ABSTRACT: This paper discusses the importance of stress determination in the context of a holistic risk management process, and how these stresses may be obtained both by direct stress measurement and by back-calculation of lining displacements. The relative reliability and accuracy of these methods will also be discussed.

RÉSUMÉ: Cet article examine l'importance de mesures de contrainte dans le contexte d'un processus global de gestion des risques, et comment on peut déterminer ces mesures directement et par calculs en arrière des déplacements. La fiabilité et précision de ces méthodes seront aussi examinées.

1 - WHY IS STRESS MEASUREMENT IMPORTANT?

Design methods for sprayed concrete tunnels are based on assumptions and simplifications that make the design at best semi-empirical (HSE, 1996). Even if the material behaviour of the ground and the sprayed concrete are well known and a 3D analysis is performed, there are still uncertainties about the true ground mass behaviour and the variability of sprayed concrete to consider. This partially explains the reliance of the successful tunneller on observation (Muir Wood, 2003) but is equally because a geotechnical design cannot completely cover "every unfavourable situation that might be disclosed by the observations" (Peck, 1969a). Muir Wood (2003) puts great emphasis on "overall holistic surveillance": that a tunnel project should be approached as a system with a continuous risk management process through all phases of design and construction.

This risk management process was outlined by Powell & Beveridge (1998) and emphasises the interdependency of prediction and verification with observation and modification. Design must be managed through several phases, from conceptual and detailed design, through construction and into operation. During conceptual and detailed design, the emphasis is on prediction. At the same time, hazards are identified, risks are assessed and the management procedures to control risk during construction are formulated. During construction, the emphasis moves to verification of the design predictions and modification of the design based on observation (the observational method). Central to the direction of this process are the management procedures, which ensure that risks are controlled and new hazards identified. These management procedures will include quality assurance and regular design review meetings.

This holistic approach to SCL tunnel design, based on risk management, prediction and verification, observation and modification is fundamental to good tunnelling practice and is central to the NATM philosophy (Brown, 1981). The importance of prediction, observation and modification within a risk management framework is widely recognised, but the importance of verification may be lost if there is a general expectation in some quarters that designs should be accurate predictions.

There is an inconsistency between the way a tunnel is designed for ultimate limit state stresses and the way its safety is usually monitored during construction by measuring displacements of the tunnel lining. Since design methods generally predict stresses more accurately than displacements, and the failure criterion defined by existing codes of practice is expressed in terms of stress, it would seem more appropriate to measure stresses in a tunnel lining to verify its performance (van der Berg *et al.*, 1998). However, the simplest and most reliable measurements that can be made in a tunnel are measurements of lining displacements (Clayton *et al.*, 2000).

The interpretation of deformations has become an art in itself (Rokahr *et al.*, 2002), and fault zones ahead of the face can be predicted as well as the performance of the ground-lining system. Accumulated experience of typical deformation trends in different rock or soil types can be used to assess whether the system is achieving equilibrium (Müller-Salzburg, 1977). This empirical approach was seen as superior to static calculations with their associated assumptions of geological behaviour. It must be remembered that in the past the majority of NATM tunnels were constructed in mountainous terrain where investigation prior to tunnelling is difficult and expensive and the variability of ground conditions is high. Problems may be encountered with this almost entirely observational and empirical approach in soft ground, such as the stability of a temporary crown invert, or indeed any invert covered with backfill, which may be unknown due to the difficulties installing and reading monitoring points (Stärk *et al.*, 2002). Also, the factor of safety of the structural lining is difficult to assess with any degree of confidence.

With the increased use of numerical models in design, good agreement between predicted and measured deformations is often taken to mean that the stress in the sprayed concrete lining has also been well predicted. Differences between calculated lining stresses and measured lining stresses are frequently attributed to unrepresentative or erroneous field measurements rather than inadequacy of the model (Negro & de Queiroz, 2000). Due to the complex behaviour of sprayed concrete, especially at early age, it is not clear that this is a reliable assumption.

Since the review of monitoring data represents the "umbilical cord that connects the growing construction with its design" (Clayton *et al*., 2003), it would be desirable to measure lining stresses directly if design assumptions are to be verified and design criteria refined for future tunnelling projects (Mair, 1998). The current dearth of estimates of stress in sprayed concrete linings impacts negatively on design by introducing uncertainty.

In conclusion, in order to estimate the factor of safety of a sprayed concrete lining it is necessary to determine stress, and in order to verify that the design predictions are reasonable it is necessary to determine stress.

Since it would be preferable to evaluate stress continuously, rather than a one-off measurement such as that provided by slot-cutting, overcoring or undercoring, especially in the first 2-3 days after installation, there are only two solutions to this problem; one is to measure stress directly using pressure cells, the other is to back-calculate stress from deformation measurements. Both these methods will be examined in this paper with reference to experience of two sprayed concrete tunnel projects in London Clay. Particular attention will be paid to the accuracy and suitability of these methods.

2 – PRESSURE CELLS

There were two types of pressure cell used for this study, tangential and radial pressure cells. Tangential pressure cells are embedded in the sprayed concrete lining and measure the tangential (or hoop) stress. Radial pressure cells are placed against the ground, usually on a thin bed of mortar to ensure good contact, and sprayed concrete is sprayed over them. Radial cells measure the total stress between the ground and the lining. The pressure cells used were Geokon 4850 series, which are shown in Figure 1 below. A typical installation is shown in Figure 2.

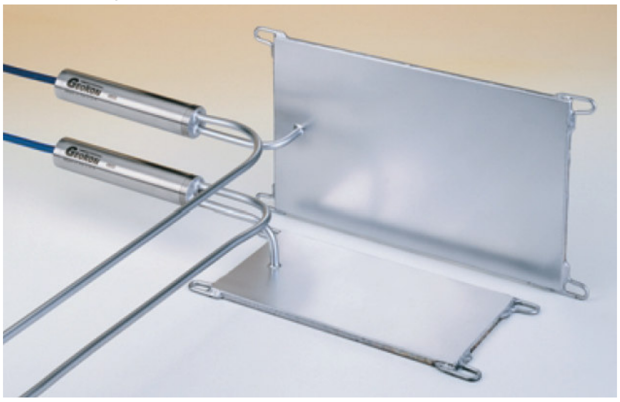

Figure 1: Geokon 4850 series radial (above) and tangential (below) pressure cells (from Geokon, 1995)

There are various factors affecting recorded pressures, which do not affect the stresses in the sprayed concrete lining. These were identified by Clayton *et al*. (2002) and by Jones *et al*. (2004), and are:

1. Installation defects.
2. Offsets due to crimping.
3. Temperature effect on the vibrating wire transducer.
4. Cell action factor.
5. Temperature sensitivity of the pressure cell embedded in the medium.

A detailed examination of the method of interpretation will be the subject of a later paper.

Figure 2: Installation of pressure cells

Installation defects can be diagnosed by the slope of the crimping curve (Clayton *et al*., 2002) or by a lack of sensitivity to temperature (Jones *et al*., 2004). Crimping will increase the pressure of the cell fluid, and provided a suitable crimping tool is used and records of the increase in pressure are kept, the resulting positive offset can be easily removed from the data. The effect of temperature on the vibrating wire transducer can also be easily removed using the manufacturer's calibration.

Cell properties affect how the pressure cell interacts with the surrounding medium. There are two recognised phenomena, cell action factor (CAF) and temperature sensitivity. CAF is the ratio of recorded pressure to the actual stress in the medium. A summary of previous studies is provided in Table 1 below. CAF is generally close to unity, and sensitivity analyses using Coutinho (1953)'s elastic solution and a 3D FE model have shown that this is true as long as the cell stiffness is greater than or roughly equal to the stiffness of the medium. This is referred to by Williams (1974) as 'hard inclusion theory'. This is usually the case for tangential pressure cells embedded in sprayed concrete or for radial pressure cells at the boundary of the sprayed concrete and the ground. The data in Table 1 suggest that the CAF is likely to be in the range 0.9-1.0, for both radial and tangential cells with a mean of 0.95. Woodford & Skipp (1976)'s large range of values of CAF found in their numerical experiment are due to the unrealistic limiting values of Young's modulus used in the parametric study. The only run which used realistic values (run J) was suspected of numerical instability. Nowadays, the use of constant strain triangle elements is treated with suspicion, particularly in the presence of large stress gradients (Cook *et al*., 2002).

Both tangential cells and radial cells are susceptible to temperature sensitivity. This occurs due to differential thermal expansion. For a tangential cell, the stainless steel casing of the pressure cell, typically 3mm thick, expands more with temperature than the cell fluid (usually hydraulic oil, 0.3mm thick) or the surrounding sprayed concrete. The sprayed concrete will then restrain the pressure cell casing's expansion by an arching action around the cell, causing a compression of the cell fluid and thus an increase in recorded pressure. Therefore, for a given pressure cell design, the temperature sensitivity is dependent on both the coefficient of thermal expansion and the stiffness of the medium in which it is embedded. The results of a parametric study using a 3D finite-element model of an embedded Geokon tangential cell are shown in Figure 3.

An estimate of the typical path taken by a tangential cell from installation to sprayed concrete maturity is shown as a broken line, beginning with a low stiffness and high coefficient of thermal expansion of the sprayed concrete (Laplante & Boulay, 1994). This would explain the slow initial response of

Experiment / Model		CAF
Load test of Glötzl radial cell at concrete-clay interface (Woodford & Skipp, 1976)		0.96
Axisymmetric elastic FE analysis of Glötzl radial cell at concrete-clay interface (Woodford & Skipp, 1976)		0.78-1.18
Air pressure calibration of Geokon radial cell (Clayton, 1995)		1.0
Load test of 2 Geokon radial cells at sprayed concrete-clay interface (Clayton *et al.*, 1995)		>0.95
Load test of ready-mix concrete panel with 2 embedded Geokon tangential pressure cells (Clayton *et al.*, 1995)		0.87-0.99
Load test of sprayed concrete panel with 2 embedded Geokon tangential pressure cells (Clayton *et al.*, 2002)		1.08
3D FE model of a Geokon tangential cell embedded in sprayed concrete (author's own)		0.89
Axisymmetric FE model of an embedded Geokon tangential cell (Clayton *et al.*, 2002)		0.95
Axisymmetric elastic solution (Coutinho, 1953)	Equivalent cell stiffness Ec = 50 GPa	1.01
	Ec = 20 GPa	0.99
	Ec = 10 GPa	0.95

Table 1 – Values of cell action factor

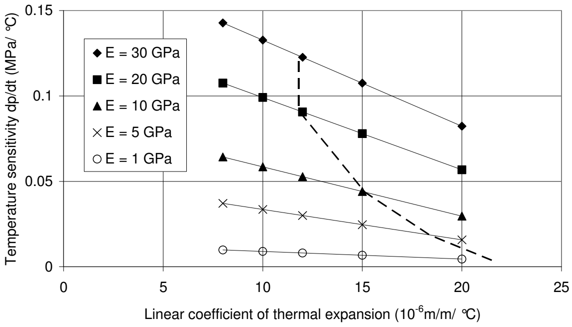

Figure 3 - Parametric study of temperature sensitivity

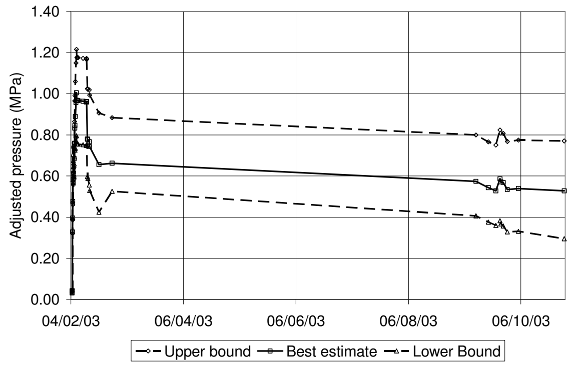

Figure 4: Confidence limits of typical tangential pressure cell readings

tangential cells to increases in temperature due to hydration.

Once the sprayed concrete has matured, the temperature sensitivity is reasonably constant. If sufficient readings are taken over a short period of time, such that changes of pressure may be assumed to be solely due to temperature, then the temperature sensitivity may be estimated from the data. At early age this would require a high frequency of readings; at least every 10 minutes. Temperature sensitivity estimated from frequent high-quality readings will have an accuracy of ±5%, based on experience of a sprayed concrete test panel with embedded tangential cells linked to a datalogger in the laboratory. However, for lower frequency readings the accuracy of estimates of temperature sensitivity is more likely to be ±10%, and perhaps as high as ±20% at early age when the coefficient of thermal expansion and stiffness of the sprayed concrete are changing rapidly (see Figure 3), and these values are applied in the example given.

If the estimated errors described above are applied to real data, the following confidence limits are obtained for a tangential pressure cell (Figure 4).

A similar calculation can be done for radial cells. A major difference, however, is that radial cells have not been observed to exhibit sensitivity to temperature until the ring of sprayed concrete is closed. Therefore, the temperature sensitivity of a radial cell must be due to expansion and contraction of the completed ring of sprayed concrete against the ground and since this represents a stress that actually exists as against the temperature sensitivity of a tangential cell which is due to the properties of the cell itself, it is debatable whether it should be removed from the data. Since adjustments to the recorded pressure due to temperature sensitivity are made based on the change in temperature, the exact point at which the system becomes sensitive to temperature must be known accurately, or a significant offset in pressure may be introduced. However, if only the change in pressure with time is required, this becomes unimportant. In the example given, a complete ring of sprayed concrete was constructed in a full-face excavation. However, the radial cells did not appear to respond to changes in temperature until 3 hours after spraying, therefore this was used as the base temperature. Since the temperature sensitivity is likely to be more dependent on the properties of the soil in this case, the temperature sensitivity is assumed constant. Again a minimum value of CAF of 0.9 and a maximum value of 1.0 was used, with a best estimate of 0.95, as well as an estimate of the accuracy of the temperature sensitivity at ±10%.

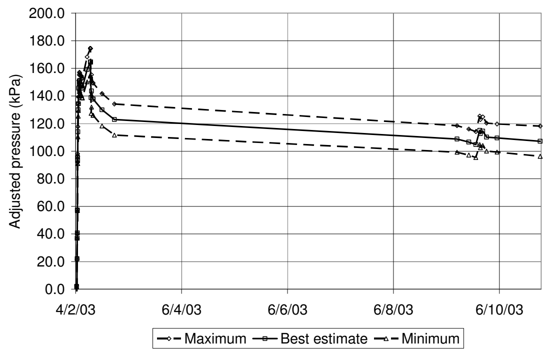

Figure 5: Confidence limits for a typical radial pressure cell

There may be other errors introduced, such as the accuracy of the vibrating wire reader, the accuracy of the vibrating wire transducer calibration, and temperature gradients across the sprayed concrete lining's thickness due to diurnal temperature changes. However, these are negligible relative to the errors previously mentioned.

Although the error shown in Figure 4 appears to be large, it can be reduced by improving the estimate of temperature sensitivity. This could be done by increasing the frequency of readings, especially at early age. On this project, only a handheld readout was provided and it was not possible to take any readings for approximately 6 months during the main TBM drive due to access restrictions. It is recommended that in future a datalogger is used; this will allow more frequent readings to be taken at all times, and it is the experience of the author that more stable readings are usually obtained.

3 – BACK-CALCULATION

There are many methods of back-calculation, the majority being rheological models based on the rate of flow method (England & Illston, 1965). A generalised Kelvin model is normally used, which accounts for elastic and delayed elastic strain (primary creep), and terms are often added to account for viscous (or secondary) creep, temperature and shrinkage. Usually, optical surveying measurements are used with assumptions of constant strain along each arc segment, no bending and uniaxial sprayed concrete behaviour. Sometimes embedded strain gauge or tape extensometer readings are used, which should be expected to improve accuracy, although proponents of the back-calculation method insist that optical convergence measurements are accurate enough (e.g. Rokahr & Zachow, 1997). Calculation methods may attempt to take variations of strain and hence bending moments into account by curve-fitting (e.g. Macht *et al.*, 2003). However, in the case of a full-face excavation with the sprayed concrete ring closed immediately, this is not possible without an assumption of curvature at one of the points and therefore it is not attempted here for this example.

To check the validity of the method, a back-calculation of the strains measured during a uniaxial utilisation test was attempted, and the results compared to the stresses imposed.

A utilisation test is a uniaxial compression test where the strain rate is controlled such that the sample is kept at a constant utilisation. Utilisation is the ratio of imposed stress to the strength. Standard uniaxial compression tests of the same batch of concrete were used to find the strength development with time and hence calculate the target stress required to maintain constant utilisation in the sample.

The following rheological model was used for the strain at timestep n:

$$\varepsilon_n = \varepsilon_{n-1} + \varepsilon_{(K)n} + \varepsilon_{(Flow)n} + \varepsilon_{(Shr)n} + \varepsilon_{(T)n}$$

- Equation 1

Where $\varepsilon_{(K)n}$ is calculated by a Kelvin model representing elastic strain and delayed elastic strain taken from Thomas (2003). Delayed elastic strain approximates the effect of primary (or recoverable) creep, which for concrete is due to water movement in the pores and occurs over a time period of the order of 10 days (Acker & Ulm, 2001) and is fully recoverable. $\varepsilon_{(Flow)n}$ is the flow strain, also known as secondary or steady-state creep, and is so called because its rate is only dependent on age and stress level. Flow strain is caused by irreversible viscous slippage between layers of hydrates, occurs over a much longer time-scale than delayed elastic strain and is irrecoverable (Acker & Ulm, 2001).

For the laboratory tests, which were over a short time period of between 7 and 11 hours and used cylinder samples, temperature strain $\varepsilon_{(T)n}$ and shrinkage strain $\varepsilon_{(Shr)n}$ were ignored. The strain at timestep n for all stress increments r in the Kelvin model was given by the following equation:

$$\varepsilon_{(K)n} = \frac{\sigma_n}{9K} + \frac{\sigma_n}{3G} + \frac{1}{3G_k}\sum_{r=1}^{n-1}(\sigma_r - \sigma_{r-1})\cdot\left(1 - e^{-G_k(t_n - t_r)/\eta}\right)$$

- Equation 2

This equation effectively adds another Kelvin element for each stress increment σ_r. The equation can be rearranged to find the stress due to elastic and delayed elastic strain at timestep n:

$$\sigma_{(K)n} = \frac{\varepsilon_n - \frac{1}{3G_k}\sum_{r=1}^{n-1}(\sigma_r - \sigma_{r-1})\cdot\left(1 - e^{-G_k(t_n - t_r)/\eta}\right)}{\left(\frac{1}{9K} + \frac{1}{3G}\right)}$$

- Equation 3

The flow rate at timestep n may be given by any of a number of different relationships. In this example the relationship between compliance rate and age given by Acker & Ulm (2001) was used:

$$\frac{dJ}{dt} = \frac{5.1}{t}$$ - Equation 4

The stress due to flow strain at timestep n is given by:

$$\sigma_{(Flow)n} = \frac{\frac{dJ}{dt}\cdot(t_n - t_{n-1})\cdot\sigma_{n-1}}{\left(\frac{1}{9K} + \frac{1}{3G}\right)}$$ - Equation 5

This flow stress is simply subtracted from the calculated Kelvin stress $\sigma_{(K)n}$ to give the total stress σ_n. Similarly, stresses due to temperature or shrinkage could be subtracted.

The value of Young's modulus used for the sprayed concrete was related to the strength by the relationship proposed by Chang & Stille (1993) in equation 6. Bulk modulus K and shear modulus G may be derived from Young's modulus E using a Poisson's ratio v of 0.2.

$$E = 3.86 f_c^{0.6}$$ - Equation **6**

The uniaxial compressive strength of the batch of sprayed concrete was found at four different ages by strength testing of cylinders. This allowed the target utilisation to be set. It also meant that the strength development with age during the time period of the test was known. This was approximated to a linear relationship, which fitted the data well with a regression coefficient close to unity.

A comparison of the back-calculated stress with the stress measured in the test by the load cell attached to the apparatus is shown in Figure 6 below.

Figure 6 generally shows good agreement. Problems can arise with a back-calculation if the timesteps are too far apart. Since the stress in the test was continuously increasing, and the back-calculation assumed stress increments occur instantaneously at a given timestep, this should be expected.

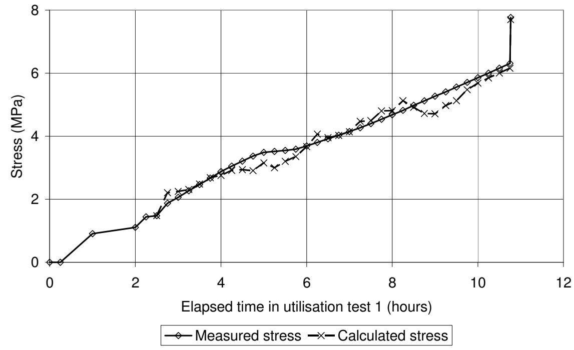

Figure 6: Back-calculation of utilisation test 1

This occurred in utilisation test 2, which is shown below in Figure 7. The interesting point to note is the back-calculation's apparent inherent ability to correct itself. It does this when the increment of strain is less than predicted by the sum of the delayed elastic strain components so far accumulated.

On such occasions, the stress increment will become negative and the method appears to correct itself. Although a seemingly beneficial numerical effect, this has important ramifications for numerical stability when the method is applied to the tunnel data, which is less accurate and fluctuates more wildly.

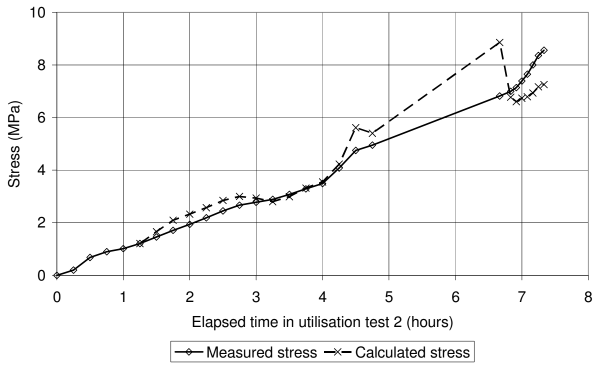

Figure 7: Back-calculation of utilisation test 2

The tunnel considered was excavated full-face, with a complete circular ring of nominal 325mm thickness and 4.8m external diameter sprayed immediately after excavation. At the monitoring section considered, the first reading of the monitoring points was 5.5 hours after the ring was sprayed. From then on, they were surveyed approximately 1-2 times per day. At each monitoring section, there were 5 monitoring points; point 1 at the crown, point 2 at the left shoulder, point 3 at the right shoulder, point 4 at the left knee and point 5 at the right knee as shown in Figure 8.

The monitoring data is in 3-dimensional coordinate form with components of chainage along the tunnel centreline and horizontal and vertical offsets from the tunnel centreline. Firstly this data must be converted to displacements by subtracting the first reading, and then the horizontal and vertical displacements must be converted to radial and tangential displacements. For the radial displacements, convergence was taken as positive, and for the tangential displacements, clockwise displacements were taken to be positive.

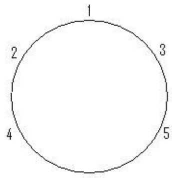

Figure 8: Position of monitoring points

Several assumptions were made at this stage:

1. The longitudinal displacements along the tunnel centreline were ignored.
2. Thermal strains were not considered.
3. Shrinkage strains were not considered.
4. The effect of multiaxial stress states was ignored; a 1-dimensional constitutive law was applied in the back-calculation.

Problems arose when trying to apply the back-calculation to the data. The reason for this can be seen in a plot of radial displacement of the monitoring points against time, shown in Figure 9 below.

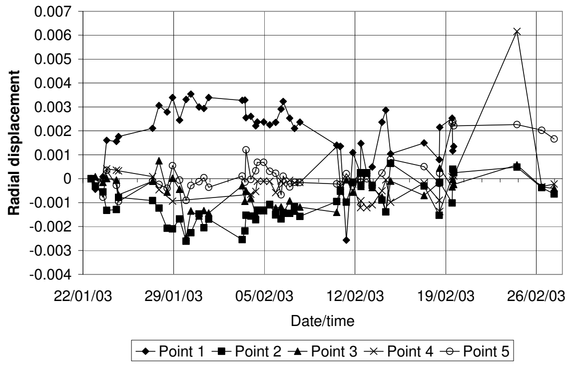

Figure 9: Radial displacement of monitoring points

Figure 9 shows that there was very little movement of the monitoring points, in general less than ±3mm. The accuracy of optical surveying techniques is typically ±2-3mm according to Bock (2003) and with best practice methods can be as accurate as ±1mm (Clayton *et al.*, 2000). In this case, the accuracy appears to be approximately ±2mm for the most part and occasionally much worse. Apart from an initial movement of the crown downwards and the shoulders outwards, no pattern is perceptible. The small fluctuations in the readings caused by error, however, cause numerical instability in the back-calculation, which then oscillates from very large positive stress increments to very large negative stress increments.

It appears, therefore, that in this case the use of a back-calculation method is a wasted effort. The pressure cell data shows that the ground pressure increases quickly once the ring is closed. By the time the monitoring points are surveyed for the first time, most of the deformation has already occurred. Coupled with the inadequate accuracy of the surveying method and the low frequency of readings, this means that the detail of load development is lost. This does not mean that this form of monitoring is not important to observe the stability of the lining, and continued deformation would be an indicator that the lining is not performing as intended. But the use of optical surveying data to determine the stress state of a well designed and constructed sprayed concrete lining in soft ground is not possible.

In order to make this method of stress determination work, both these conditions would need to be met:

1. The magnitude of expected deformations would need to be much larger than the accuracy of the displacement measurements.
2. The period over which the deformations develop would need to be several times longer than the surveying frequency.

Condition 1 could be met by improving the accuracy of the surveying, for example by using a tape extensometer, which has an accuracy of ±0.13mm over a 10m span (Dunnicliff, 1993 or Hanna, 1985). Clayton *et al.* (2000) state that optical surveying with a total station will only provide warning of impending failure if the accuracy is ±1mm or better for a 10m diameter tunnel. In a smaller tunnel, such as the one studied in this paper, accuracy would need to be better than this since the same strain at failure would cause a smaller convergence. Another solution could be the use of embedded strain gauges. Condition 1 could also be met in a rock tunnel, where the deformations are often much larger than in soft ground.

Condition 2 could be met by increasing the frequency of surveying, particularly in the first 5 days. This could be quite disruptive to construction activities, especially if tape extensometers were used. Although the use of a total station should reduce disruption, in practice, especially in smaller diameter tunnels, surveying hinders production. Equally, condition 2 implies that if the deformations are large and occur gradually over several weeks, as is often the case in swelling rocks (e.g. quartzitic phyllites at Strenger tunnel in Austria - Novotný & Mařík, 2004), a typical surveying frequency of once per day or once per shift may be sufficient.

4 - DISCUSSION

That both the convergence and pressure cell readings stabilised so quickly should not be surprising, in fact it should be the aim of the soft ground tunnelling method employed to limit ground deformations as much as possible. In 1969, Peck said, "Long experience has demonstrated that, except possibly in certain swelling clays, no tunnelling method [in soft ground] has yet been developed in which the strains and deformations are so small that the strength of the soil is not largely mobilised. Therefore, it is quite properly considered good practice to keep the deformations as small as possible, in order to hold the avoidable loss of ground and consequent settlement to a minimum and to prevent deterioration of the soil due to excessive local distortions or remoulding" (Peck, 1969b). The tunnel was designed with these principles in mind; to build a complete, stiff, circular ring as soon after excavation as possible.

By the time the first survey was made, 5.5 hours after the ring was sprayed, the ring of sprayed concrete would already have attained considerable stiffness and would not allow much further deformation. The rock tunneller's philosophy of using a thin, flexible lining that allows the ground to deform and thus mobilise an arch in the ground which will reduce lining loads is not applicable to soft ground, where it is more important to tightly control deformations to conserve the ground's ability to support itself as much as possible. There is evidence to suggest that although allowing the ground to deform will reduce lining loads in the short term in cohesive soils (Negro *et al.*, 1996), it is likely that the effects of increased consolidation due to large shear strains and positive excess pore pressures will more than reverse this in the long-term.

Pressure cells can give continuous, reliable readings of stress on and in sprayed concrete linings, provided care is taken in their installation and interpretation. The example data show that potential errors in interpretation are likely to be tolerable. It is possible to further reduce these errors by using a datalogger and increasing the frequency of readings to obtain a more accurate estimation of temperature sensitivity. The back-calculation of lining displacements method, on the other hand, was swamped by error to such an extent that the results were meaningless.

An alternative to the back-calculation of lining displacements method could be to back-calculate strain gauges embedded in the sprayed concrete lining. Since the assumptions made in order to derive the strains from lining displacements would not have to be made, the use of strain gauges would be an improvement. However, unlike tangential pressure cells, to determine stresses from strain gauge data would require the use of a constitutive law similar to that applied in this paper to strains derived from lining displacements. This was done by Golser *et al.* (1989). However, even if site-specific creep tests were performed to calibrate the model the variability of sprayed concrete properties is usually large. For example, experience of two construction sites indicates that the standard deviation of sprayed concrete compressive strength obtained from cores was approximately 20% of the strength. The standard deviation of the difference between the strengths of two cores from the same batch of sprayed concrete was 5%. Therefore, it would be better to measure stresses directly.

5 - CONCLUSIONS

It is necessary to determine stress in a sprayed concrete lining as part of a holistic risk management process, to verify that the design assumptions are correct.

Any method employed to determine stress in a sprayed concrete lining should be chosen with due consideration of its accuracy and convenience.

The aim of soft ground sprayed concrete tunnelling is to minimise deformation of the ground as much as possible to conserve the ground's ability to support itself; to resist rather than to control deformations (Powell & Beveridge, 1998).

Pressure cell readings from sprayed concrete tunnels in London Clay show that load increases quickly once the ring is closed and stabilises completely within 1-4 days with no further long-term change. Similarly, optical surveying of targets from 5.5 hours after completion of the ring showed little discernible deformation, and this deformation was of the same order of magnitude as the surveying accuracy.

The nature of the surveying data meant that it was not possible to determine stress by back-calculation of displacements. It is likely that this will always be the case in soft ground unless more accurate and more frequent surveying is undertaken. Accuracy would need to be at least as good as ±1mm and the first survey would need to be almost immediately after spraying the lining, with a frequency of surveying of every 2-4 hours until about 3 days after completion of the ring.

The least disruptive and most accurate method of determining stress in a sprayed concrete tunnel in soft ground is to install pressure cells.

NOTATION

ε_n	strain at timestep n (n = 1,2,3…)
$\varepsilon_{(K)n}$	strain due to Kelvin model at timestep n
$\varepsilon_{(Flow)n}$	strain due to irrecoverable creep (flow) at timestep n
$\varepsilon_{(Shr)n}$	strain due to shrinkage at timestep n
$\varepsilon_{(T)n}$	strain due to temperature changes at timestep n
σ_n	stress at timestep n
σ_r	stress increment (r = 1,2,3….n-1)
η	Kelvin viscosity parameter
K	bulk modulus K = E/(3(1-2 v))
G	shear modulus G = E/(2(1+v))
G_k	Kelvin spring stiffness modulus
J	'compliance' = deformation per unit stress
t	age of sprayed concrete in hours

DEFINITIONS OF KELVIN CONSTANTS

$$\eta = \frac{1.5 \cdot 10^{11} \cdot e^{\left(\frac{-1.5}{t^{0.6}}\right)}}{2(1+\nu)} kPa.s$$, where t is the age in days

$$G_k = \frac{8.0 \cdot 10^{6} \cdot e^{\left(\frac{-1.0}{t^{0.4}}\right)}}{2(1+\nu)} kPa$$, where t is the age in days

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