was mentioned earlier in Cooper & Chapman (1998).

The asymmetry was first reported by Cording & Hansmire (1975) and it has been followed up by subsequent researchers. For a number of conditions for tunnels at varying distances apart, Cording & Hansmire (1975) plotted the ratio $dV_s/Vs2$, against the distance apart normalised by diameter.

Figure 10 is based on their plot and shows their points, with their original curve. Additional points from other sources are also plotted. All these other points relate to surface settlement. Four additional points from the Upline/Concourse and the Downline/Concourse on the Inner and Outer Piccadilly Line tunnel at CTA are also plotted. The correlation of the Heathrow points with the original plot varies.

No theoretical methods have been found by the authors which are commonly used to predict rotation and distortion of an existing tunnel when new tunnelling is undertaken nearby. However, part of a research project underway at Birmingham University is aimed at formulating methods for this.

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Field investigations of long term ground loading on an old tunnel in London Clay

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ABSTRACT: This paper presents the results of a field study carried out early in 1998 to investigate the current ground conditions around an old cast iron lined tunnel in London Clay. The eventual loading carried by these tunnel linings is unknown, but concern exists that there may be a gradual increase in loading with time. Considerable lengths of the London Underground network run through London Clay in tunnels constructed with cast iron segmental linings. The investigation therefore provides information vital to the estimation of the remaining design life of London's underground transport infrastructure.

1 INTRODUCTION

London Underground Limited (LUL) is currently undertaking a project to evaluate the long term durability and capacity of segmental cast iron tunnel linings in service on the London Underground network. The project runs in parallel with an EPSRC grant awarded to Cambridge University Engineering Department (CUED).

For the purpose of this collaborative investigation LUL chose as a case study a section of a single running tunnel of significant age (constructed in 1924) with a cast iron segmental lining. From historical records and geological maps the tunnel is known to lie within a substantial thickness of London Clay. The section chosen runs under a green-field site and is thus a suitable location for conducting a site investigation. In December 1997 LUL contracted Soil Mechanics Limited (SML) to carry out a site investigation of the study area and Geotechnical Consulting Group (GCG) were appointed as the consulting engineers. The fieldwork was carried out between January and May 1998.

The field study which is presented in this Paper included comprehensive monitoring of ground water pressures with vibrating wire piezometers installed in boreholes at various depths and distances from the tunnel and radially from within the tunnel. The piezometer data was complemented by a programme of self-boring load cell pressuremeter, expansion pressuremeter and permeameter field tests to determine the *in situ* stress state and permeability of the clay both adjacent to and remote from the tunnel.

2 BACKGROUND

It is usually assumed that segmental tunnel linings in low permeability clays do not provide an impermeable barrier but cause permanently reduced pore water pressures in the near vicinity and seepage into the tunnel, i.e. the tunnel acts as a drain (Ward & Pender, 1981). However, this has never been systematically verified by measurement of pore water pressures.

The mechanisms of pore water pressure dissipation and equilibration following construction of a tunnel in clay cause swelling and consolidation of the clay around the tunnel, leading to a time dependent increase in loads carried by the lining. Knowledge of the eventual ground loading acting on tunnel linings is very limited and the nature and extent of the gradual increase in loading with time is poorly understood.

Investigations to identify the development of load carried by old cast iron tunnel linings in London Clay with time have been undertaken in the past using methods of direct measurement, for example with strain gauges mounted on lining segments or pressure cells inserted between the segments (e.g. Ward & Thomas 1965; Barratt et al., 1994). These studies suggest equilibrium vertical loads acting on tunnel linings between 60 and 100% of the full overburden pressure. Further to the difficulties in interpreting these data to quantify the loading applied to the tunnel lining by the ground, much of the data refers to multiple closely spaced tunnels, the development of load around which would be

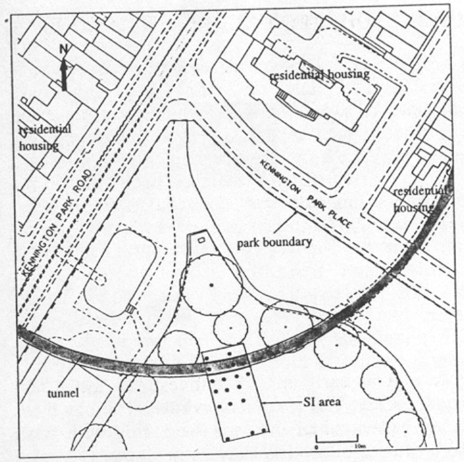

Figure 1. Plan of case study area

expected to be very different from that around a single tunnel.

The key to understanding the long term ground loading on tunnel linings in clay is the measurement of the ground water pressure regime, close to and remote from the tunnel, and very few such measurements exist around old tunnel linings in London Clay.

3 SITE DETAILS

A plan of the case study area is shown in Figure 1. The tunnel section chosen for the investigation forms part of the Kennington Loop on the Northern Line of the London Underground network. Kennington is situated in the Borough of Lambeth in South London.

The site is grassed, bounded on its northern side by several mature trees, and is used solely as a recreational facility. Residential housing bounds all sides of the park, the closest dwellings to the study area being within 60m on the north side of Kennington Park Place.

According to the geological map of the area the site is underlain by River Terrace Deposits resting on London Clay followed by the Lambeth Group (Woolwich and Reading Beds), the Thanet Sand Formation and the Upper Chalk.

The tunnel has an internal diameter of approximately 3.6m (11 feet 8 ¼ inches), lined with six cast iron segments bolted together and grouted into position. At the tunnel section chosen for study the tunnel axis is located approximately 21m below ground level. The ground is reasonably level over the entire study area so for clarity throughout this paper depths are stated in metres below ground level.

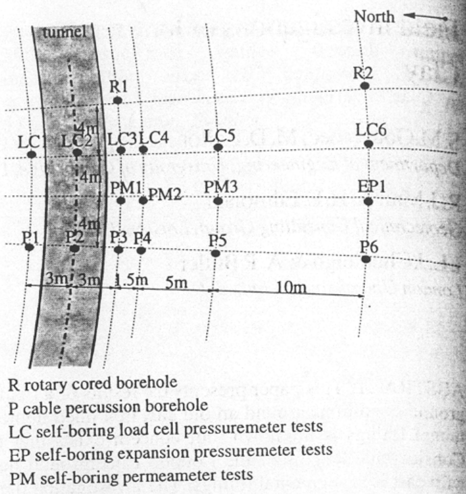

Figure 2. Plan of site investigation boreholes

4 SITE INVESTIGATION

The fieldwork for the site investigation comprised eighteen exploratory holes located in a 12m east-west zone extending 3.0m to the north of the tunnel centreline and 19.5m to the south. A plan of all the site investigation boreholes with respect to the position of the tunnel is shown in Figure 2. The two R boreholes located along the eastern boundary of the study area were formed by rotary coring, the six P boreholes located along the west boundary of the study area were formed by cable percussion and the remaining 10 holes (LC, PM and EP) were commenced by cable percussion and completed with *in situ* self-boring devices.

4.1 *Stratigraphy and soil classification*

Records of the materials encountered during formation of the R and P boreholes identified the sequence of deposits at the study area as: Made Ground (0-1.8m); River Terrace Deposits - sand and gravel (1.8-7.5m); London Clay - stiff fissured clay (7.5-15.0m) inter-laminated with silt and sand partings (15.0-25.5m); Lambeth Group – very stiff clays, dense silty sands and gravels (25.5-40.0m); Thanet Sand – dense silty sand (40.0–50.0m); Chalk (below 50m). The drilling records also showed that the River Terrace Deposits were above the water table, ground water being encountered at the top of the London Clay stratum.

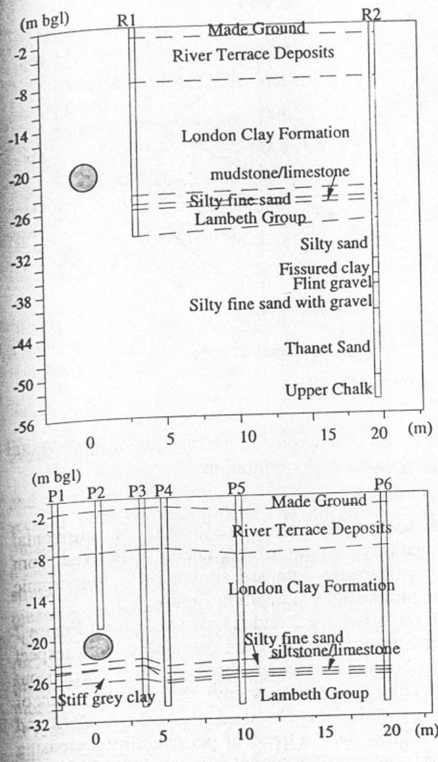

Figure 3. Stratigraphy

A summary of the stratigraphy identified by core description at the cross sections of boreholes R1-R2 and P1-P6 are shown in Figure 3 (the position of the tunnel is also shown).

Samples were retrieved during formation of the cable percussion borehole P5 for visual description and laboratory testing. The laboratory tests and soil descriptions were carried out in accordance with the British Standard code of practice BS1377. A summary of the geotechnical parameters of the London Clay is presented in Figure 4.

The tunnel is located within the London Clay although its invert is very close to the interface with the Lambeth Group, where sandy facies predominate. The London Clay at tunnel axis level was described as a very stiff dark greyish brown closely fissured fine sandy clay with closely spaced partings of light greyish brown silty fine sand, the fissures being generally sub-horizontal and sub-vertical, smooth, regular, planar and undulose.

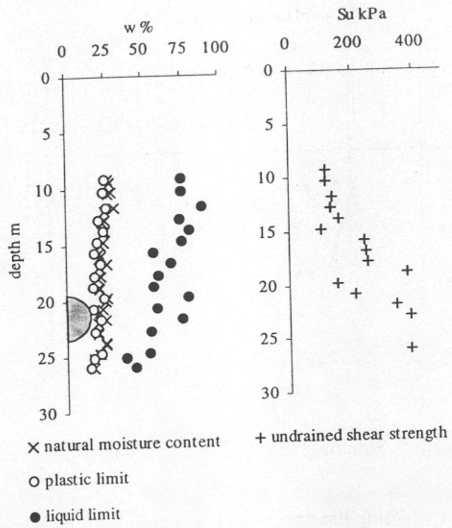

Figure 4. Summary of geotechnical properties

4.2 *In situ testing*

In situ lateral earth pressures and horizontal permeability of the London Clay were derived from tests with self-boring instruments at depths between 10 and 23m and distances from the tunnel centreline between 3 and 19.5m. The lateral earth pressure tests were carried out in boreholes LC1-LC6 and the permeability tests in boreholes PM1-PM3. This phase of the field work was sub-contracted to Cambridge Insitu (CI), and is described below.

In situ lateral stress measurements were made with a self-boring load cell pressuremeter (LCPM), measuring total stress in the surrounding soil with six independent load cells distributed evenly around the circumference of the instrument sleeve. The LCPM is a new instrument, developed by the Transport Research Laboratory (TRL) and CI for the assessment of Ko (Darley et al. 1996), and is designed to minimise soil disturbance during installation. Previous experience with the instrument is very limited, and so the far field lateral stress conditions measured in borehole LC6 were verified against results from control tests carried out in borehole EP1 with a conventional self-boring expansion pressuremeter.

The arithmetic mean of the six independent total horizontal stress measurements from each of the LCPM tests are shown against depth in Figure 5. Considerable scatter is apparent indicating that averaging the individual load cell readings provides an inappropriate representation of the stress state.

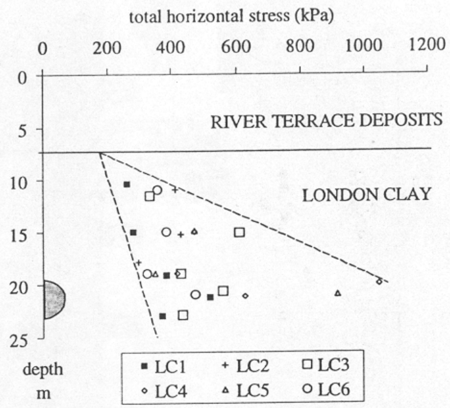

Figure 5. Summary of total horizontal soil stresses

Considering the upper and lower bounds of the data points gives an *in situ* effective earth pressure coefficient, Ko, in the range 0.7 to 3.3. Taking the arithmetic mean of the data points to produce a single profile of horizontal stress with depth indicates a value of Ko of 1.3, which is reasonable for London Clay.

The benefit of the LCPM is its ability to identify scatter at a given location with respect to orientation around the perimeter of the instrument. As an illustration Figure 6 shows the individual total horizontal stress readings from the six load cells at a depth of 10.4m in borehole LC1. Inconsistency between diametrically opposed cells, i.e. on the same orientation (1/4, 2/5, 3/6) indicated localised, non-uniform disturbance during installation.

When the instrumented section shifts laterally due to changes of verticality during insertion (i.e. "wobbles"), one cell will rise while its diametrical partner will fall. Just as with the translation of a retaining wall due to external forces, the increase in pressure on the passive side will not generally balance the reduction on the active side. A programme of back-analyses of the "wobble" using finite elements is underway to refine the interpretation of the raw data.

In situ permeability measurements were made using a new self-boring instrument also developed by CI. A rigid perforated sleeve encapsulates a conventional self-boring expansion pressuremeter, forming the outer body of the instrument. Once at the required test depth water is injected from ground level at a constant rate to the soil through the perforations and the supply pressure required to establish constant flow conditions is recorded. The test is repeated at a number of flow rates. The gradient of a plot of pressure against flow rate is

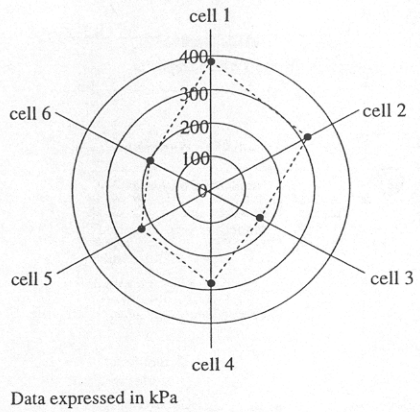

Figure 6. Variation in total horizontal stress measurements with orientation

used to calculate the coefficient of horizontal permeability. Values of permeability derived from the permeameter data are shown on a logarithmic scale, plotted against depth in Figure 7.

The profiles L1, L2 and L3 also shown on Figure 7 result from regression of the data points with respect to the borehole in which the tests were carried out (neglecting the two very low values at a depth of approximately 20m observed in boreholes PM2 and PM3). They show a trend of permeability increasing towards the tunnel at tunnel axis level (i.e. PM1 (L1) indicates higher values than PM2 (L2) and PM3 (L3)). It is also of interest to note that the permeability increases with depth, the average permeability increasing by an order of magnitude over the first 10m depth of London Clay.

Furthermore, the permeability of the London Clay encountered at the Kennington site is unusually high, reaching 10^{-8}m/s near its base, while commonly London Clay is found to exhibit values of horizontal permeability in the range $5x10^{-11}<k_h<5x10^{-10}$m/s. This is probably as a result of the London Clay becoming more granular with depth, with closely spaced partings of sand being observed in the undisturbed samples.

4.3 *In situ piezometer monitoring*

Ground water conditions across the site were identified by vibrating wire piezometers installed in each of the R and P boreholes between depths of 7 and 48m. In addition vibrating wire piezometers were installed radially from within the tunnel to measure water pressures close to the tunnel. These were located 1m behind the tunnel lining, at axis level on both the north and south side of the tunnel

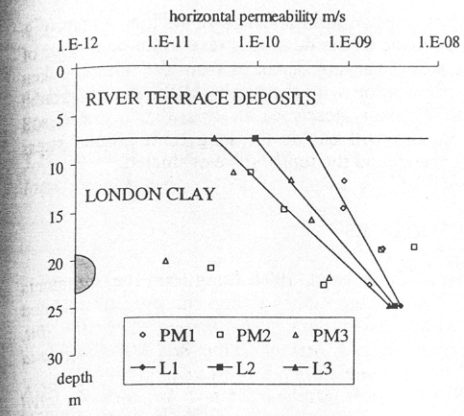

Figure 7. Summary of permeameter tests

and from the knees and the invert, at the same cross section as the P boreholes. The installation of the piezometer from the invert led to a rapid water entry, a clear indication of the high permeability of the soil strata beneath the tunnel. The position of the piezometers is illustrated in Figure 8.

Ground water pressures recorded by the borehole piezometers 12 months after installation are shown in Figure 9 providing an indication of the general piezometric conditions. The piezometers were read at regular intervals after their installation and steady state conditions were achieved within this time frame. No significant variation in pore water pressure is apparent with proximity to the tunnel, suggesting that the tunnel is not acting as a drain and hence not causing seepage towards it.

The pore water pressure data within the clayey strata indicate a slightly sub-hydrostatic profile L consistent with a water table at 5m depth in the River Terrace Deposits with under-drainage into the Thanet Sand. The two data points within the Thanet Sand conform to a hydrostatic profile H2 with a piezometric surface corresponding to a depth of 15m. The reduced piezometric level in the permeable aquifer (Thanet Sands and Chalk) underlying the London Clay and the Lambeth Group arises from historical pumping from deep wells in the London area. Cessation of the pumping sometime ago has resulted in a steady rise of the piezometric level in the underlying aquifer (Simpson et al, 1989).

Some evidence of seepage into the tunnel is indicated by the ground water pressures measured by the piezometers installed just behind the tunnel lining. Figure 10 shows the readings of the piezometers installed from within the tunnel and it is clear that the ground water pressures very close to the tunnel, over the depth of the tunnel, fall below

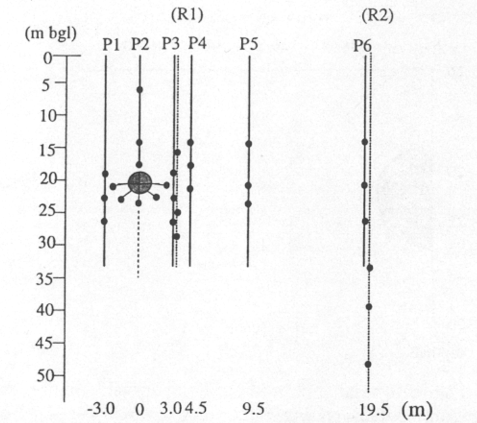

Figure 8. Position of piezometers

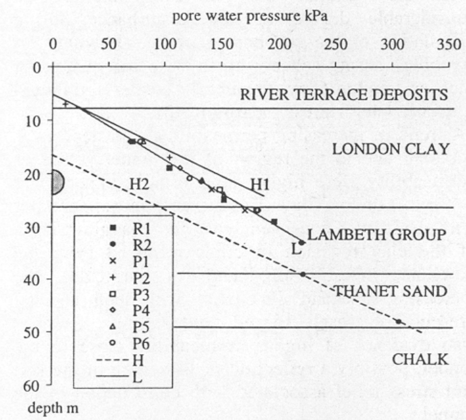

Figure 9. Summary of piezometer readings

the regression line L of the borehole data shown in Figure 9.

5 CONCLUSIONS

A comprehensive investigation of the ground conditions around a 75 year old tunnel has been undertaken. The London Clay was found to be a varied material, generally a stiff fissured over-consolidated clay, with a comparatively high proportion of sand and silt as well as bands of mudstone and limestone, especially near its base.

The mean horizontal stresses measured by the

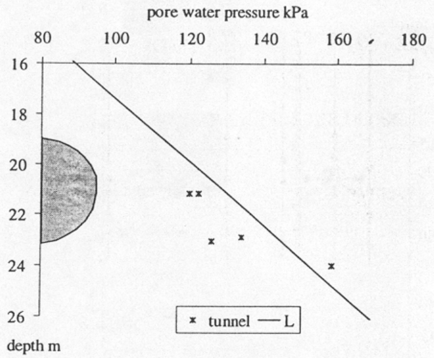

Figure 10. Ground water pressures around the tunnel

LCPM exhibited considerable scatter but were within the expected range for London Clay. The individual results did however indicate a considerable degree of soil disturbance during installation of the self-boring probe, showing that "wobble" during self-boring in very stiff materials, such as London Clay, apparently creates significant scatter in lateral stress measurements.

A trend of increasing permeability with depth was apparent and in the region of the tunnel values of permeability were higher than usually expected at depth in London Clay. The increase in permeability with depth was consistent with the changing fabric of the clay revealed in cores retrieved from the borehole investigation, and in particular the increasing sand and silt content with depth and the presence of closely spaced sand partings. There is also evidence of higher permeability close to the tunnel, possibly a reflection of loosening of the soil and stress relief associated with construction of the tunnel.

Piezometer data from the boreholes showed little evidence of seepage towards the tunnel. However the data from the piezometers installed from the tunnel, close to the lining, showed a reduced pore water pressure. The combined data suggest that some local seepage towards the tunnel is occurring, with pore water pressures corresponding to the far field being reached within about 1.5m from the tunnel.

The investigations of the ground conditions, horizontal stresses, permeability and pore pressures existing around a 75 year old tunnel in London Clay have provided important data for the assessment of the ground loading acting on the tunnel lining. A programme of triaxial tests, for which small strain measurements are being made, is currently being undertaken on soil specimens taken from the high quality rotary cored samples both close to the tunnel and at distance from it. This will enable comparisons to be made of the detailed stress-strain behaviour of the soil at varying distances from the tunnel. Taken in conjunction with the results of the *in situ* ground investigations described in this Paper, these soil properties will enable the long term ground stress regime around the tunnel to be evaluated.

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Compensation grouting to control tilt of Big Ben Clock Tower

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ABSTRACT: Big Ben Clock Tower is the most famous of many historic and important structures affected by ground movements associated with construction of the Jubilee Line Extension Project in London. The construction works for the new Westminster Station are outlined. Compensation grouting undertaken to control the tilt of the Tower is described, drawing particular attention to the instrumentation used to monitor tilt and the precision to which the tilt could be controlled.

1 INTRODUCTION

The construction of Westminster Station on London Underground Limited's (LUL's) new Jubilee Line Extension (JLE) project in London was predicted to produce significant movements of The Big Ben Clock Tower and the adjoining Palace of Westminster (Fig. 1) as a result of excavation of two 7.4m OD (outer diamater) tunnels and the 39m deep station escalator box. These activities produce two components of movement; an initial, immediate movement directly associated with the progress of excavation and a time related component due to drainage and consolidation of the London Clay. Protective measures, primarily in the form of compensation grouting below the Clock Tower, have been implemented during the construction period to control settlement and tilt of the structure.

This Paper describes the compensation grouting and monitoring procedures undertaken over a 21 month period to control the tilt of the Clock Tower during construction of the tunnels and the deep excavation for the station box.

2 JLE WESTMINSTER STATION

The layout of the new Westminster Station on the JLE is shown in plan on Figure 2 and in section on Figure 3. The station comprises bored 7.4m OD platform tunnels in a vertically stacked arrangement below Bridge Street with a 39m deep diaphragm wall box to the north. Access to the platforms is provided by four adits between the tunnels and the box at both tunnel levels. Escalators to the ticket hall and Bridge Street with interchange facilities to the new District and Circle station are provided within the station box. Design of the station is described by Carter et al. (1996).

Prior to any substantial excavation within the station escalator box, the running tunnels were driven from east to west as pilot tunnels. The lower, westbound (WB) tunnel was constructed in March 1995 and the upper, eastbound (EB) tunnel in October 1995 at depths of 30m and 21m below ground level respectively. The running tunnels are 4.85m OD and were built in expanded concrete segmental linings, each of which was 1.0m in length. A Howden open face shield with a backactor was used for both drives.

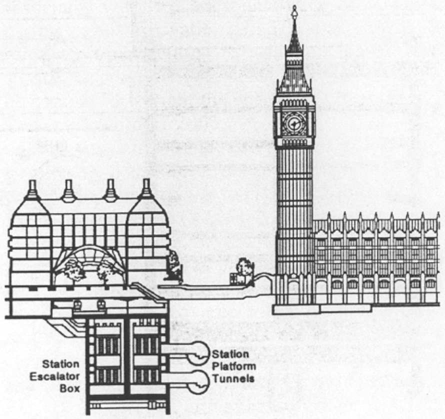

Figure 1. Big Ben and the Palace of Westminster.