Ground Movements and Pore Pressure Variation Caused by EPB Shield Tunneling - Shanghai (China) Sewage Tunnel

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SYNOPSIS The observed pore pressures and deformations induced by an earth balance shield used to construct the Furongjiang sewer tunnel in soft saturated ground in Shanghai, China, are briefly described. Continuous records of immediate surface settlements and ground movements were obtained as the tunnel face approaching and moving away from the observation points. A correlation between resulting heave/settlements and driving force is observed. A finite element technique for predicting these pore pressures and deformations is developed. A comparison is then made between numerical results and the results of field measurements and it is shown that there is encouraging agreement between the calculated and observed response.

INTRODUCTION

Rapid growth in urban development has resulted in increased demand for the construction of water supply, sewage disposal and transportation systems. Economic and environmental considerations favour the location of the major portion of these facilities underground. Tunnels are an essential component of these schemes and constitute a major portion of project expenditure.

Over the past decade, innovations have been made in soil tunnelling shield machines (eg. slurry, earth pressure balance and mud pressurized shields) which allow application of continuous support to the face of the tunnel. However, because of the novelty of the technology, it is not known exactly what degree of ground control can be achieved using the new machines over conventional ones. The problems posed by the new tunnelling technique necessitates detailed investigation into the understanding of the response of the ground in and around the tunnelling shield.

In 1986, a sewage tunnel was constructed in Furongjiang, located in the northwest of the city of Shanghai, China. This tunnel was constructed using Earth Pressure Balanced (EPB) tunnel-boring machine together with a segmented, precast concrete tunnel lining. Since this was the first time that this construction technique had been used in China, an extensive field-instrumentation program was undertaken. The objective of this present paper is to discussed the observed behaviour of this well-documented case history and to illustrate the applicability of a finite element model to simulate this tunnelling process and the associated ground deformation and pore water pressure behaviour.

THE FURONGJIANG TUNNEL

A 5738 mm long and 4330 mm diameter Earth Balance Pressure Shield, known as a "blind" shield, was used to excavate the Furongjiang sewer tunnel. In its tail there was a space which can cover two rings of the lining. The screw conveyor, 600 mm in diameter, was installed at the lower position of the front face. The rates of soil excavation were controlled by the rotating speed of the screw conveyor. The machine was driven by 12 jacks, giving a maximum total push force of 23.5 mN. Two earth pressure cells were installed directly on the front face for monitoring the front resistance pressure. This pressure was kept balanced with (or a little larger than) the predicted at-rest lateral earth pressure by adjusting the shield advance rate, the total push force and the soil excavation rate.

The lining was of 4.2 m outside diameter and consisted of 4 reinforced segments of 0.7 meters wide and 0.5 meter thick with a grout hole in the middle. The mined diameter was 4.33 m and the physical gap clearance was 0.13 m which was the difference between the concrete lining and the excavated surface.

The tunnel was driven in three shifts. In each shift the shield advanced about two rings. In each ring length it paused three times for transporting the excavated soil. In each shove it advanced about 230 mm, with 4 m^3 soil being excavated. The normally adopted push force was about 5 mN. The rotating speed of the screw conveyor was about 12 rpm.

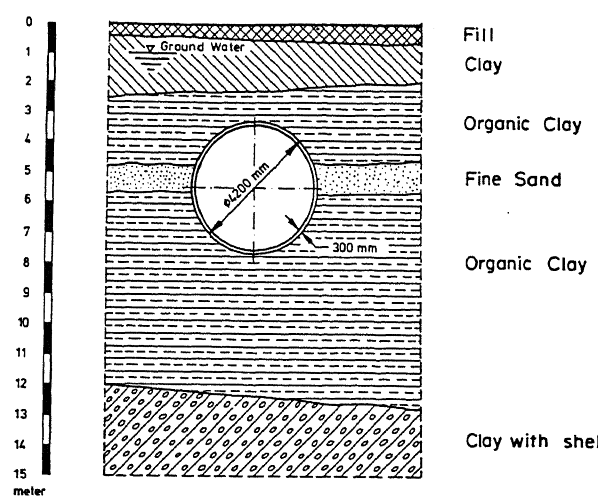


Fig. 1 The Stratigraphy and the tunnel position.

The stratigraphy and the tunnel position in the instrumented section are shown in Fig. 1. The soil properties and description is given in Table 1. The depth from surface to the tunnel axis was 5.6 m, giving a diameter ratio of 1.3. The tunnel was located in a saturated organic clay with a thick layer of 0.9 m fine sand at the tunnel springline. The ground water table was 0.8 m below the surface.

Table 1 Soil Properties

Depth (m)	0.0 to 2.5	2.5 to 4.3	4.3 to 5.2	5.2 to 12.0	12.0 to 17.0
Water content W (%)	40.0	47.1	29.2	50.9	45.2
Total unit wieght γ (kN/m^3)	18.3	17.7	19.2	17.2	17.5
Dry unit weight γ_d (kN/m^3)	13.1	12.0	14.9	11.5	12.8
Void ratio e	1.11	1.28	0.81	1.40	1.29
Degree of saturation S_r (%)	99.5	100.0	97.0	99.5	97.6
Specific gravity G_s	2.76	2.74	2.69	2.74	2.74
Liquid limit W_L (%)	44.6	40.2	-	40.3	41.6
Plastic limit W_P (%)	22.8	20.2	-	22.0	23.1
Plastic index I_P	21.8	20.0	-	18.3	18.4
Liquidity index I_L	0.79	1.35	-	1.58	1.27
Coefficient of Compressibility a_v (kpa^{-1})	0.0005	0.001	-	0.0011	0.0007

Depth (m)	Description of soils
0.0 - 2.5	clay, brownish yellow, saturated, medium plastic, with fine sand.
2.5 - 4.3	organic clay, grey, saturated, very soft plastic clay with fine sand.
4.3 - 5.2	fine sand, grey, with fine horizontal sandwich layers.
5.2 - 12.0	organic clay, grey, saturated, with fine sand and shell.
12.0 - 17.0	clay, dark grey, with shell.

The measurement scheme consists of the following three aspects:

i) Piezometers were installed to monitor the continuous variation of pore water pressure induced by different stages of construction procedures, such as shield driving, soil delivery, tail leaving and lining installation, back filling and grouting.

ii) Telescopic tube settlement gauges were installed in a cross-section perpendicular to the direction of tunnelling to observe the transverse settlement profiles at different depths. The measured settlements were then compared with the obtained pore pressure to identify the proportion of immediate and consolidation settlements at different depths.

iii) Observation points were installed on the ground surface to measure the surface settlements during the shield passage.

A plan and elevation showing the instrumentation are shown in Fig. 2 and Fig. 3 respectively.

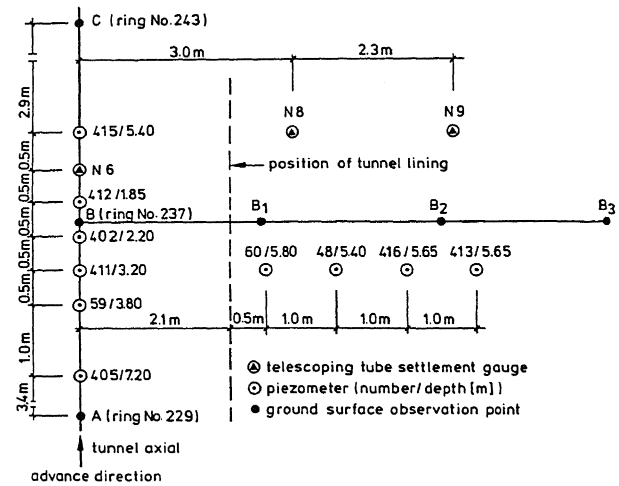


Fig. 2 Position of the instruments.

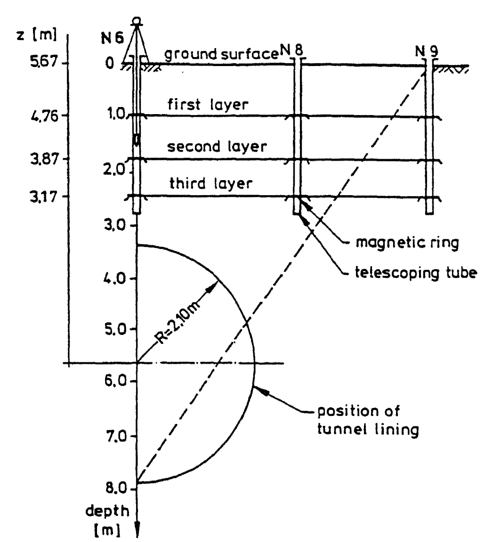


Fig. 3 Profile of the settlement gauges.

RESULTS OF FIELD MEASUREMENT

Excess pore pressure

The progress of the shield face and the observed excess pore pressures along the horizontal direction at the cross section are shown in Fig. 4. Curve 1 shows that the excess pore pressures were zero when the shield face was at the position of 13.3 m from the instrumentation section. When the shield face was at position 2, 7.7 m from the instrumented section, the excess pore pressures were negative, -11.7 kPa at Piezometer No. 60 (which is at a location about 1.5 m from the tunnel axis) as shown by curve 2. When the shield face approached the instrumented section, excess pore pressures began to rise, as shown by curve 4. However the shield were allowed to stop at position 4 for three days. Drainage into the tailpiece were occurred and therefore the excess pore pressures near the lining were smaller than those farther away from the lining. When the shield was at position 5 (where its face was just at the instrumented section), the excess pore pressures decreased markedly (curve 5) because, at this time, the rate of soil excavation was faster than that of the shield advance. By the time the face had advanced to 0.7 m beyond the instrumented section (at position 6), the excess pore pressures had increased substantially and reached their maximum value of 41.0 kPa (curve 6), as a result of the increased face pressure which had been applied between position 5 and 6. When shield tail had just left the instrumented section and the face was at position 7, the closure of the gap between shield cylinder and lining caused the excess pore pressures to drop quickly, as shown by curve 7. Curve 8 shows the distribution of the excess pore pressures caused by tailpiece grouting. Curve 9 shows that the excess pore pressures began to dissipate and curve 10 shows that the excess pore pressures were almost completely dissipated after 35 days.

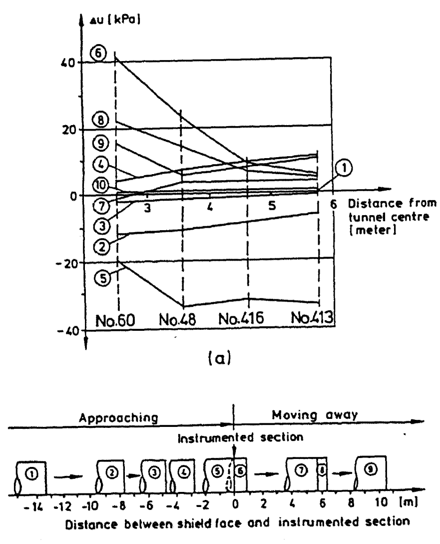


Fig. 4 (a) Observed excess pore pressure distributions along horizontal direction at the instrumented cross section versus shield face position; (b) Corresponding shield face position.

Vertical displacement

Fig 5(a) shows the observed vertical displacements on the ground surface and Fig. 5(b) the corresponding positions of the shield face. The vertical displacements at different depths versus the shield face positions are shown in Fig. 6(a), (b), (c) and (d) respectively. The maximum heaves, shown by curve 4 in Fig. 5(a) and curve 6 in Fig. 6(a), (b) and (c), occurred after the shield face had advanced 0.7 to 2.1 m beyond the instrumented section, i.e., the maximum heave occurred behind the shield face, demonstrating the control of the face pressure had been sufficient to prevent excessive heaving ahead of the tunnel face (see curves 1 and 2 in Fig. 5a).

The immediate settlements when the shield tail just left the instrumented section are shown by curve 5 in Fig. 5 and curve 7 in Fig. 6(a), (b) and (c). These settlements were mainly induced by the inward movement of the ground into the tailpiece void.

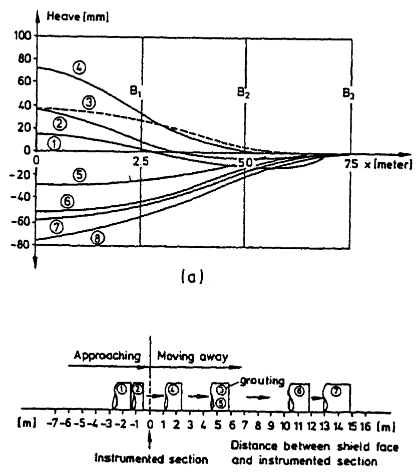
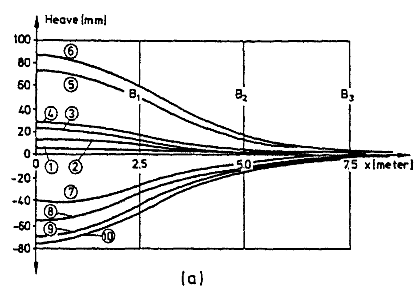
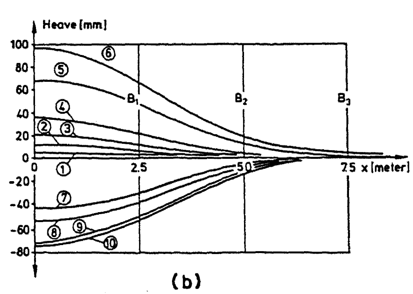


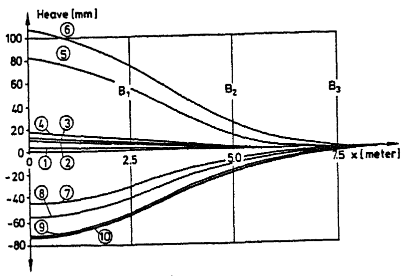
Fig. 5 (a) Vertical surface displacements at cross section versus shield face position; (b) Corresponding shield positions.



(a) Vertical displacements at the first layer

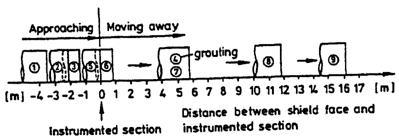


(b) Vertical displacements at the second layer



(c)

(c) Vertical displacements at the third layer



(d) Coresponding shield face positions.

Fig. 6 Observed vertical displacements at different layers and the corresponding shield face positions.

Consolidation Settlements

Consolidation settlement is related to the thickness of the soil layer. In general term, the thicker the layer, the larger the consolidation settlement. At the ground surface, the thickness (from ground surface to the crown of the lining, i.e., the overburden) was 3.5 m. The consolidation settlement of the ground surface, S_c=46 mm, may be added to the immediate settlement, S_i=29 mm, to give the total settlement $S=S_i+S_c$=75 mm. Thus S_c was 61% of the total settlement while S_i was 39%. At the third instrumented layer, which was one meter above the lining, the corresponding values were S_c=30 mm, S_i=45 mm, $S=S_i+S_c$=75 mm, and so S_c was 40% of the total settlement while S_i was 60%. Thus, although the immediate and consolidation settlements at these two different depths were different, the total settlement, S_i+S_c, was the same (after the excess pore pressures had dissipated).

The observed immediate and consolidation settlements at the tunnel centre versus relative depth are shown in Fig. 7. It can be seen that the immediate settlement decreased and the consolidation settlement increased with the distance from the tunnel axis, as shown by curve 1 and curve 2 respectively. These two curves are symmetric about the lines $s=(S_i+S_c)/2$. Curve 3 gives a superposition of the immediate and consolidation settlements.

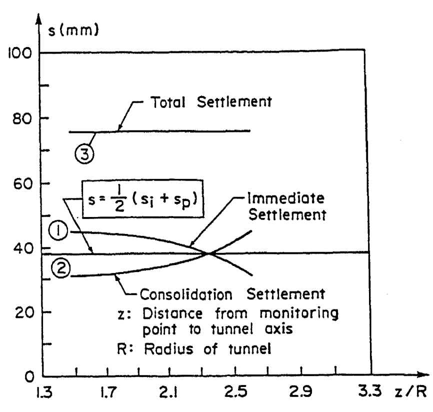
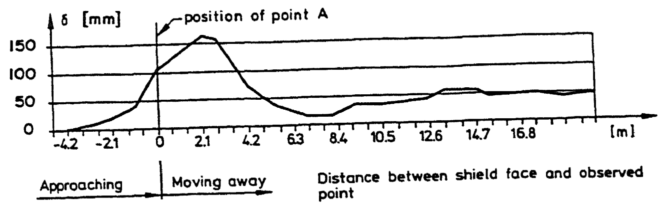


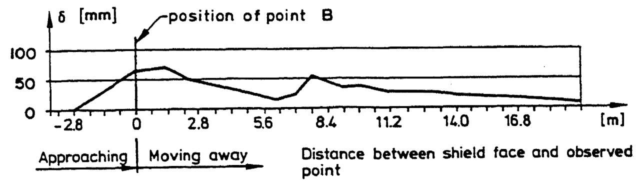
Fig. 7 Relationship between settlements and Z/R.

Relationship between face resistance and vertical displacement

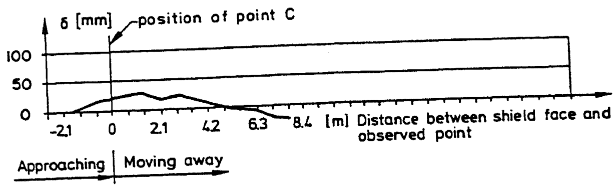
Fig. 8 (a), (b) and (c) show the vertical displacements measured at surface settlement pins A, B and C along the longitudinal axis, at the locations shown in Fig. 2.



(a) Vertical displacements at PIN A.



(b) Vertical displacements at PIN B



(c) Vertical displacments at PIN C

Fig. 8 Observed vertical displacements when shield face approaching and moving away from the observed points.

The readings of the pressure cell in front of the shield face, i.e., the face pressure at the position of the pressure cell and the corresponding heave are shown in Table 2. The predicted at-rest lateral earth pressure P_s at the position of the pressure cell was about 60 kPa, which was estimated by $P_s=K_o\gamma h$ with value of K_o=0.7, γ=18.4 kN/m^3 and h=4.665 m (here γ is the total unit weight and h the depth of the pressure cell).

Table 2 Face pressure and ground heave

Observed position	Pressure (kPa)	Ground Heave (mm)	Corresponding Figure
PIN A	95.9	156	Fig. 8(a)
PIN B	78.6	71	Fig. 8(b)
PIN C	68.5	29	Fig. 8(c)

It can be seen that at PIN A the front earth pressure was relatively large and therefore a larger heave (of 156 mm) occurred. After a period of time, there was still 50 mm heave on the surface as shown in Fig. 8(a). At PIN C the front earth pressure was relatively small, after the shield had passed the observed point, transient settlement occurred as shown in Fig. 8(c). At PIN B the earth pressure was 78.6 kPa, 30% larger than P_z and the corresponding maximum surface heave was 71 mm. After a period of time, the heaved ground surface tended to return to its initial position. This result suggests that intentionally inducing a small amount of heave can reduce the final surface settlements. Similar results were obtained by Clough et al. (1983) and Finno & Clough (1985). It is found that the magnitude of the initial heave correlates directly with the level of the earth pressure measured inside the soil retaining area in the shield. Intentionally inducing a small amount of heave with an EPB shield caused smaller surface settlements than those without inducing any initial heave. Therefore it is suggested to operate an EPB shield in this manner to help minimize the surface settlements.

RESULTS OF ANALYSIS

The analyses reported in the present paper were performed using the author's coupled viscoplastic-consolidation finite-element program. Details regarding this form of the soil model, the construction simulation, and solution technique have been described by Yi (1991) and will not be repeated herein.

Ground deformations

The resulting calculated ground surface settlements and horizontal displacements at different times are shown in Figs. 9 and 10 respectively. The immediate surface settlement is shown by curve No.1. The maximum calculated centre-line settlement (δ_{max}) is 100 mm. Based on Peck's formula (Peck, 1969) the settlement at the inflection point of the settlement curve is equal to $0.61\delta_{max}$ and at the point of maximum curvature $0.22\delta_{max}$. These two points are indicated by A and B in Fig. 9. It can be found that the settlements at these two points are very close to the values that would be predicted based on Peck (1969). It is also found that surface settlements increase with the dissipation of excess pore pressures. The maximum time-dependent settlements at time=102 day has increased by about 20% of the immediate settlements. The maximum horizontal displacements of ground surface occur at the position of 4 meters from the centre line (approximately at a distant of one tunnel diameter from the centre-line axis as shown in Fig. 10). It is therefore suggested that this position is a key location for the observation of horizontal ground deformations.

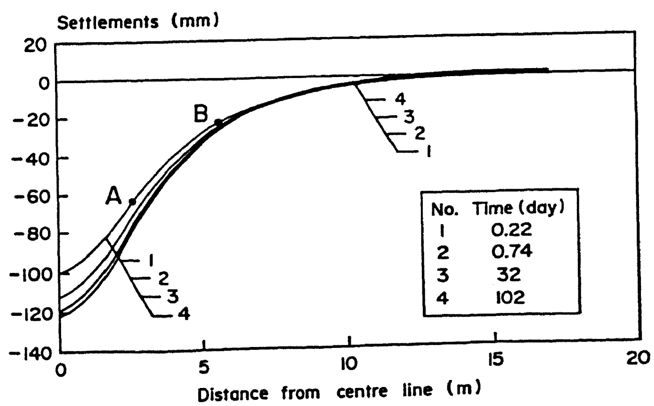


Fig. 9 Calculated surface settlements at different times.

The effect of the disturbed zone due to shield advance

During the shield passage, local stresses induced by advancing the shield will create a zone of disturbed soil around the tunnel. If the soil parameters which were obtained using undisturbed samples are adopted for the entire area in the finite element analysis the settlements will be underestimated. The extent of the disturbed zone will depend, in part, upon the workmanship and construction factors. With the advance of the shield, soil in front of the shield face will move both radially and axially towards the face. The effect of shield advance on ground disturbance and the movements of the soil ahead of the tunnelling machine are evidently three-dimensional. During the shield advance, a field investigation was carried out at the same time. Pore pressure cells were buried in the front and at the side of the shield. Based on the results of excess pore pressure changes (as shown in Fig. 4), the influence distance was estimated to be about 4.6 m from the tunnel centre and hence in the analysis, a disturbed zone around the tunnel is modelled, as shown in Fig. 11. In the zone 4, the strength of the soil was reduced to 25% of the original strength based on Huang (1984).

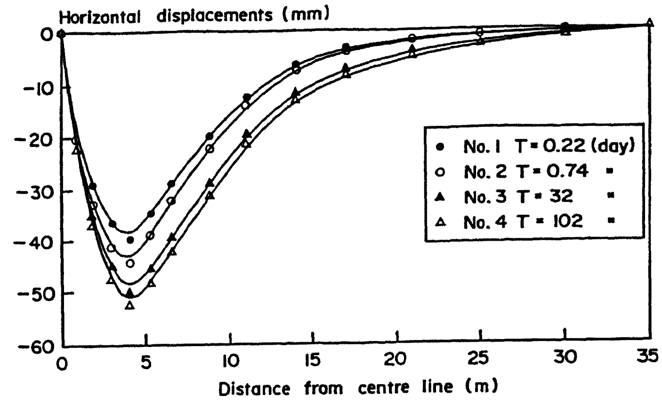


Fig. 10 Calculated surface horizontal displacements at different times.

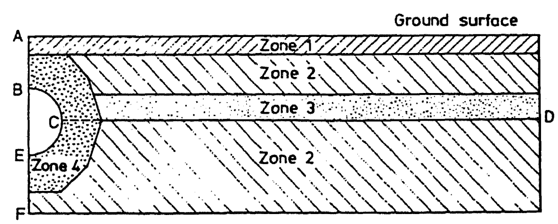


Fig. 11 Disturbed zone around the tunnel.

The calculated surface settlements and horizontal deformations are shown in Figs. 12 and 13 respectively. Comparing them with Figs. 9 and 10, it is found that the immediate settlement and horizontal displacement are almost the same and the maximum time-dependent settlements and horizontal displacements have increased about 30% and 40% respectively because of the disturbance of soil around the tunnel. It is reasonable that the disturbance of ground only affects the time-dependent deformations.

The measured and calculated settlements are shown in Fig. 12. The calculated immediate settlement (curve No.1) is in good agreement with that measured (curve No.4). The calculated settlement at time= 32 day (curve No.3) is slightly larger than that measured at time +35 day (curve No.5). Since the shapes of the calculated and measured curves are similar, the difference would appear to be a result of the difference between the laboratory and field coefficients of permeability and soil parameters.

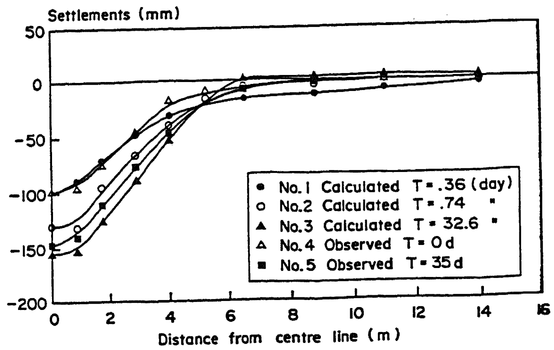


Fig. 12 Comparison between the measured and the calculated surface settlements.

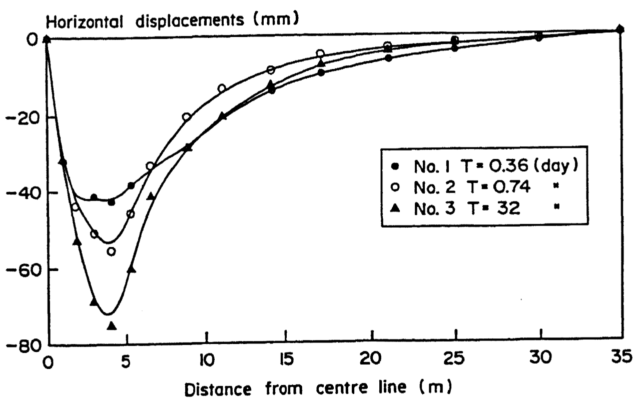


Fig. 13 Calculated surface horizontal displacements at different times.

CONCLUSIONS

The observed pore pressures and deformations induced by an earth balance shield used to construct the Furongjiang sewer tunnel in soft saturated ground in Shanghai, China are briefly described. A finite element technique based on a coupled viscoplastic-consolidation analysis for predicting pore pressures and deformation is adopted. Based on the results of field observations and finite element analysis, a number of conclusions may be drawn as following:

1) It is found that the development of excess pore pressure is highly dependent upon the construction activities and the relative rate of shield advancement as compared to soil delivery rate. If the soil delivery rate is relatively slow so that high face pressure is allowed to develop ahead of the tunnel face, excessive ground heave and positive pore pressure will develop.

2) Intentionally inducing a small amount of heave with an EPB shield will result in smaller surface settlement. It is therefore suggested to operating an EPB shield so that it will induce a small amount of ground heave.

3) Based on the measured results, the face pressure about 30% larger than the predicted at-rest lateral pressure appeared to provide optimal results.

4) Based on the measured pore pressures, it can be shown that the tunnel advance rate should be kept at an uniform rate in order to minimize ground disturbance and subsequent consolidation settlement.

5) The technique for modelling coupled viscoplastic-consolidation provided encouraging agreement with the observed excess pore pressure field as well as the immediate and time-dependent deformations induced by shield tunnelling in soft ground.

6) The calculated immediate settlement trough and the key points on the settlement curve are very close to the classical shape of the settlement trough proposed by Peck (1969). The shape of the normalized settlement curve becomes narrower with time. The shape of the observed and calculated settlements were in reasonable agreement. The maximum horizontal displacements of the ground surface at different times occur at a horizontal distant about one tunnel's diameter from the tunnel centre.

7) The effect of partial disturbance around the tunnel is simulated by reducing the parameters related to the soil strength in an assumed zone. The calculated maximum settlement increased by about 30% and the maximum horizontal displacement increased by about 42% when compared with the results calculated without considering the disturbed zone. The excess pore pressures are much larger than the results without considering the disturbed zone.

ACKNOWLEDGEMENT

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