A 20 YEAR HISTORY OF STRESS AND STRAIN IN A SHOTCRETE PRIMARY LINING

Benoît D Jones
Inbye Engineering, United Kingdom
bjones@inbye.co.uk

What happens to tunnels in the long-term? Does the ground load increase to the in situ value or does it reach an equilibrium at a lower value? What is that lower value? Is there load sharing between a primary and secondary lining? Designers mainly have to guess the answers to these questions because there are so few data. This results in significant overdesign of tunnel linings. The lack of data is partly due to the difficulties of measuring stresses in tunnel linings, partly due to the planning and commitment required to continue to monitor tunnels for long periods of time after handover to the client, and partly due to a bizarre lack of interest on the part of the tunnelling industry in a subject that should be of critical interest. This paper will present a summary of nearly 20 years of monitoring stresses and strains in the Heathrow Terminal 4 Station Concourse Tunnel primary lining. It will compare these measurements to other measurements made in the literature and come to some conclusions about how we have been getting things wrong and how we should design tunnels in the future.

INTRODUCTION

Tunnel lining design is inherently conservative, in that characteristic values of ground strength parameters and characteristic values of the tunnel lining materials are used, along with partial factors on the tunnel lining materials and on the ‘actions or the effects of actions’ (i.e. the loads in the tunnel lining). This should make failure very unlikely, which is a good thing. However, the initial prediction we make, that is then factored, is clouded in more uncertainty than we care to admit.

The design of tunnel linings is based on trying to predict the stresses that will be present in the structure throughout its life, from construction to the design life of 120 years. We usually attempt to predict these lining stresses using analytical solutions or numerical models. A methodology often followed when modelling in 2D is to vary the amount of relaxation allowed prior to lining installation until the ground movements approximately match the expected ground movements. This could be called a ‘semi-empirical approach’, in that it is based partly on a calibration to the expected reality and is partly deduction from theory and geotechnical laboratory tests. However, Jones (2012) showed that very similar ground movements could be obtained using different constitutive models for the ground, while giving very different values of lining stresses. This is shown in Figure 1, where different constitutive models give very different values of lining stress at the same value of ground deformation. For example, to achieve a target volume loss of 1%, the lining stress could be anywhere between 24% and 60% of full overburden pressure[1], depending on the constitutive model used.

[1] ‘Full overburden pressure’ is the initial in situ vertical total stress at the axis level of the tunnel.

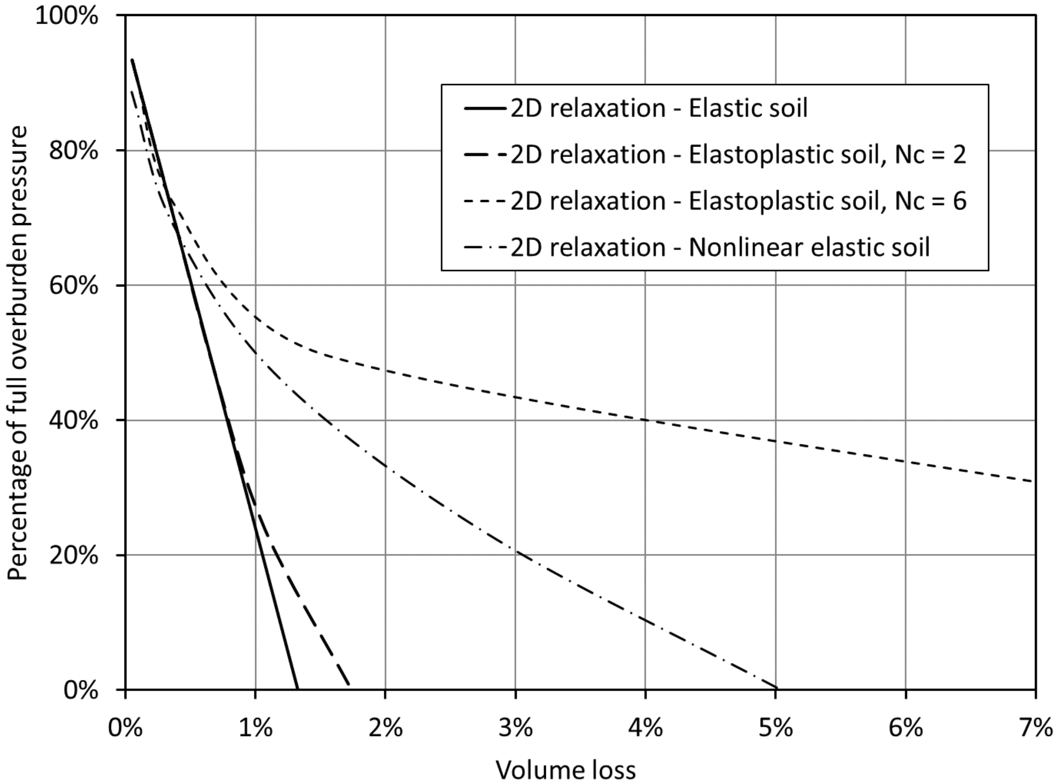


Figure 1: Relationship between lining stress (expressed as percentage of full overburden pressure) and ground deformation (expressed as volume loss), from Jones (2012).

Another area of uncertainty is the role of groundwater in low permeability soils that can be said to exhibit ‘undrained’ behaviour in the short-term followed by ‘drained’ behaviour in the long-term. During excavation, the ground around the tunnel is unloaded in the radial direction, but in low permeability soils the pore water cannot move quickly enough during this timeframe and so changes of mean total stress are experienced by the soil as changes in pore pressure. These excess pore pressures (note they are ‘excess’ to some long-term hydrostatic equilibrium or steady state flow, not excess to the initial in situ pore pressure) will dissipate over time, causing changes in the effective stress (the ‘grain-to-grain’ stresses in the soil) and volume changes in the soil (swelling or shrinkage). In addition, shear stresses in the soil may cause contraction or dilation, depending on how overconsolidated the soil is. Overconsolidated stiff clays tend to dilate when sheared (generating negative excess pore pressures), and normally consolidated or lightly consolidated soft clays tend to contract when sheared (generating positive excess pore pressures).

It used to be assumed that in the long-term the initial in situ stresses in the ground would come to act on the tunnel lining hydrostatically, and this seemed to be supported by stress measurements in tunnels by Skempton (1943), which showed full overburden pressure acting on the tunnel lining only 2 weeks after construction, and stress measurements made in tunnels up to 50 years after construction by Ward & Chaplin (1965) and Ward & Thomas (1965), from which they concluded that the full overburden pressure would always eventually come to act on the tunnel lining. However, since then stress measurements by Belshaw & Palmer (1978), Bonapace (1997), Barratt et al. (1994), Muir Wood (1969) and Bowers & Redgers (1996) have shown that lining stresses rarely exceed 70% of the full overburden pressure in the medium- or long-term. The measurements that correspond to stiff overconsolidated clays are shown in Figure 2.

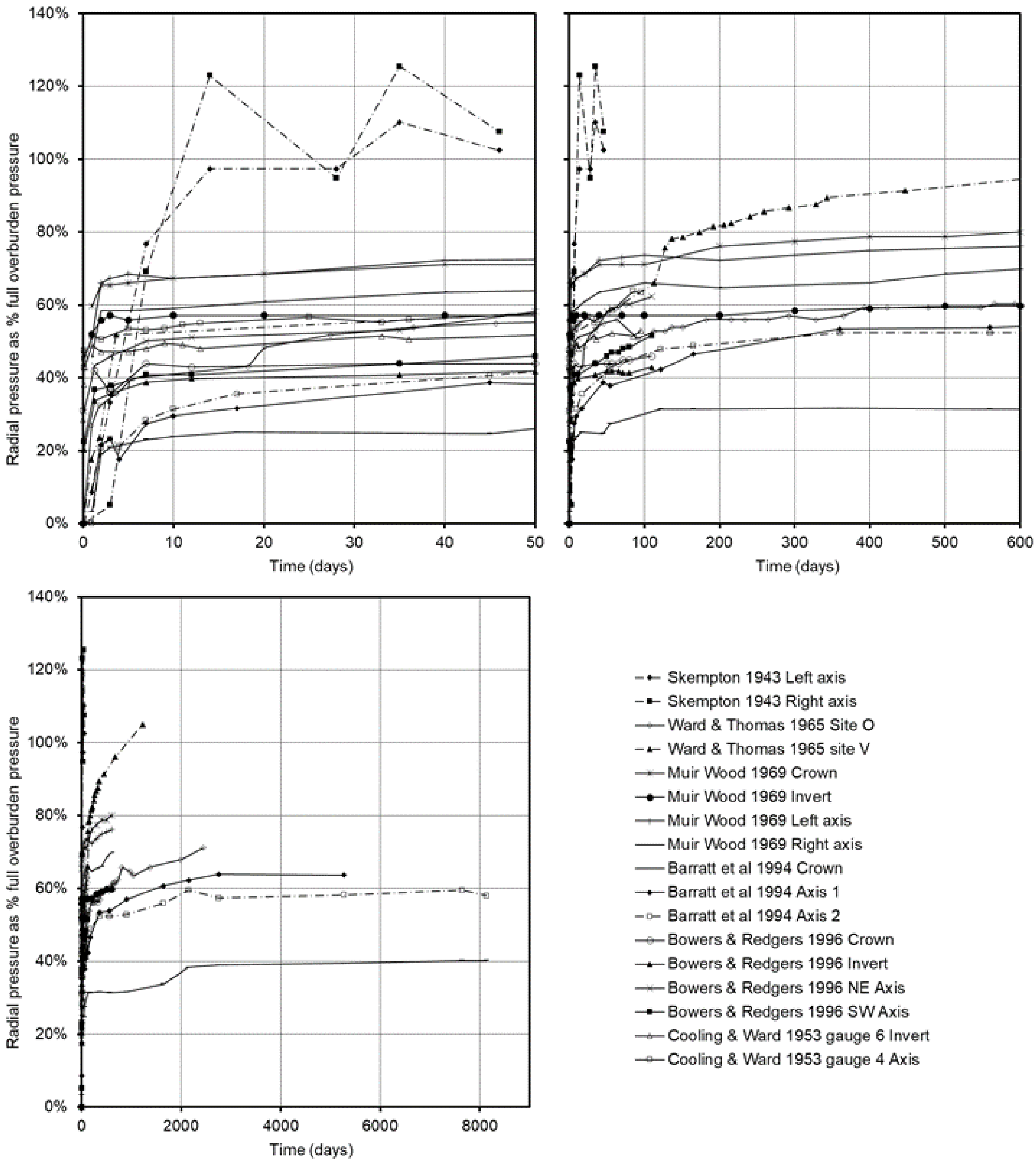


Figure 2: Historic stress measurements in segmental tunnel linings in stiff clay, shown on 3 timescales – the first 50 days, the first 600 days, and 9000 days (note a log-time chart was deliberately not used as it gives a false impression of how stresses change in the long-term).

Another area of uncertainty is not just the final long-term value of stress in the tunnel lining, but the period of time over which this develops. Long-term measurements of load in the Jubilee Line tunnels at Regent's Park in London over 19.5 years by Barratt et al. (1994) showed a gradual increase in load over time, though the majority of the increase occurred in the first 3-4 years (Figure 2). They found similar increases in load in an Oxford sewer in overconsolidated clay. Similarly, measurements by Bonapace (1997) showed radial stresses increasing from a mean of 250 kPa at 3 months to a mean of 350 kPa at 12 months. In contrast, long-term stress measurements in the Heathrow Cargo tunnel in London Clay by Muir Wood (1969 – see Figure 2) showed a less than 10% increase in the load between 2 days and 600 days. Similar measurements in an instrumented ring by Bowers & Redgers (1996 – Figure 2) over 100 days showed a less than 30% increase in load.

Jones (2005) discussed the possible reasons for these very different behaviours. It may be that the degree of unloading of the ground is critical to the subsequent behaviour, i.e. that if there are large ground deformations during construction then the short-term load will be lower but there will be a gradual increase over the long-term as excess pore pressures dissipate, and if the ground deformations during construction are kept very small then the short-term load may be higher but there will be very little increase in the long-term. Another complicating factor is whether the tunnel lining acts as a drain or is impermeable and what effect this has on the long-term pore pressure distribution.

In summary, there are many unanswered questions remaining about how loads come onto tunnel linings in the short and long-term that have a negative impact on design. This is particularly true of sprayed concrete linings, for which there are very few data. This paper will present selected results from a detailed case study of stresses and strains in a shotcrete primary lining in London Clay over almost 20 years and will seek to answer some of these questions.

INTRODUCTION TO THE CASE STUDY

The main tunnels at Heathrow Express Terminal 4 station were constructed between May 1994 and November 1996. In order to confirm the adequacy of design, particularly of the sprayed concrete primary lining, a considerable array of instrumentation was installed and monitored during construction. In previous papers the movements ahead of the advancing concourse tunnel (van der Berg et al., 2003), and the in-tunnel displacements and surface settlements (Clayton et al., 2006) were presented. An earlier paper by Clayton et al. (2002) studied the performance of pressure cells in sprayed concrete linings, focussing mainly on laboratory tests and numerical modelling to improve understanding of cell action factor, temperature sensitivity and installation effects, but did not present a complete set of field data. The radial pressure cell data up to 8 years were previously published in Jones (2005).

The layout, geology, construction sequence and construction details of the concourse and platform tunnels were described in van der Berg et al. (2003) and Clayton et al. (2006), but important details will be replicated here. The layout of the Heathrow Express Terminal 4 station is shown in Figure 3. It consists of two platform tunnels with a central concourse tunnel at the north-eastern end. These tunnels are connected by a series of cross-passages and connected to the north and south ventilation tunnels at each end. The concourse tunnel was constructed after the platform tunnels but before the crosspassages. The downline ventilation tunnel, which connects the north ventilation tunnel to the downline platform tunnel, underpassed the concourse tunnel while the concourse tunnel was itself being constructed.

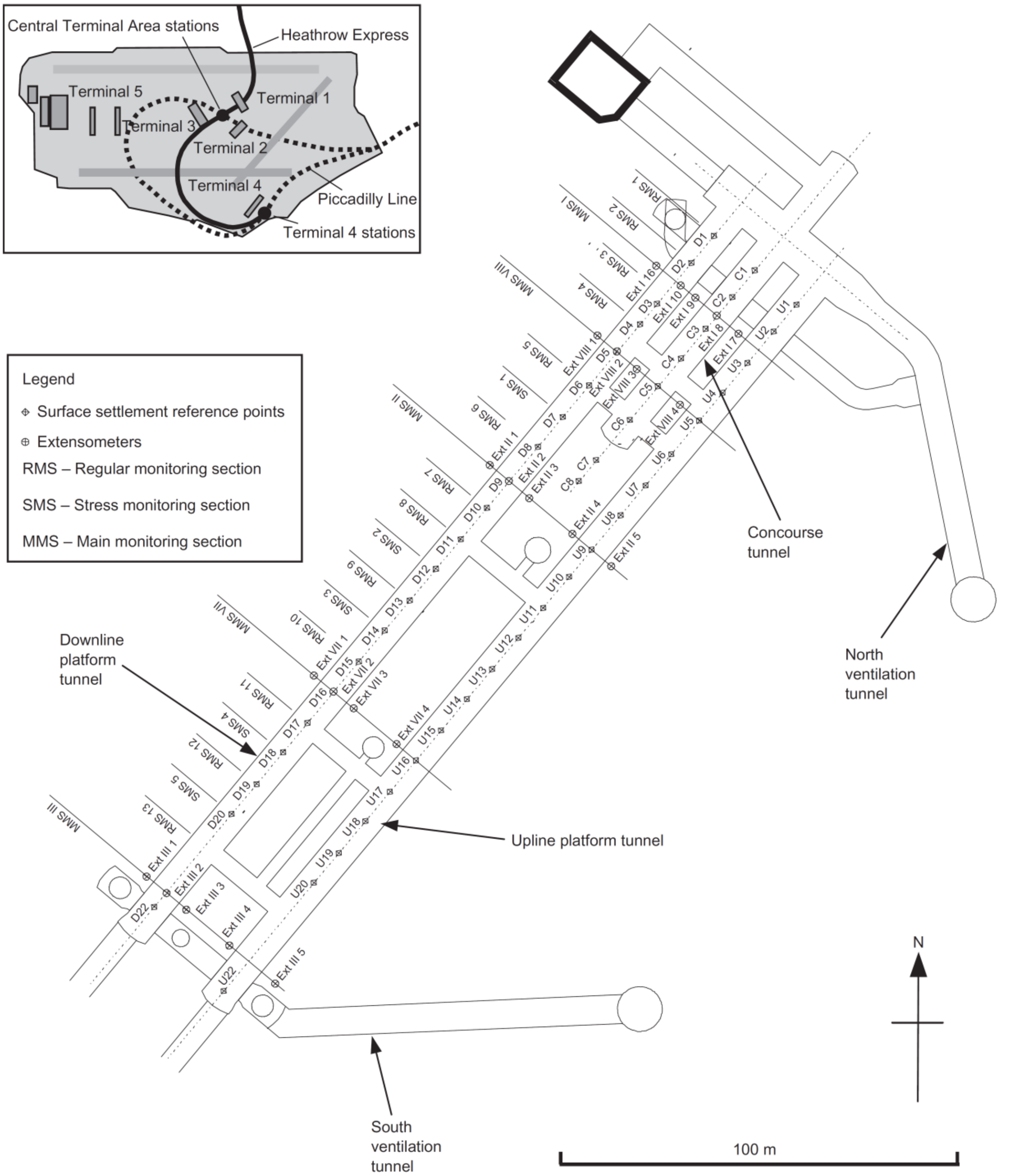


Figure 3: Plan of tunnels at Heathrow Express Terminal 4 station, showing location of concourse tunnel and layout of monitoring points and instruments (from van der Berg et al., 2003).

The platform tunnels were over 220 m long with a cross-sectional area of 62 m^2, and the concourse tunnel was 64 m long with a cross-sectional area of 49 m^2. A typical cross-section of the concourse and platform tunnels is shown in Figure 4, which also shows the surface level and geological strata. The concourse tunnel axis is at a depth of approximately 17.2 m below ground level and the tunnel is entirely within the London Clay. Piezometers across the site and at different depths indicated a piezometric level in the Terrace Gravels at approximately ground level with a hydrostatic distribution from there down to the basal beds of the London Clay, well below the tunnel horizon (van der Berg et al., 2003). The centreline spacing between the concourse tunnel and the platform tunnels was 13.5 m.

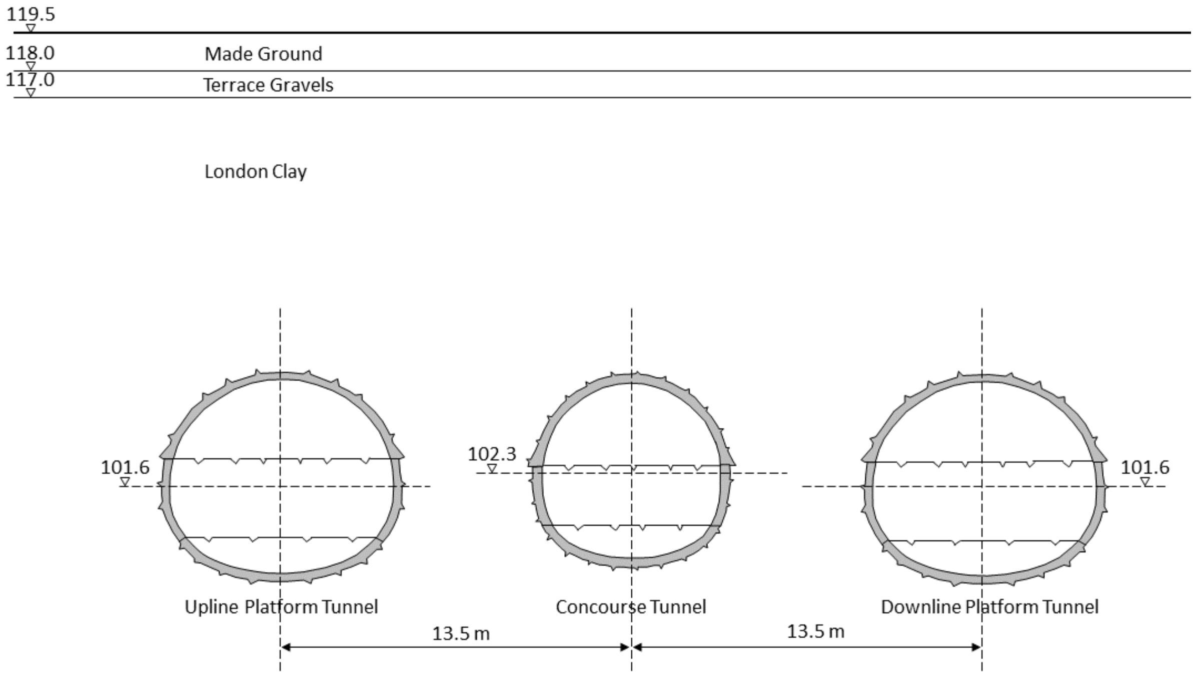


Figure 4: Cross-section of concourse and platform tunnels at MMS I or MMS VIII, looking south. Levels are in m above tunnel datum, which was set 100 m below Ordnance Datum.

The construction sequence for the concourse tunnel used a top heading, bench, top heading, bench, double-invert sequence. The invert was closed a maximum five rounds from the face. The construction sequence is schematically illustrated in Figure 5. The advance length varied from 0.8 m to 1.2 m depending on ground conditions and design requirements, including the proximity of sensitive structures. The primary support for the concourse tunnel consisted of 350 mm of shotcrete, reinforced with two layers of welded wire mesh (8 mm diameter at 150 mm centres) and full-section lattice girders 'Type 110 ROM E3'. The exposed ground was supported by a 50–100 mm shotcrete sealing layer applied immediately after each advance. The 350 mm total thickness included the sealing layer.

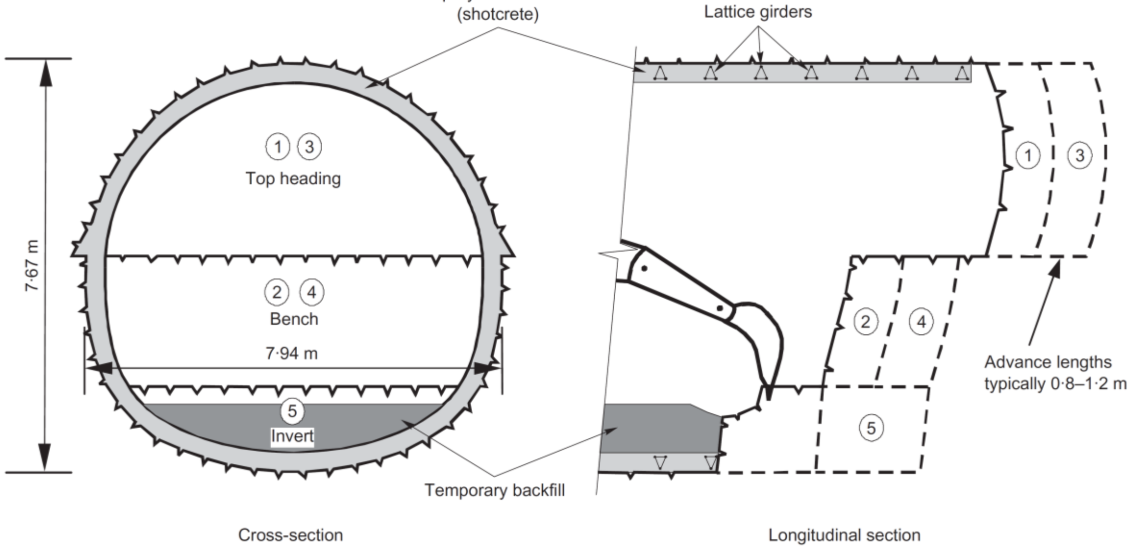


Figure 5: Concourse tunnel construction sequence (from van der Berg et al., 2003).

The typical construction procedure was as follows. Excavation was carried out using a track-mounted excavator. Areas where the radial pressure cells were to be placed were prepared and covered with timber (van der Berg et al., 1998). The nozzleman then applied a sealing layer of shotcrete, typically between 50 mm and 100 mm thick, on all the exposed London Clay surfaces, including the face. Following the application of the sealing layer, the radial pressure cells were installed against the ground on a bed of weak mortar. Then the lattice girder and first layer of mesh were installed. The first layer of shotcrete was then applied. The tangential pressure cells were then secured in their positions in the centre of the lining thickness with their longest dimension (200 mm) in the longitudinal tunnel direction and 100 mm dimension in the radial orientation. A second layer of shotcrete was then applied. Then the second layer of mesh was fixed and finally, a third layer of shotcrete was applied. Some excavated material was used to fill the invert of the tunnel, which served as a working platform and access when excavating the top heading and bench. Strain gauges were welded to approximately 500 mm long 8 mm diameter reinforcement bars and the bars were then fixed to either the outer or the inner layer of mesh in the tangential orientation.

This paper will focus on the long-term readings from pressure cells and strain gauges installed in Main Monitoring Section I (MMS I) and Main Monitoring Section VIII (MMS VIII) of the concourse tunnel. The locations of these sections were shown in Figure 3. At each section, 12 tangential pressure cells and 12 radial pressure cells were installed. The locations are shown in Figure 6.

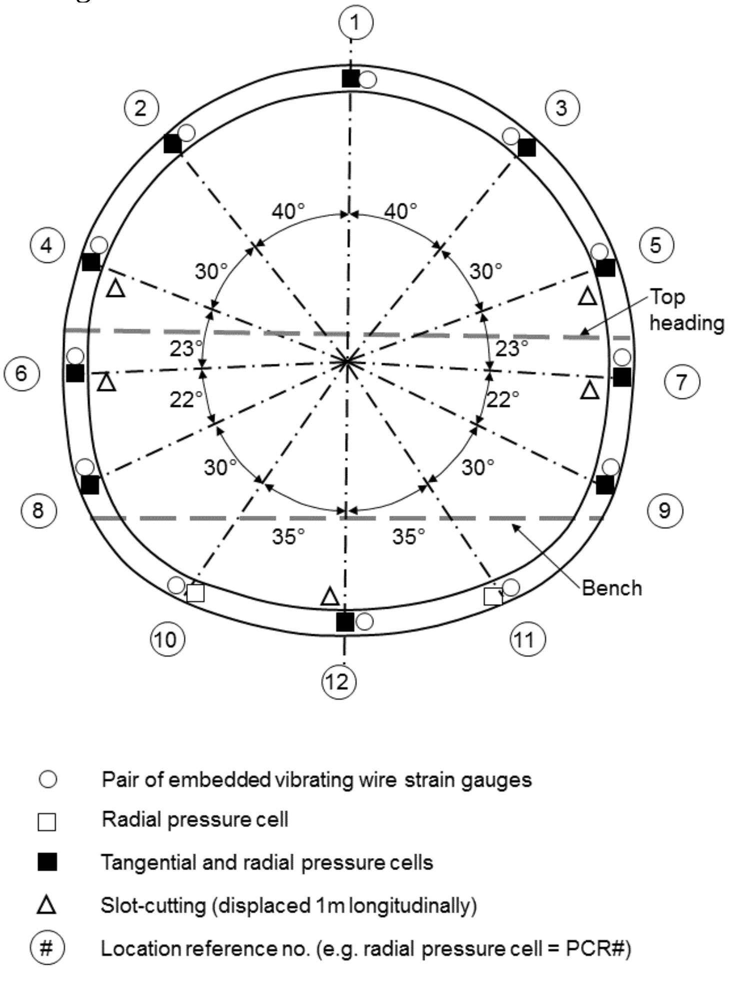


Figure 6: Section schematically showing locations of pressure cells and strain gauges embedded in the sprayed concrete primary lining at MMS I and MMS VIII.

RESULTS FROM RADIAL PRESSURE CELLS

Selected results from the MMS I and MMS VIII radial pressure cells will be presented. The readings are compared to the in situ stress normal to the lining calculated at the positions of the radial pressure cells, based on a bulk unit weight for the Made Ground, Terrace Gravel and London Clay of 19.5 kN/m^3 and a coefficient of earth pressure at rest (K_0) of 1.5 (Powell et al., 1997).

MMS I radial stresses

Figure 7 shows the average of all the radial pressure cells in MMS I as a percentage of the average in situ radial stresses. Also shown are the average temperatures measured by the thermistors attached to the pressure cells. Casting the invert section of the secondary lining caused a transient increase in temperature and an increase in radial pressure at the invert. Underpassing by the downline vent tunnel, only 5 m below the concourse tunnel, caused decreases in radial pressure, particularly at the bench and invert. Shortly afterwards the radial pressure cells in the crown stopped functioning, probably caused by damage to the cables when the upper part of the secondary lining was cast.

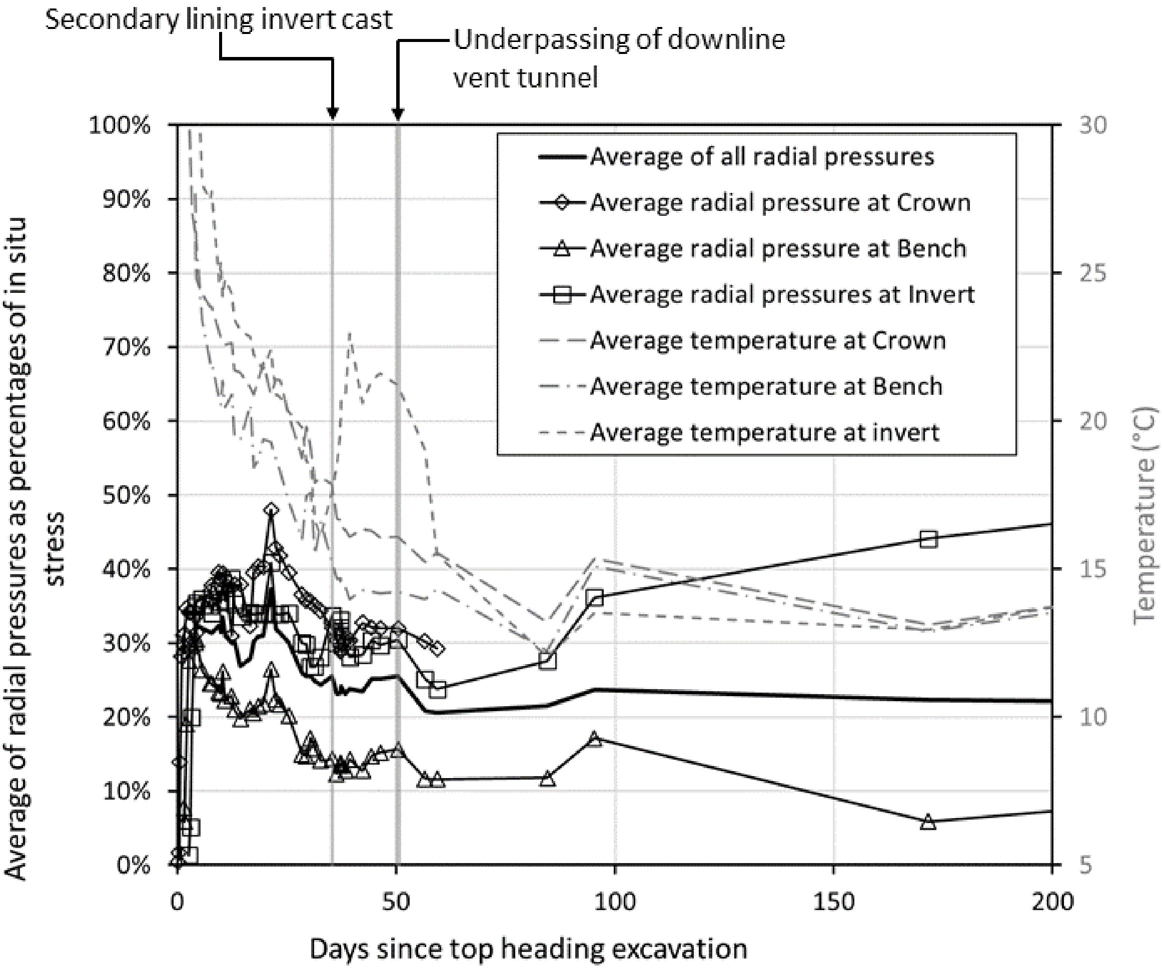


Figure 7: MMS I - average of radial pressures as percentages of in situ radial stresses at Crown (PCR1-5), Bench (PCR6-9) and Invert (PCR10-12), and average temperatures, from invert closure to 200 days.

It is important to remember that this was not a greenfield situation, and the radial pressures were influenced by the adjacent north vent tunnel enlargement, downline vent tunnel and the platform tunnels, as well as at times by compensation grouting. However, the long-term trends may still provide valid insights into the behaviour of sprayed concrete tunnels. The MMS I radial pressures over 18.6 years are shown in Figure 8. The two most recent readings of average radial pressures in the crown were from position 3 only. The radial cell at position 3 had not responded for many years and so these values should be treated with some caution.

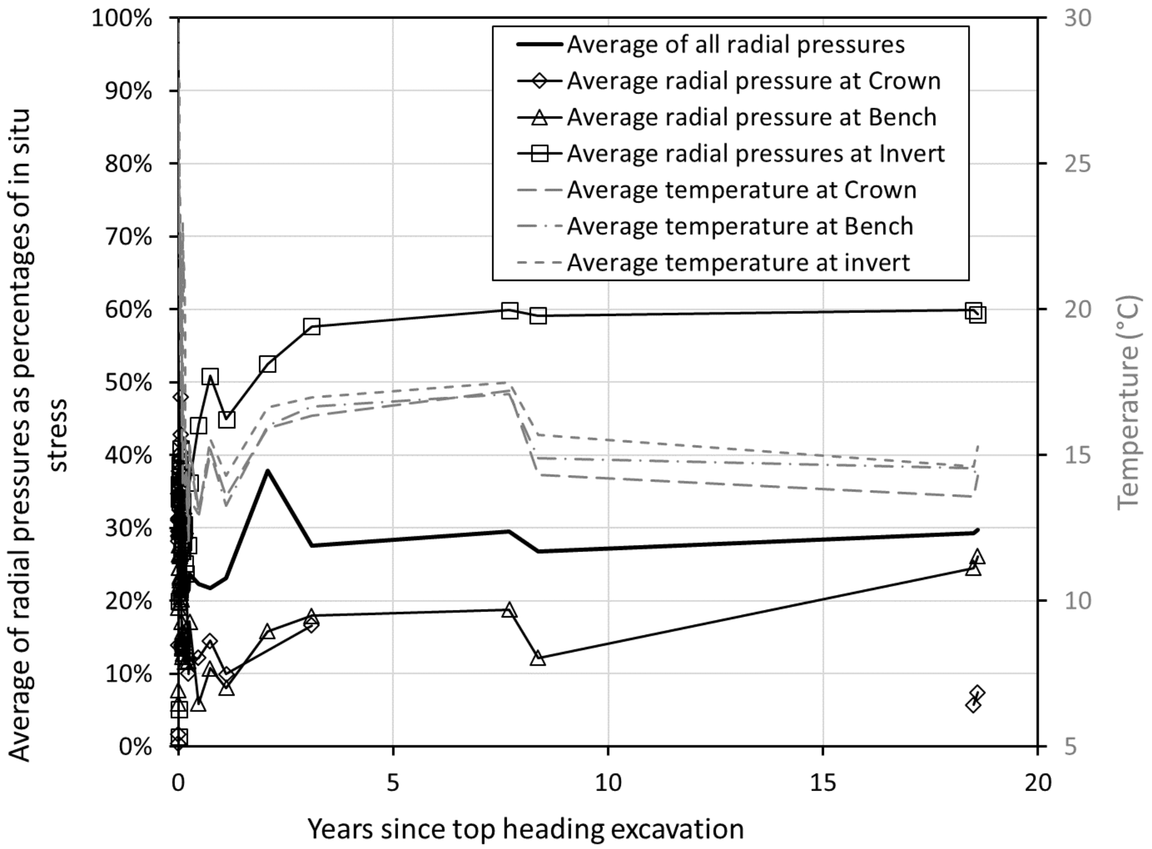


Figure 8: MMS I - average of radial pressures as percentages of in situ radial stresses at Crown (PCR1-5), Bench (PCR6-9) and Invert (PCR10-12), and average temperatures, from invert closure to 18.6 years.

The fluctuation of average radial pressure between readings at 2.1 and 3.1 years is because readings could not be obtained from all the pressure cells each time and so the average was affected. Apart from this blip, we can see a strong dependence of radial pressure on temperature, which is particularly noticeable between the readings at 7.7 and 8.4 years. This has been estimated to be of the order of 7 kPa/°C and is due to thermal expansion and contraction of the tunnel lining, increasing and decreasing the radial pressure between the ground and the lining. This was referred to by Jones (2005) as 'ground reaction temperature sensitivity', to differentiate it from other kinds of temperature sensitivity that affect radial and tangential pressure cells. Therefore, a tunnel lining does not have a single state of long-term equilibrium, but an equilibrium that changes as the temperature in the tunnel varies.

MMS VIII radial stresses

The survivability of the MMS VIII radial pressure cells was much better than for MMS I, and so there is more detail available in the results. The readings over the first 100 days are shown in Figure 9. Again, casting the invert section of the secondary lining caused an increase in temperature and radial pressure at the invert. The increase in radial pressure was partly caused by temperature and partly caused by added weight.

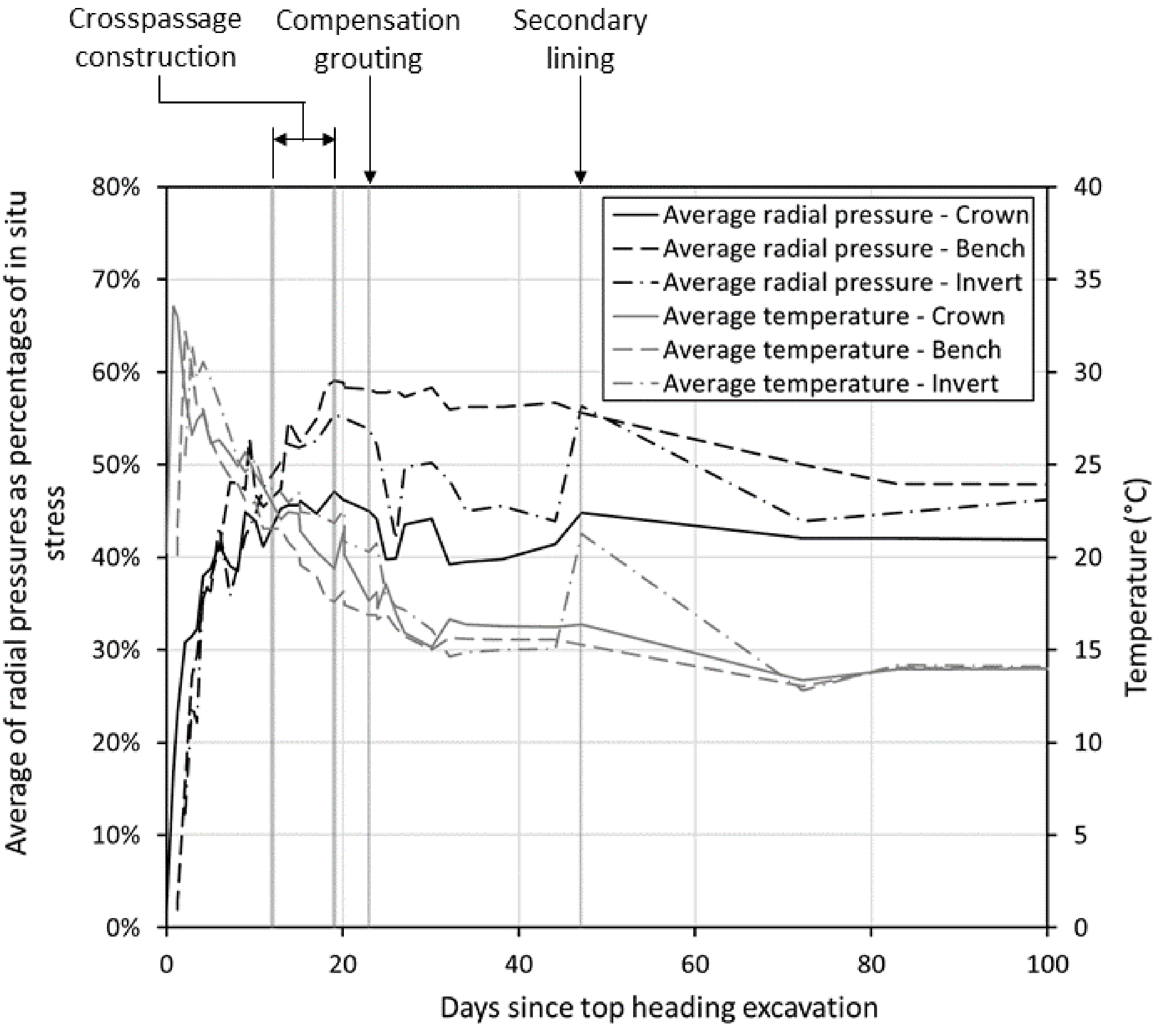


Figure 9: MMS VIII – average of radial pressures as percentages of in situ radial stresses at Crown (PCR1-5), Bench (PCR6-9) and Invert (PCR10-12), and corresponding average temperatures.

Figure 10 shows the long-term trends of average radial pressures at the crown, bench and invert at MMS VIII. Although there was a gradually slowing increase of radial pressures, radial pressures over this timescale were dominated by temperature effects, with a magnitude of around 7 kPa/°C. There will be no final equilibrium value of radial pressure, as it will constantly be changing as temperature changes. However, the measured radial pressures were between 40 % and 77 % of the in situ radial stress at 18.6 years, with an average of 59 %, at an average temperature of 14.2°C.

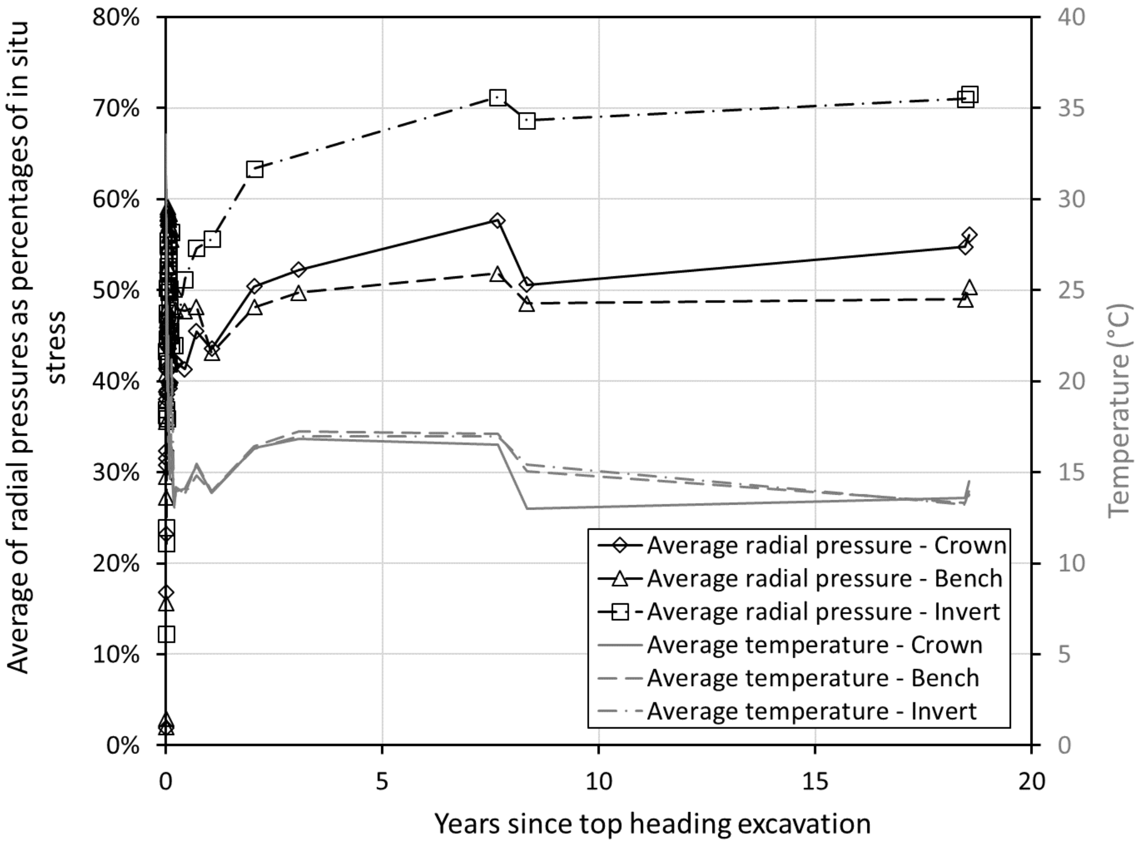


Figure 10: MMS VIII - average of radial pressures as percentages of in situ radial stresses, and average temperatures, at Crown, Bench and Invert, from construction to 18.6 years.

The invert radial pressures in Figure 10 are higher than at the crown and bench, and this was also true in MMS I (Figure 8). The vertical pressure acting on the invert should be in equilibrium with the vertical pressure on the crown plus the weight of the tunnel lining plus any shear stress on the sides of the tunnel, so it is reasonable for the invert radial pressures to be higher than the crown radial pressures. The reason why the bench radial pressures appear to be lower is that pressures are presented in the graphs as a percentage of the in situ total stress at the location and orientation of the pressure cells. The horizontal in situ total stress is higher than the vertical because K_0 has been taken as equal to 1.5. It is known from large numbers of lining measurements by Wright (2013) , that after tunnel excavation in London Clay horizontal stresses acting on a tunnel lining are always lower than vertical stresses, and the in situ K_0 does not reassert itself.

COMPARISON WITH TANGENTIAL PRESSURE CELLS AND STRAIN GAUGES

There is insufficient space in this paper to present even the radial pressure cell results in full, let alone the tangential pressure cell and strain gauge results, which require significantly more interpretation. These will be published in detail elsewhere. However, some comparisons will be made to highlight or confirm certain aspects.

What happens in the long-term after construction activities have ceased?

The radial pressure cells indicate that there is little or no increase in radial pressure acting on the tunnel lining in the long-term. This is corroborated by the tangential pressure cells, which show negligible changes in the stress state of the primary lining in the long-term that cannot be attributed to ground reaction temperature sensitivity.

It is also corroborated by the 48 strain gauges, which showed a very small increase in strain over the first 3 years, followed by a very slight decrease in strain over the subsequent 15-16 years. An example is shown in Figure 11. It should be noted that strain gauges will not notice changes in temperature, as their coefficient of thermal expansion is very similar to that of concrete, so they do not register the effect of ground reaction temperature sensitivity.

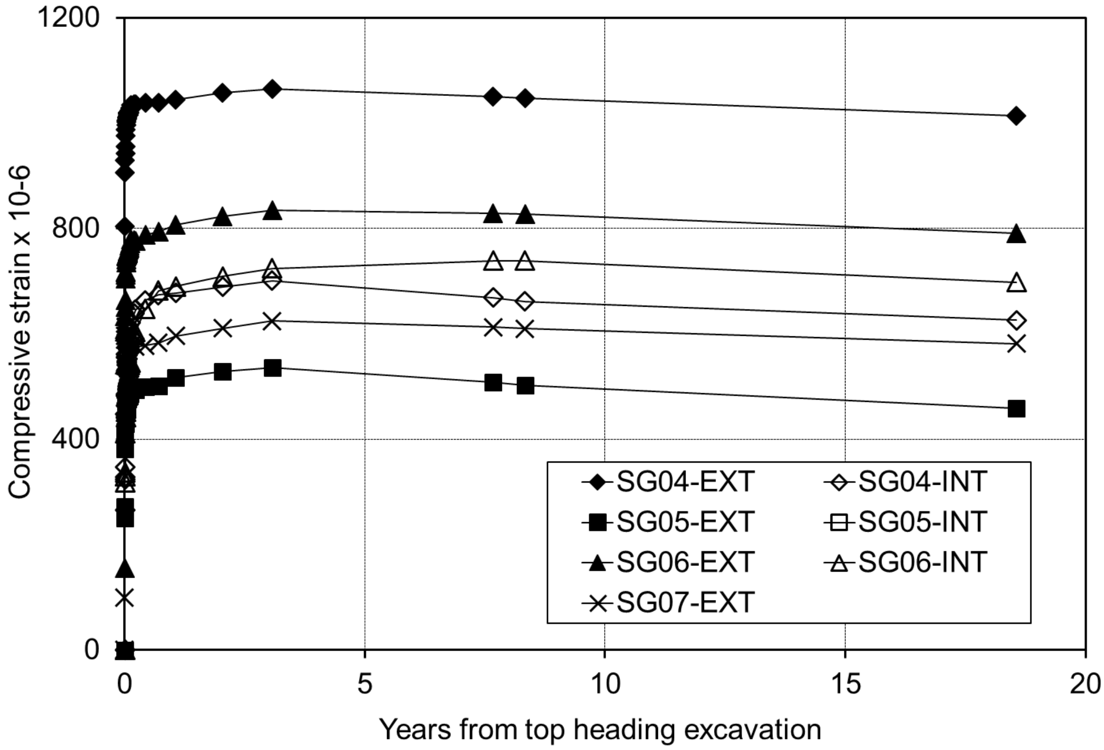


Figure 11: Extrados (-EXT) and intrados (-INT) strain gauges at positions 4-7 in MMS VIII.

One possible explanation for the slight increase in compressive strain between 1 and 3 years is a slight increase in radial pressure during this period and ongoing compressive creep of the lining. It seems unlikely that much shrinkage would occur after installation of the waterproof membrane and after the lining is already 1 year old. The subsequent slight decrease in the longer term may be due to water penetration, which will be discussed in the following section.

Is the groundwater pressure acting on the secondary lining in the long-term?

In the original design for this tunnel, the primary lining was assumed to be permeable and temporary, and so a sheet waterproof membrane was installed, followed by a secondary lining to support the water pressure and the ground loads. Unfortunately, no instrumentation was installed in the secondary lining, so it is difficult to know what its stress state actually is.

In the short-term, ground and water loads are applied to the outside of the primary lining. Then the waterproof membrane and secondary lining are installed, and there are 2 possible scenarios:

- If the primary lining is watertight, the situation will remain unchanged. Any further increments in ground or water load after secondary lining installation may be shared with the secondary lining, but this would need to be a significant increment to overcome the differential shrinkage and any compliance in the geotextile fleece and waterproof membrane. In fact, the increment of stress needed to make contact with the secondary lining is so large (perhaps > 10 MPa) that it is theoretically impossible for a tunnel at this depth.

- If the primary lining is permeable, groundwater will penetrate to the back of the waterproof membrane, flowing around the outside of it and applying a hydrostatic pressure. This water pressure can only be supported by the secondary lining.

Given the results of long-term structural monitoring of the primary lining, we know it has not substantially degraded or experienced structural failure. Therefore, there are only two possibilities: either water has penetrated to the back of the waterproof membrane, or it has not. The ground loads are still supported by the primary lining, but the water load may be supported by either the primary or the secondary lining.

Radial pressure cells measure *total* stresses, so if the primary lining were saturated and the water pressure was acting on the secondary lining, then they would not notice any change. This is illustrated in Figure 12.

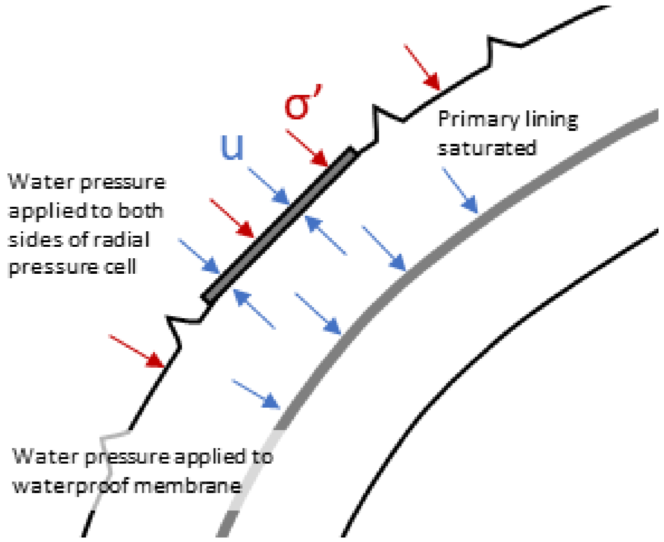


Figure 12: Effect of primary lining saturation on radial pressure cell

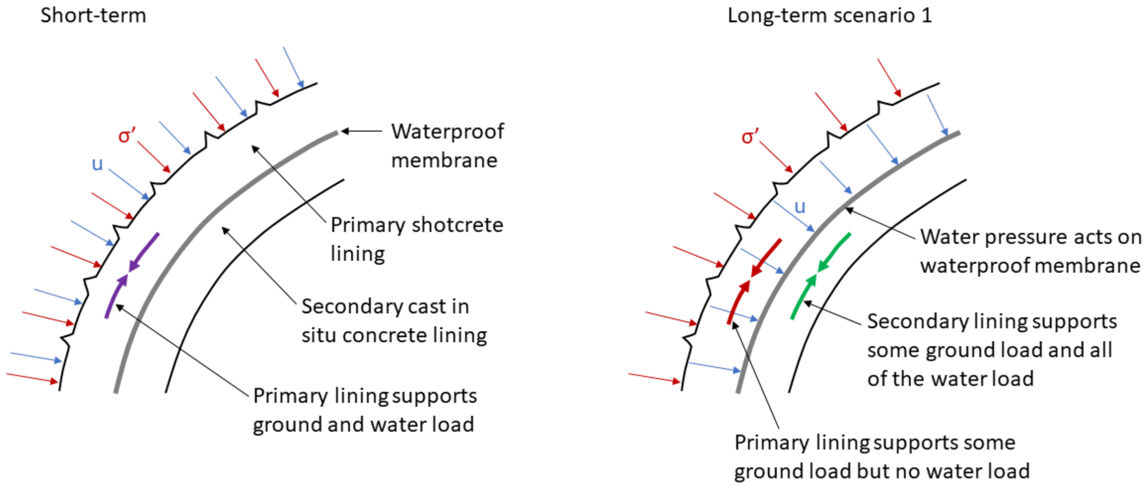
If the water did penetrate the primary lining to act on the waterproof membrane, then this would result in a reduction in tangential compressive stress in the primary lining as the water pressure on its extrados and intrados balanced out. But if the primary lining were saturated, then the water pressure would still be applied to the tangential pressure cells and they would not register a change in stress state.

What would happen to the strain gauges measuring tangential strains in the primary lining? Unloading of the concrete as water pressure is applied to both the extrados and the intrados should result in a reduction in compressive strain as the primary lining stress is based only on the applied radial effective stress from the ground. This may be the cause of the gradual reduction in compressive strain between 3.1 and 18.6 years as shown in Figure 11, which was on average 44 x 10^{-6} for MMS I and 34 x 10^{-6} for MMS VIII. This reduction may have other causes (e.g. chemical changes in the concrete causing swelling) but given the lack of response of the pressure cells, the most likely explanation is penetration of water.

It is impossible to say for sure in this case whether the water load is being taken by the primary or the secondary lining, but it seems likely that water has penetrated, and the water pressure is acting on the secondary lining. It is recommended that instrumentation is installed in a secondary lining (or perhaps piezometers installed behind the waterproof membrane) of future tunnels to investigate this.

Is there sharing of ground load between the primary and secondary linings?

Load sharing of ground load between the primary and secondary linings can be partial (scenario 1), complete transfer of all loads to the secondary lining (scenario 2), or no transfer to the secondary lining (scenario 3). This is illustrated in Figure 13.



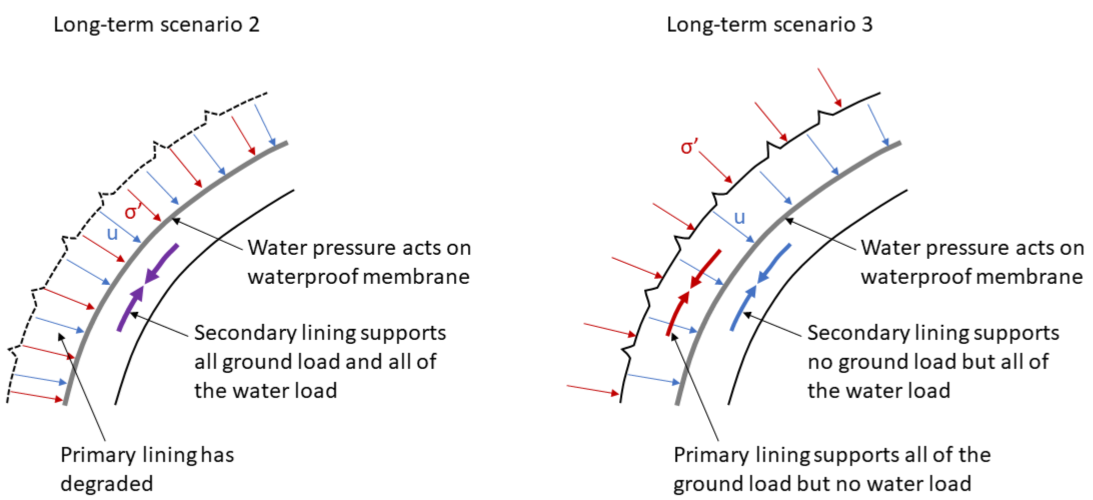


Figure 13: Different scenarios for transfer of ground and water loads to primary and secondary linings

As discussed previously, partial load sharing (scenario 1) is unlikely, as for this case study it would require failure of the primary lining, in which case one would assume most if not all the load would be transferred (scenario 2). The current situation in the Heathrow Terminal 4 concourse tunnel is probably where the water load is applied to the secondary lining, but the ground loads are supported by the primary lining (scenario 3), though it is possible that the short-term situation where both ground and water loads are applied to the primary lining is still active.

CONCLUSIONS

Long-term structural monitoring of the Terminal 4 Station concourse tunnel indicates that ground loads on shotcrete primary linings will stabilise at a value well below full overburden pressure, as has been found by several other previous studies on tunnels in London Clay. The reasons why some tunnels experience loads up to full overburden pressure are not well-understood but could be due to the amount of ground deformation allowed during construction, and whether the tunnel lining is impermeable or acts as a drain on the ground around the tunnel.

The reasons why some tunnels achieve their long-term stable load within a few weeks or months of construction, but some take up to 3-4 years or more are also not well understood. This may be to do with how long it takes for pore pressures to reach equilibrium or a steady state.

Tunnels do not have a single long-term state of equilibrium. As temperature in the tunnel increases and decreases, the tunnel lining expands and contracts against the ground, increasing and decreasing the radial pressure at the ground-lining interface. This has been termed 'ground reaction temperature sensitivity'. In some cases, for instance for London Underground tunnels where during tunnel operation the temperature in the lining and the surrounding ground gradually increases over many years, one may expect a gradual long-term increase in lining stress as a result.

If the groundwater has permeated through the primary lining and the water pressure is acting hydrostatically on the outside of the waterproof membrane, this will not be registered by radial or tangential pressure cells because they measure total stresses.

It is likely that for this case study, by about 3 years after construction the groundwater had permeated through the primary lining and the secondary lining is now supporting the groundwater load. On the other hand, it is very unlikely that the secondary lining is not supporting any of the ground load (the effective stress).

Stresses and strains in tunnel linings are very rarely measured. To improve design predictions for future tunnels, and to better understand the tunnels we have already built, we need this kind of data.

REFERENCES

Barratt, D. A., O'Reilly, M. P. & Temporal, J. (1994). Long-term measurements of loads on tunnel linings in overconsolidated clay. In *Proc. Tunnelling '94*, pp.469-481. London: IMM.

Belshaw, D. J. & Palmer, J. H. L. (1978). Results of a program of instrumentation involving a precast segmented concrete-lined tunnel in clay. *Can. Geotech. J.* **15**, 573-583.

Bonapace, P. (1997). Evaluation of stress measurements in NATM tunnels at the Jubilee Line Extension Project. In *Proc. of Tunnels for People* (eds. Golser, Hinkel & Schubert), pp.325-330. Rotterdam: Balkema.

Bowers, K. H. & Redgers, J. D. (1996). Discussion: Observations of lining load in a London Clay tunnel. In *Proc. Int. Symp. on Geotechnical Aspects of Underground Construction in Soft Ground* (eds. Mair & Taylor), London, UK. Rotterdam: Balkema.

Clayton, C. R. I., van der Berg, J. P., Heymann, G., Bica, A. V. D. & Hope, V. S. (2002). The performance of pressure cells for sprayed concrete tunnel linings. *Géotechique* **52**, No.2, 107-115.

Clayton, C. R. I., van der Berg, J. P. & Thomas, A. H. (2006). Monitoring and displacements at Heathrow Express Terminal 4 station tunnels. *Géotechnique* **56**, No.5, 323-334.

Jones, B. D. (2005). Measurements of ground pressure on sprayed concrete tunnels using radial pressure cells. *Proc. Underground Construction 2005*, London, UK. London: Brintex.

Jones, B. D. (2012). A loaded question. *Tunnelling Journal*, April-May issue, 26-28.

Muir Wood, A. M. (1969). Written contribution, plenary session 4. In *Proc. 7th Int. Conf. on Soil Mechanics and Foundation Engineering*, Vol.3, Mexico, pp.363-365.

Skempton, A.W. (1943). Discussion of Groves, G.L., Tunnel linings with special reference to a new form of reinforced concrete lining. Journal of the Institution of Civil Engineers, Vol.20, No.5, March, 53-56.

van der Berg, J. P., Clayton, C. R. I. & Hope, V. S. (1998). An evaluation of the role of monitoring during the construction of shallow NATM tunnels in urban areas. *Proc. North American Tunnelling '98* (ed. L. Ozdemir), Newport Beach, California, USA, pp.251-257.

van der Berg, J. P., Clayton, C. R. I. & Powell, D. B. (2003). Displacements ahead of an advancing NATM tunnel in the London clay. *Géotechnique* **53**, No.9, 767-784.

Ward, W. H. & Chaplin, T. K. (1965). Existing stresses in several old London Underground tunnels. In *Proc. 4th Int. Conf. on Soil Mechanics and Foundation Engineering*, Montreal, Vol.2, pp.432-436.

Ward, W. H. & Thomas, H. S. H. (1965). The development of earth loading and deformation in tunnel linings in London Clay. In *Proc. 6th Int. Conf. on Soil Mechanics and Foundation Engineering*, Toronto, Vol.2, pp.432-436.

Wright, P. J. (2013). Validation of soil parameters for deep tube tunnel assessment. *Proc. ICE Geotechnical Engrg* **166**, No.1, 18-30.