reach up to almost 50% of the overburden in average within four months, compared to years in the other cases. After one year, the soil pressures measured in this study are up to 80% of the overburden. This difference may be due to the method of tunnel excavation (i.e. EPB in this study versus open face in the other cases). The earth balance pressure minimizes the stress release of the ground induced by excavation and the volume loss in this case was 0.7%. When the volume loss is smaller, higher soil pressure is expected (see Figure 13b).
Mair and Taylor (1997) expressed the lining load variations as a percentage of the load equivalent to the overburden pressure. These data was collected from the load cell monitoring of relatively short term which is from one week to one year. Figure 14 shows the lining load variation against the C/D ratio for each tunnel, where C is cover above the tunnel crown and D is the tunnel diameter. The other projects except National Grid tunnel were excavated by open face TBM. In cases of open face TBM, applied load to segment lining has a trend to decrease with C/D value increase. The load increasing ranges between one week and one year is relatively lower as well. However, although C/D ratio of National Grid tunnel is higher than the other, increment of the load applied to the segments is higher than the other.
In this study, the authors have attempted to evaluate the circumferential load variations of segment lining during one year by using distributed fibre optic sensors. The important conditions to affect the results of this study are as follows: to use distributed fibre optic sensors; to excavate the tunnel by EPB TBM; to construct in London clay. Findings from this study are summarized as follows:

The analysis of the monitoring data from the continuous BOTDR technology for National Grid power cable tunnel highlight the relative benefits of using a distributed sensing technique, such as the BOTDR in identifying specific points of significant interest, as compared to the limited information from discrete monitoring systems, such as VWSG.

(1) The BOTDR distributed monitoring system is able to provide a continuous profile of the induced strain within tunnel segments and this offers more confidence in determining the developed circumferential load and bending moment profiles. The embedded fibre optic sensors are also ideal for long term monitoring of structures due to the material itself.

(2) The large compressive strain is developing within the extrados of the tunnel shoulder (segments 1, 3, 4, and 6) and within the intrados of the tunnel crown and invert (segments 2 and 5), respectively. Otherwise, tensile strain is developing with the intrados of the tunnel shoulder and with the extrados of the tunnel crown and invert. Even though most tunnel excavated by open face TBM within London clay have a squatting shape with the time increasing, this tunnel excavated by EPT TBM has a long egging shape. Because earth balance pressure applies almost 35 % of the overburden pressure to the tunnel face.

(3) The circumferential load of segment lining already reaches up to almost 80 % of the overburden after one year. This result is much higher than the other results excavated by open face TBM with London clay. The volume loss is also about 0.7 % which is less than the other results excavated by open face TBM with London clay as well. Because earth balance pressure helps to reduce the stress release of the ground induced by excavation.

The results of this paper show that the circumferential load of segment lining in London clay is influenced by the tunnel excavation methods. In case of EPB TBM, the earth balance pressure can make the stress release of the excavated ground minimize. Because of this, the applied load of the segment increases as close to overburden load. In other words, EPB TBM can get higher stability of tunnel face and lower ground settlement but segment lining can have higher stress because of lower stress release of excavated ground.
REFERENCES


