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Instrumentation of a Sewer Tunnel in Weak Singapore Soils

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SYNOPSIS The effects of tunnelling in soft ground consisting mainly of Singapore marine clay were recently monitored to establish ground response characteristics. A sewer tunnel of 2.1 m square section was driven at a depth of 6.3 m in this soil by jacking conventional shield and face supports against installed timber lining. Ground response was monitored by an assortment of field instruments read over several weeks' duration. Peck's proposal (1969) of fitting a normal distribution profile to lateral surface settlement field plots when a heading is well past, and the suggestion by Oshikoshi et al (1978) that similar profiles may be drawn across an error function fitted to longitudinal surface settlement field plots, have been confirmed for the site. In addition, similar relationships to the above were found to apply with depth. Thus, taking into account Lo's (1982) determination of standard deviation and ground loss volume for marine clay based on the relationships proposed by Peck (1969) and Yoshikoshi et al (1980), it should, in principle, be possible to determine the vertical ground displacement pattern associated with any tunnel excavation in this soil.

INTRODUCTION

In May 1981, the stretch of sewer tunnel between manholes MH16 and MH17 in Fig. 1 was instrumented to determine the effects of tunnel excavation on weak, waterlogged soils occuring at the site as well as along significant tracts of the 3 km Singapore Ministry of the Environment Bukit Timah Sewerage Scheme, of which the site was a part. After a preliminary site investigation to determine insitu ground conditions, an assortment of inclinometers, standpipes, piezometers and settlement gauges were installed along the tunnel axis as well as at selected lateral sections. All instruments were read regularly from commencement of tunnelling at manhole MH16 until manhole MH17 was breached. In addition, a complete lining frame embedded with total pressure cells was installed at an appointed lateral section to measure the development of earth pressure around the frame. Push-in cones were driven ahead of the heading at two locations to determine the nature of face take.

The following sections describe the site investigation and instrumentation works carried out and include selected findings on ground response to tunnel construction.

SITE INVESTIGATION AND FIELD INSTRUMENTATION

Fig. 1 also shows the instrumentation layout. A site investigation was initially carried out which included seven boreholes, I1 to I6 and AD1. Undisturbed clay samples were taken from the boreholes for laboratory testing and standard penetration tests were performed in granular soils. SINCO inclinometer casings were inserted in the boreholes after completion to measure lateral ground movements. Six water

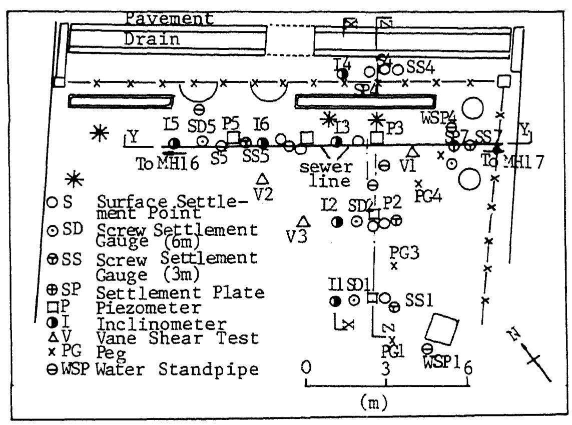


Fig. 1 Site Layout And Instrumentation

standpipes WSP1 to WSP6 were embedded in the unweathered upper marine clay to monitor groundwater table fluctuations. GEONOR vane shear tests were carried out in this clay to determine insitu shear strengths. The results of the site investigation are summarised in the following section.

Field instruments were installed in parallel with the site investigation to allow sufficient settling-in time before the start of tunnelling. Settlement plates SP, settlement points S and PG, and deep settlement screw gauges SS and SD were located in the surface fill deposit and underlying weathered as well as unweathered upper marine clay, respectively, to monitor settlements during tunnelling. Hydraulic piezometers with ceramic tips connected by PVC tubing to a sensitive transducer and voltmeter read-out unit, via a junction box, were located around a selected tunnel section to measure pore pressure response. During tunnelling, the lining frame embedded with total pressure cells shown in Fig. 2 was set in position "X-X" in Fig. 1. The build-up of earth pressures around the frame as the shield drew away was read off a strain gauge meter connected to the pressure cell wiring. At two heading locations on either side of section "X-X", push-in cones were driven into the tunnel face at various penetration depths to study the face take mechanism.

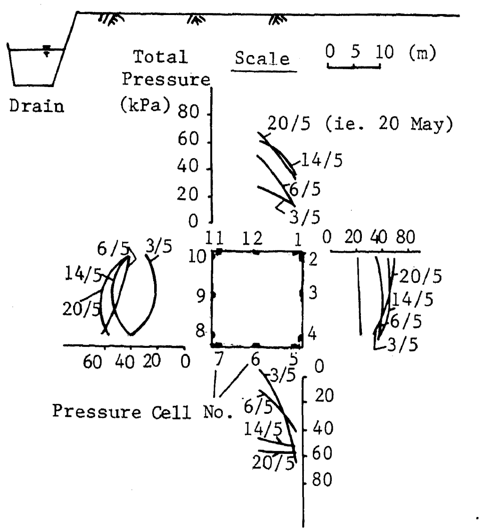


Fig. 2 Earth Pressures On Lining

GROUND CONDITIONS

As shown in Fig. 3, a surface layer of brick and sand fill embedded in weathered upper marine clay covers the site to thicknesses of between 1.7 m and 1.85 m. Underlying this layer is an intercalating sequence of weak, waterlogged, unweathered marine clays and estaurine and fluvial deposits, attaining depths of between 19.7 m and 23.6 m.

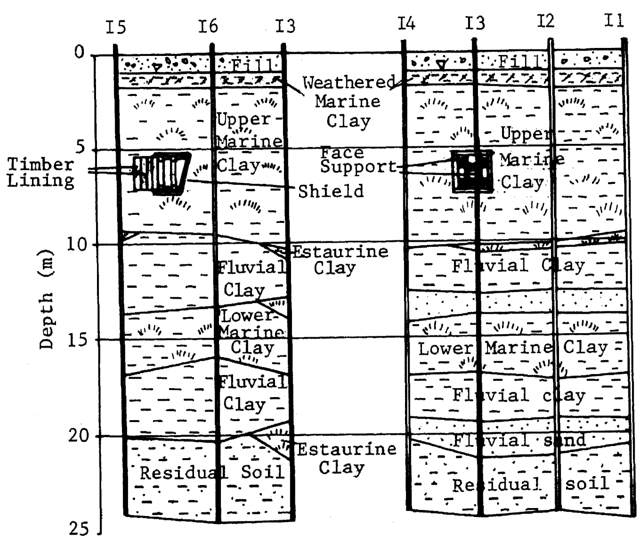


Fig. 3 Site Geology

Laboratory test results for the unweathered upper marine clay, in which the sewer tunnel was driven, are summarised in Table I. These indicate the clay is very soft and highly compressible with correspondingly high water contents. Field vane shear test results indicate the clay has been overconsolidated to a fairly uniform consistency,

TABLE I. Index Properties of Unweathered Upper Upper Marine Clay

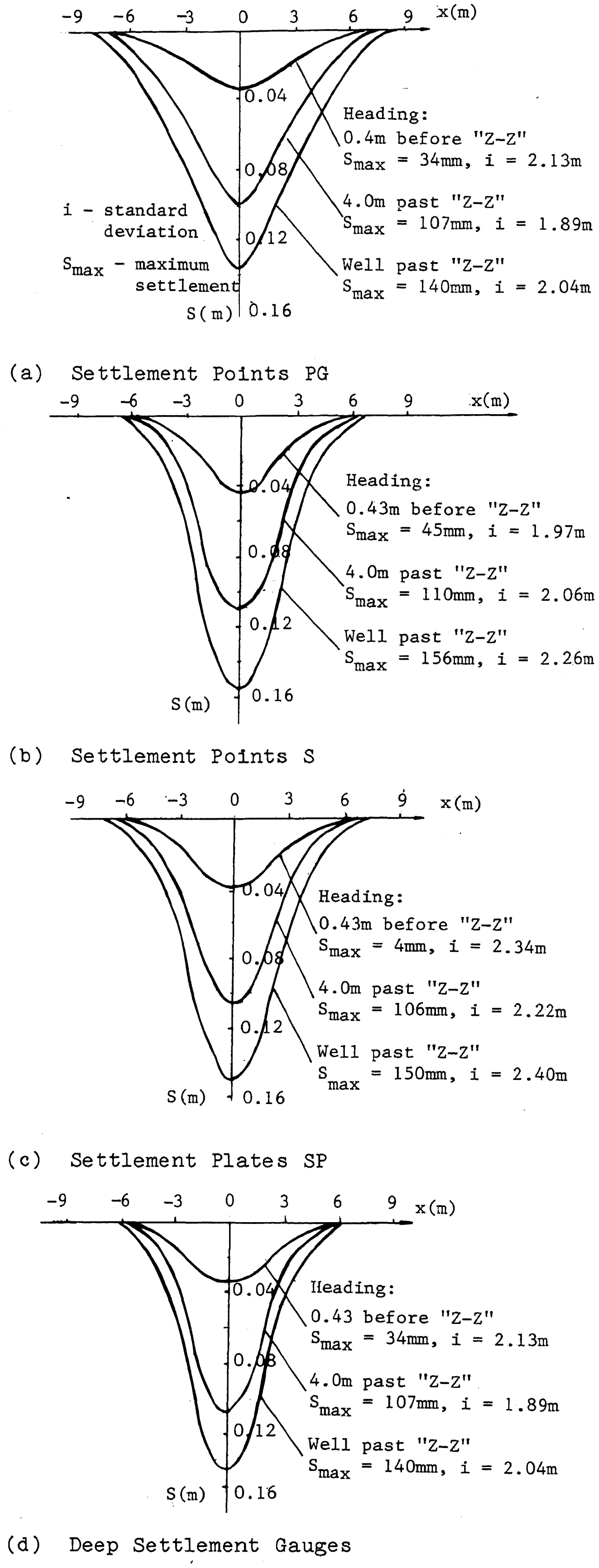
Item No	Test Performed	Results
1	Water Content	113%
2	Liquid Limit	117%
3	Plastic Limit	55%
4	Specific Gravity	2.62
5	Bulk Unit Weight	15 kN/m^3
6	Particle size distribution	
	(a) Passing 2 μm	30% (by
	(b) Passing 63 μm	87% weight)

with c_u = 15 kN/m^2 on average. Insitu constant head permeability tests conducted in similar soils at a different location of the sewer indicated very low permeabilities of the order of 5×10^{-5} mm/s. The underlying fluvial and lower marine clays have similar properties to the upper marine clay, except that the fluvial clays occasionally alternate with thin granular seams connecting to free-draining surfaces, thereby increasing horizontal permeability substantially. The estuarine materials tend to be even softer and more compressible with higher water and organic contents than the marine and fluvial clays. However, field tests show that the insitu permeabilities of the estaurine soils are similar to those of marine clay. The fluvial sands have a loose consistency, registering standard penetration blow counts of the order of 4, and are highly permeable. The bottom residual soils, in which blow counts of about 50 were recorded at upper levels, are relatively stiff and incompressible, and of low permeability.

RESULTS OF MONITORING

Ground Movements

Fig. 4(a) shows the development of surface settlement as the tunnel passed the line of settlement points PG. Following Peck's (1969) suggestion, a normal distribution curve has been fitted to field plots when the heading was well past. Yoshikoshi et al. (1980) noted that the same curve fit could be applied to transitional surface settlement profiles, to which similar values of standard deviation would be applicable. The results in Figs. 4(a) and 4(b) appear to bear out their observation. Furthermore, the same conditions apparently hold with depth in the upper marine clay as the settlement plate and deep settlement gauge plots of Figs. 4(c) and 4(d) and show.



(a) Settlement Points PG

(b) Settlement Points S

(c) Settlement Plates SP

(d) Deep Settlement Gauges

Fig. 4 Lateral Surface Settlement Profiles

Fig. 5 shows an error function fitted to the longitudinal surface settlement field plots with the heading well past, following a suggestion by Yoshikoshi et al. (1980). As also noted by them, the standard deviation

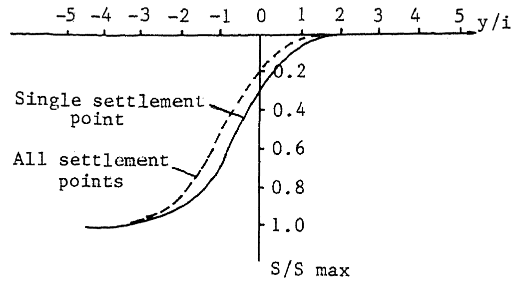


Fig. 5 Longtudinal Surface Settlement Profiles

obtained is fairly close to that for the lateral profile. Slight non-uniformity of ground conditions is reflected by a similar plot in Fig. 5 for a single surface point, for which the error function was generated with the passage of the heading. It was also found that the same curve fit may be applied with depth in the upper marine clay.

Pore Pressure

Fig 6 shows the variation of pore pressure in piezometers P3 and P2, which are located on and 3.6 m away from the centreline of the tunnel respectively. Measurements in P3 are recorded for positions of heading approaching the

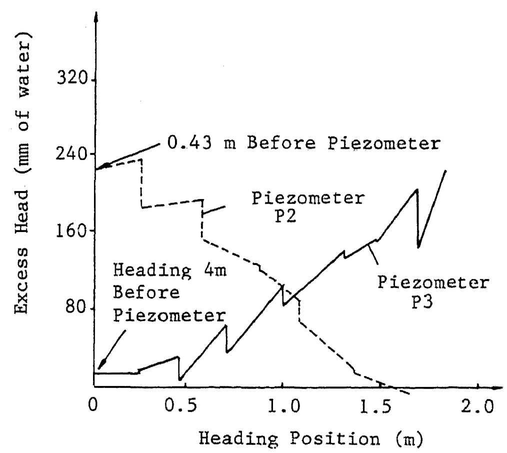


Fig. 6 Pore Pressure During Tunnelling

piezometer from a distance of 4 m. The increase in pore pressure during jacking operation and the subsequent decrease when the heading stops for mucking are quite apparent in the case of P3. For piezometer P2 the pore pressure record shown is when the heading is from a distance of 0.43 m from section "Z-Z". As the heading crosses the section, pore pressure begins to decrease with jacking, illustrating a decreasing mean total stress.

Total Pressure on Lining

A total of 12 total pressure cells were installed on the timber lining. They were made by strain gauging a thin aluminium diaphragm. Cell Nos 7 and 11 which were set with araldite in timber cavities recorded jacking pressure in addition and hence their readings were discarded. All other cells set in bitumen filled cavities indicated a pressure distribution in the four linings as shown in Fig. 2. The distribution on the upper lining shows a higher pressure at the centre, which reflects the reduction of total pressure at the sides due to arching. On the lower lining the pressure is more at the two ends. This is in accordance with the high contact pressure observed under rigid bases in soft clay. In addition, the two ends of the lower lining are bound to settle more than the centre due to the transmission of the overburden soil load through the vertical linings.

The two vertical linings show unsymmetrical pressure distributions, at the upper ends the intensity of the pressure on 20 May 1981 being nearly the same at 42 kPa, and at the lower ends the pressures being 35 kPa and 40 kPa. The largest pressure occurred at the centre of one lining. These observations can be partly ascribed to readjustment of the timber linings, both in rotation and translation.

CONCLUDING REMARKS

Following Peck's (1969) suggestion, the site relationship between i/R and z/2R has been plotted in Fig. 7, superimposed on that established for other sites in Singapore marine clay (Lo, 1982). The performance of the site under consideration falls in the very soft clay range. The corresponding plots for percentage ground loss against overload factor are shown in Fig. 8, based on a proposal by Yoshikoshi et al (1980). The results appear to compare well with the relationship suggested by Peck et al (1969). Thus, by means of Figs. 7 and 8, and assuming the staged development of settlements defined by Figs. 4 and 5, it should, at least

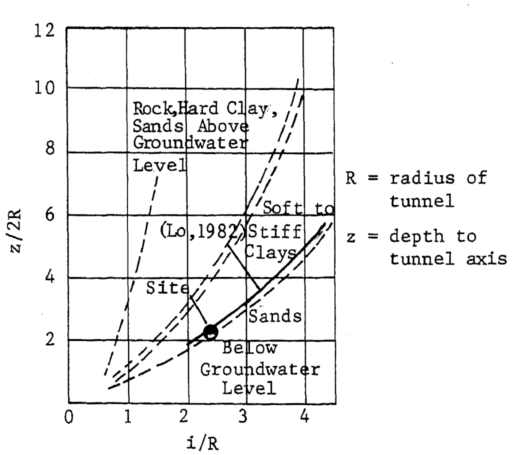


Fig. 7 Relations Between z/2R And i/R

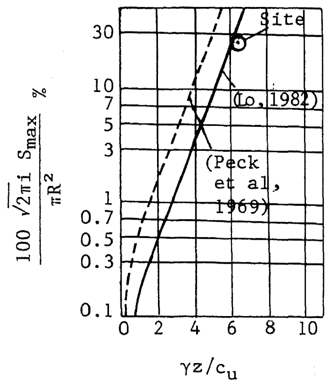


Fig. 8 Relations Between Percentage Ground Loss And Overload Factor

in principle, be possible to construct the entire settlement trough for various diameters and depths of tunnels in Singapore marine clay. Details of lateral movements and face take mechanism, and their parametric studies, are beyond the scope of this paper and will be reported elsewhere.

ACKNOWLEDGEMENT

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