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THE DISPLACEMENT OF A DIAPHRAGM WALL IN LONDON CLAY

LE DEPLACEMENT D'UN MUR-DIAPHRAGME DANS L'ARGILE LONDONIEN

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SYNOPSIS: The deformation behaviour of a diaphragm wall in London has been investigated. Observations have been carried out on the deflection of the diaphragm wall and the prop loads in the struts. Stress path testing has been employed to determine K_o and the soil strength parameters. Numerical analyses have been performed to investigate the relationship between the soil parameters and the field data.

INTRODUCTION

A deep excavation, supported by a diaphragm wall, close to the London Underground station at Piccadilly Circus was carried out in early 1990. A plan view of the location of the excavation(about 40x40m^2) and the diaphragm wall is shown in Fig. 1a where three tunnels lie beneath and adjacent to the diaphragm wall at different depths. The diaphragm wall was installed from within the existing basement to 18m depth below ground level prior to excavation. Temporary steel struts were installed in longitudinal and transverse directions to prop the top of the diaphragm wall. Excavation was carried out to a depth of 8m from the basement into the London Clay. There are 5 vertical shafts within the excavation area. Fig. 1b shows section A-A through the excavation and the Piccadilly tube line tunnel located 14m below the toe of the diaphragm wall. Fig. 1c shows section B-B through the excavation and the two Bakerloo tube line tunnels located 7m below the toe of the wall.

Five inclinometer tubes and four pairs of vibrating wire strain gauges were installed in the wall and on the struts respectively. These were monitored for a period of about eight months from the beginning of excavation to the completion of the basement works. The deflection of the wall and load on the props were observed and analysed to determine the movement as the excavation took place.

The stress-strain behaviour of London Clay was investigated using stress path testing of undisturbed samples taken from the excavation. Coefficient of earth pressure at rest K_o and Young's moduli corresponding to appropriate stress paths were determined. The estimation of the distribution of K_o with depth has been made from a few K_o consolidation tests.

The numerical analysis has been carried out using the boundary element method(BEM), the finite element method(CRISP, Britto & Gunn, 1987) and a coupled method(LAWWALL, Wood, 1984) with elastic model and Mohr-Coulomb failure criterion. It has concentrated on the parameter evaluation, basic assumption and interpretation of numerical results.

OBSERVATION

The observation of the deformation of the wall was carried out using inclinometer tubes. Temporary steel struts were installed at the top of the wall after a 1m excavation, and strain gauges were attached to them at the locations shown in Fig. 1. The progress of excavation and construction

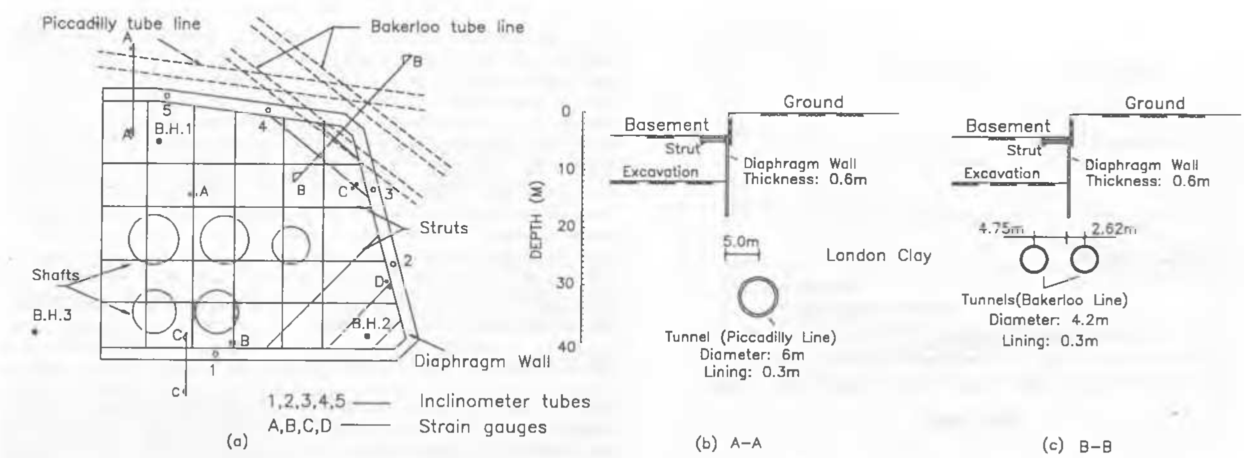

Fig. 1. Excavation at Piccadilly Circus

levels is presented in Fig. 2. The excavation was completed in June 1990 and foundation construction followed. The struts were removed when the new construction reached ground level in July 1990, when the monitoring work was terminated.

Measurement of Strut Loads

The measurement of strain gauges began in Feb. 1990 and ended in July 1990. The strains obtained from each gauge were calculated using the zero strain readings obtained on removal of the struts. Due to difficulty in accessing the strain gauges during the excavation and subsequent construction, there were no zero strain readings for strain gauge 'D'. Referring to the strut positions and strain gauges the prop loads from the pairs of gauges 'A' and 'B' in Table 1 can be used for the calculation of the performance of the retaining wall. The mean prop force has been calculated to be 849 kN at each strut perpendicular to the diaphragm wall. With the struts at 7.5m centres, the distributed line load suggested for the analysis to represent the strutting effect is equivalent to 114 kN/m in plane strain condition.

Table 1. Mean prop loads observed

Gauge	A1	A2	B1	B2
Load(kN)	711.7	783.2	1054.5	846.2

The total deformation of the struts, i.e., the movement at the top of the wall has been calculated from the strain gauge readings. From strain gauge A and B, the total deformation of the 40m long steel strut can be found to be 4.2mm. Considering the strut supporting two sides of the diaphragm wall(with tube 1, and tube 4 and 5 respectively), the wall has a 2.1mm mean inwards movement at the strut position. This movement is the minimum one along the longitudinal direction of the wall compared with the intermediate sections between the struts.

Wall Movement

The deflection of the diaphragm wall was observed by means of a biaxial vertical inclinometer instrument. The first set of readings was obtained on 8th Dec. 1989 as base readings for each tube. Due to an error in installation it was only possible to read tube 4 to a depth of 11m. Deflection of tube 4 below this level has been obtained by extrapolation. Due to the restriction of construction site in central London, there was no survey data to cross check the wall movement. The absolute final deflection for each tube shown in Fig. 3 has been produced by taking account of the 2.1mm deflection of the top of the wall determined from the strain gauge readings. The maximum deflection developed was 15.8mm, 17mm, 12.7mm, 13.7mm and 10.1mm for tubes 1 to 5 respectively. The deflection at 6m(approximating to the maximum deflection) below the top of the wall for each tube is shown in Fig. 2 with time. The rate of increase in deflection reduces as the basement construction proceeds.

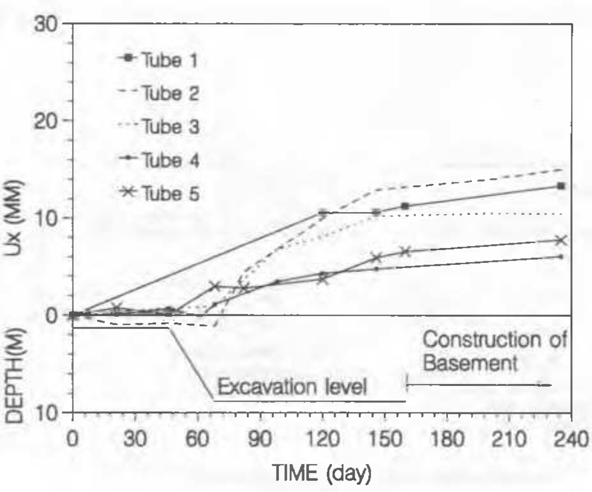

Fig. 2 Excavation and deflection at 6m depth

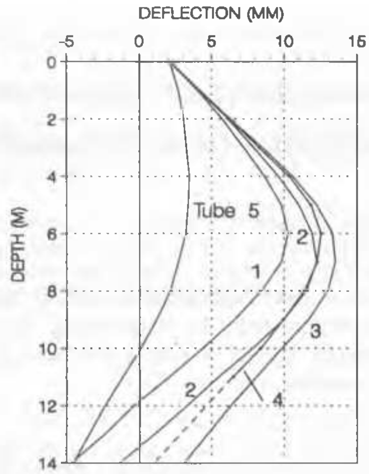

Fig. 3. Observed deflection from all tubes

SOIL PARAMETERS AND STRESS PATH TESTING

Geological Aspect and Pore Water Pressure

The soil strata was typically as shown in Fig. 4. Up to 7m of made ground, comprising dark orange brown slightly clay, sandy gravel, some brick and concrete fragments was underlain by the 27m thick London Clay deposit. The latter is a stiff dark grey and brown fissured silty clay overlying the Woolwich & Reading beds.

Pore water pressures were monitored by piezometers installed in the boreholes. The water table is at 3m below the ground surface. Fig. 4 shows the non-hydraulically static distribution of pore pressure in the period between 8th June 1989 and 31st July 1989, which is thought to be due to long term drainage into the existing shafts. As shown in Fig. 1, borehole 1 is close to section A-A, and therefore the readings from this borehole have been used for the determination of the in-situ stresses for the analysis. For section C-C, neither borehole 1 or 2 is close to the section. Thus a mean pore water pressure from both boreholes has been considered in the calculation of in-situ stresses.

K_o Consolidation

The coefficient of earth pressure at rest K_o has most influence on the determination of the lateral in-situ stresses. K_o determinations were made on three samples from each of two depths. The assumed vertical effective stresses were 333 kPa for 18m depth and 363 kPa for 21m depth. Although there is no obvious distinction between the samples from 18m and 21m depths, the clay from the deeper depth is generally less over-consolidated. From K_o consolidation, the mean value of K_o at 18m depth from three samples is 1.32 and the mean value of K_o at 21m depth is 1.2.

It was not possible due to time constraints to carry out K_o consolidation of samples from more depths in the low permeability London Clay. Mayne and Kulhawy's(1982) and Wroth and Houlsby's(1985) formulae have been used to evaluate the appropriate distribution of K_o by utilising the limited number of K_o test results. Fig. 5 shows the distribution of K_o with depth using both methods based on the K_o value from the samples at a 18m depth with constant ϕ'. Both solutions give values close to each other but the latter shows slightly smaller values of K_o at deeper depths, which agrees with the value of K_o found from the samples at 21m. As London Clay is found between 7 to 34m below ground level, K_o values vary from 1.95 to 1.10 with the former, and from 1.95 to 1.07 with the latter, with constant ϕ'. The practical distribution of K_o applied in the analysis is also shown in Fig. 5 obtained using the different values of ϕ' determined from the samples at 18m and 21m depths.

Stress Paths and Shear Strength

Due to the stress path dependence of soil behaviour, the associated deformation moduli of the clay have been determined for the stress history and variation during the excavation. The stress paths are approximately shown in Fig. 6. The mean Young's Moduli determined from a number of

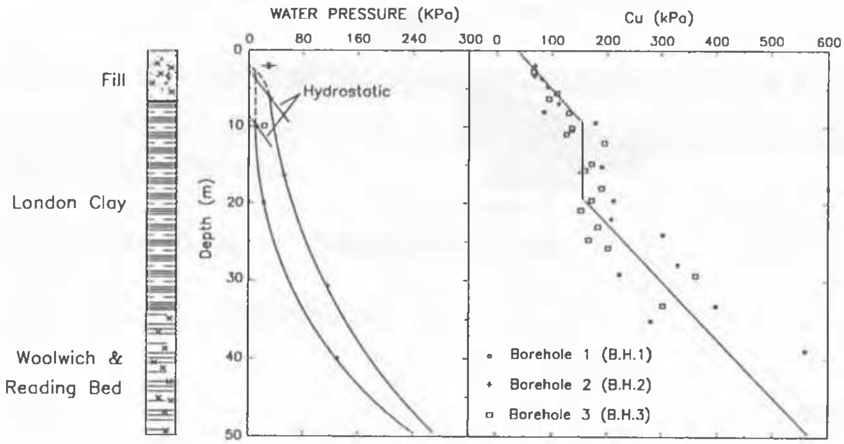

Fig. 4. Pore water pressure and undrained shear strength

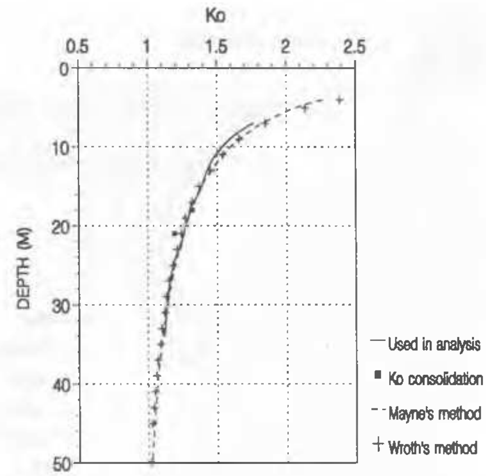

Fig. 5. Distribution of K_o

undrained and drained tests for 18m depth sample are E = 45.2 MPa and E' = 38.7 MPa for the stress range associated with the relief due to the excavation. Due to the low permeability of the London Clay the behaviour during excavation may be characterised as undrained. Under these conditions the angle of effective shear strength has been found to be ϕ'=21.3 in compression and ϕ'=19.5 in extension. The cohesions of the sample are c'=40 kPa in compression, c'=24.8 kPa in extension and C_u=200 kPa.

NUMERICAL ANALYSES AND COMPARISON

Numerical prediction was performed prior to the excavation using FEM(CRISP) and LAWWALL. The back analysis has been carried out later based on stress path testing and site observation with above approaches and a newly developed programme using the boundary element method(Lin & Wood, 1991) where London Clay is treated as a transversely isotropic soil.

Numerical Geometry and Loads

Although this is a three dimensional excavation as shown in Fig. 1, some sections, such as section A-A and C-C, can be approximately modelled in the plane strain condition. Since the retaining wall was installed between 4m to 18m depth from ground level, the soil above 4m depth has been modelled as a uniform surcharge. The foundation is divided into 8 different subregions according to soil profiles and a closed boundary is specified around the half geometry by assuming symmetry. All existing tunnels are modelled with a lining. Although an infinite boundary element mesh can be applied, a closed boundary is preferred for obtaining more precise results for the comparison with FEM. The geometries of section A-A and C-C start from the centre of the excavation and extend to 70m including a 20m width of excavation.

The distribution of K_o found from soil testing is applied for the calculation of horizontal in-situ stress. The in-situ stresses applied in sections A-A and C-C are different due to the different pore water pressure regimes. The uniform surcharge at basement level is assumed to be 78 kPa in accordance with the bulk unit weight of the soil. A 2.1mm horizontal displacement along the longitudinal direction has been applied to represent the effect of the prop in plane strain condition.

Application of Soil Strength Parameters

For London Clay the multi-linear distribution of C_u with depth is shown in Fig. 4. The effective angle ϕ' has been obtained from consolidated undrained testing with pore water pressure measurement. Undrained vertical Young's moduli can be estimated from the empirical relationship of E_v/C_u=200~1000(Butler, 1975). They can be also obtained from triaxial stress path testing directly. In stress path testing, the undrained Young's modulus is about 45.2 MPa at 21m below ground giving a E_v/C_u ratio of 250 for the undrained shear strength of 200 kPa at the same level, see Fig. 4. Therefore the distribution of E_v has been obtained by assuming a constant ratio in the London Clay. Although the ratio of 250 is a reasonable value for over-consolidated London Clay, this relationship is found on the basis of laboratory testing, where the samples undergo different degrees of disturbance. Therefore it may not be a realistic ratio for applying in numerical analysis. In the circumstance without field testing, it is expected a realistic ratio will be evaluated with the comparison of numerical results with field observation. Due to the over-consolidated history, London Clay is treated as transversely isotropic soil in which the horizontal Young's modulus E_h=1.8E_v in the undrained condition. The Possion's ratio in horizontal plane, υ_{hh}=0.1 and between the vertical and horizontal directions, υ_{vh}=0.49. The shear modulus G_{vh} is related to E_v and υ_{vh}.

Linear Analysis

It is a common problem that much larger movements of diaphragm walls are derived in the calculation with the Young's modulus from laboratory testing. The calculation is made by assuming the wall having a 2.1mm movement at the top of the wall on section A-A and C-C. When the wall is analysed with soil stiffness from stress path testing, i.e., E_v/C_u=250, large movements are given by BEM and FEM when compared with those observed. Subsequent calculations have been carried out with the assumption of E_v/C_u=450, 650, and 1000. More satisfactory results are obtained with much higher values of Young's moduli(e.g., E_v=1000C_u) than those from laboratory testing shown in Table 2. Such a high ratio of E_v/C_u implies that the test results are more greatly affected by the magnitude of Young's modulus than that of undrained shear strength due to the disturbance of samples and the associated stress relief. Since the diaphragm wall is taken to be continuous along its longitude direction, it implies that a similar shape of deformation should be presented at prop position to those in tubes 4 and 5. However the results from FEM and BEM show a little larger movement around the toe than those from the

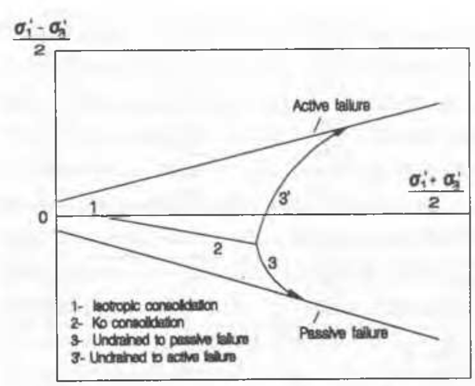

Fig. 6. Stress path of London Clay

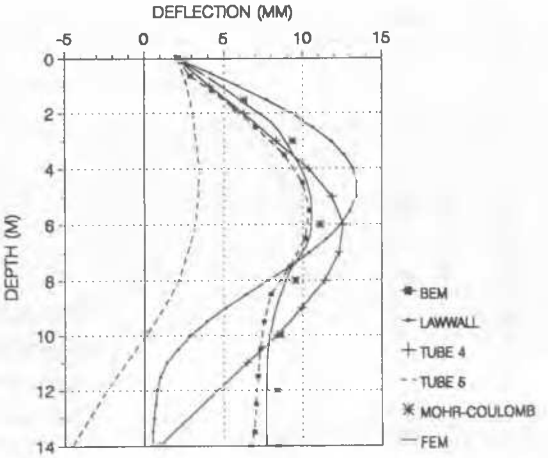

Fig. 7. Wall movement from numerical analysis(E_v/C_u=1000)

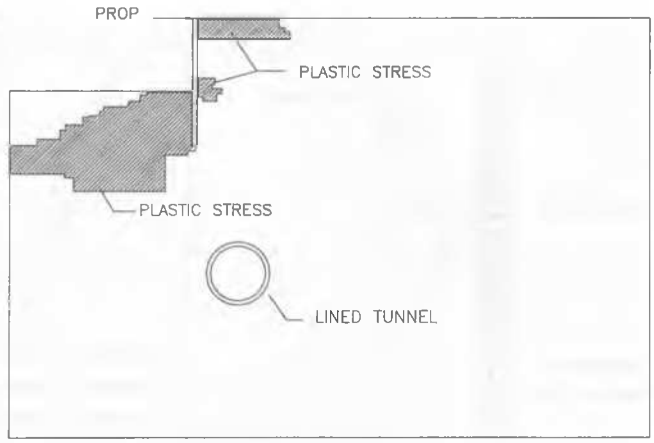

Fig. 8 Plastic zones around diaphragm wall

field measurements shown in Fig. 7. The prop reactions obtained by BEM and FEM are also listed in Table 2.

The results on the section C-C have shown a 0.7mm extra movement on average when compared with those at section A-A. This is due to no tunnel being close to the wall and the differences in the pore water regime. Hence, the effect of the stiffness of the existing tunnels is shown to have little influence on the wall deformation. However, the horizontal movement at the tunnel is reduced from 14.7mm to 5.1mm when the ratio E_v/C_u increases from 250 to 1000.

Elastic-perfectly Plastic Analysis

A Mohr-Coulomb failure criterion has been used in the analyses to represent the behaviour of the London Clay. The development of plastic zones is shown in Fig. 8. The deflection of the wall shown in Fig. 7 is very similar to that computed from the linear analysis. However, somewhat higher strut loads have been computed with the latter. The results obtained from the simpler LAWWALL model are also given in Fig. 7 and Table 2 and exhibit reasonable agreement with the observed values of deflection and strut load.

Table 2. Comparison of analysed results in section A-A

Analysis	Ev/Cu	Maximum deflection(mm)	Strut Load(kN)
FEM (linear)	250	44.6	1021
	450	25.3	783
	650	17.1	629
	1000	10.5	432
BEM (linear)	250	40.3	974
	450	22.7	742
	650	16.3	600
	1000	10.4	424
LAWWALL	1000*	13.7	1012
FEM (Mohr-Coulomb)	650	18	720
	1000	11.1	556
Observed tube 4		13.7	
Observed tube 5		10.1	
Observed gauge A			747
Observed gauge B			950

* E_v is taken equal to E_h

CONCLUSIONS

The monitoring work was successfully carried out using inclinometer tubes and strain gauges. The maximum deflection of the diaphragm wall varied from 10.1mm to 17mm between depths of 4m and 7m below the top of the wall during excavation and construction. It is also seen that the struts play a significant role in limiting the inward movement at the top of the wall to 2.1mm.

The distribution of K_o has been obtained from the combination of test results and empirical formulae to determine the in-situ stress state of the London Clay. The Young's moduli obtained from stress path tests are shown to be between 1/3 and 1/4 of those determined from back analysis of the field measurement.

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