Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
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ubilee Line Extension, Bermondsey Station box: Design modifications, nstrumentation and monitoring

I. P. Dawson & A. R. Douglas
Maunsell & Partners, Beckenham, UK

.F. Linney & M. Friedman
ondon Underground Ltd, UK

.Abraham
letanche Ltd, Alton, UK

BSTRACT: Excavation of Bermondsey station diaphragm wall box was by top down construction. Extensive onitoring was carried out. This was used to ensure compliance with restrictions on movements of adjacent strucres and to enable a simplified construction sequence to be adopted by using the observational method. The structre was back-analysed to confirm actual soil parameters.

INTRODUCTION

ermondsey is one of eleven new stations on the 15km tension to the Jubilee Line which forms part of the ondon Underground Railway network. The station is a omposite structure consisting of twin bored 7m diamer platform tunnels connecting into an open cut ctangular station box, which houses escalators, ticket all and plant rooms. Provision has been made to onstruct a 12-storey building over the station. The chitectural design was carried out by Ian Ritchie ssociates and the structural design of the station by Sir 'illiam Halcrow and Partners. The construction was t to AOKI Soletanche Joint Venture who employed Maunsell & Partners to carry out temporary works esigns.

A large amount of the instrumentation was designed into the works, which enabled the observational technique to be used to monitor the performance of the structure in relation to the contractor's alternative construction sequence. In addition, monitoring installed in the surrounding ground and on structures, provided the control necessary to ensure that building damage remained within contractual limits.

This paper briefly describes the design and construction of the Bermondsey Station box in relation to the geology and soil properties, and describes in more detail the results of the instrumentation and how it was used to monitor the alternative construction sequence.

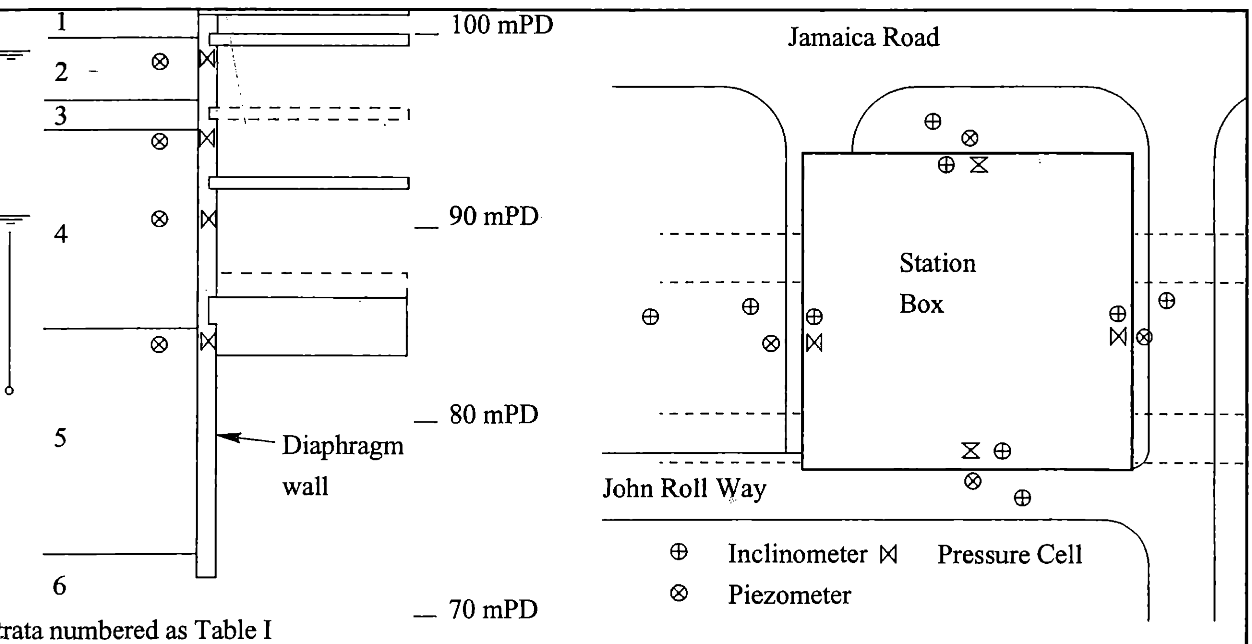

Figure 1: Plan of Site and typical construction

1.1 Background

The Bermondsey Station box is rectangular in plan, measuring 50m by 45m and is bordered on three sides by roads and on the fourth side by two storey structures. A plan of the box is shown in Figure 1. The depth of the box to formation level is approximately 19.5m.

It was a requirement of the contract that no damage should be caused to adjacent structures greater than Degree 2 of the Building Damage Classification in BRE Digest 251.

2 OBSERVATIONAL METHOD

The contractor proposed an alternative construction sequence in an attempt to reduce the excavation time and cost of the work. Maunsell analysed this sequence and demonstrated that it was structurally practical. The original construction sequence was to use the top down method with a temporary support at 90mPD. The revised sequence was to construct the upper plant room truss at 48mPD at the end of excavation and to omit the temporary prop. This arrangement removed several construction stages but increased the unsupported height of the diaphragm wall during construction. The final supports to the wall were the same as for the original design. This change in design was based on the use of the observational method. Best estimates of the soil properties and structural behaviour were made and an alternative strategy for completing the work was agreed if any of the measurements indicated that these assumptions were not adequate.

3 CONSTRUCTION ASPECTS

In considering the actual construction of the wall and in particular plant selection, a number of factors had to be considered:

a) The environmental constraints of the site - the site is surrounded on three sides by residential properties;

b) The fact that the wall forms an architectural feature within the station box and remains exposed;

c) The provision for tunnel boring machines to both enter and exit the station box;

The dig itself was suited to any one of Soletanche's usual range of diaphragm walling equipment - crane mounted hydraulic/short kelly grab (KS 3000), Hydrofraise (excavation using milling techniques linked to a reverse circulation system) and standard rope operated grabs. In conclusion rope operated grabs were adopted on the basis of experience gained in the excavation of similar soils elsewhere in London. The proportion of cohesive materials weighed against the use of Hydrofraise.

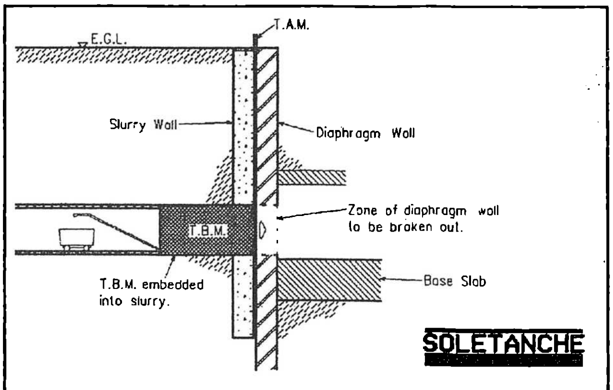

Figure 2: TBM Break-in detail

The working hours for the site were strictly controlled in view of its location and working until late in the evening in order to extract stop-ends was obviously going to be viewed with some concern. It was therefore proposed that the *Stopsol* system of stop-ends be used. For this system the stop-end does not have to be pulled out as the concrete sets but is peeled away as the secondary panel is excavated at a later date.

The entry and exit of the TBMs into the structures is always a critical activity, particularly when this is to occur below the water table in granular material.

Having considered various techniques including grouting, jet grouting and de-watering, it was considered that diaphragm walling techniques would be used. The design for break-ins was simple enough - a slurry wall panel would be constructed immediately outside the diaphragm wall such that the TBM could embed itself into the slurry thus effectively sealing it to the wall prior to the wall being broken out from the inside to allow passage of the TBM through the wall. The arrangement is shown in Figure 2 and a photograph of the TBM in Figure 3.

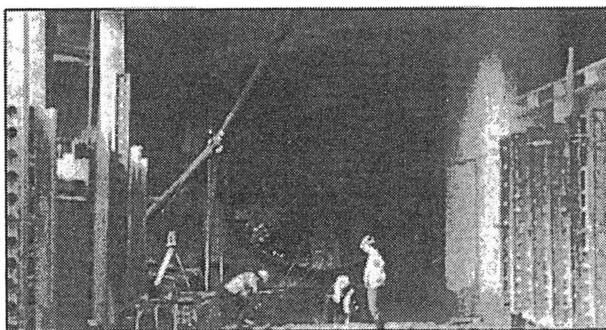

Figure 3: Photograph of TBM Break-in

The break-outs required more thought as there exists the temporary condition whereby the wall has to be broken out without having the support from the TBM, but the TBM then has to pass through. For this condition, the slurry wall was constructed, as for the break-ins but inserted into the slurry and positioned against the outside face of the wall were sheet piles. These were greased to limit bond to the slurry and will be extracted once the TBM is sealed into the wall and is providing pressure to balance that from the soil.

4 SOIL PROPERTIES AND GEOLOGY

4.1 Geology

The ground conditions at the site of Bermondsey Station box consist of approximately 3m of Made Ground and Alluvium, overlying 4m of Terrace Gravel which in turn overlies 1m of London Clay. Beneath the London Clay are the predominantly cohesive soils of the Woolwich and Reading Beds, which are approximately 10.7m in thickness, and beneath which lie 8m of granular Woolwich and Reading Beds. Beneath these lie the Thanet Sands. A profile of the geological sequence is shown in Figure 1. A more detailed description of the nature of these soils is given in the paper on Site Investigation for the JLE, which is being presented at this Symposium. (Linney and Page, 1996).

Groundwater is present within the Terrace Gravel at a level of about 96mPD, and also within the granular soils of the Woolwich and Reading Beds and Thanet Sands. Piezometers installed in the lower granular soils recorded water levels approximately 89mPD.

4.2 Soil Properties

The soil strains behind the wall were originally estimated at greater than 0.1% of the excavated height. However for a braced excavation effectively restrained within the granular layers of the Woolwich and Reading Beds and Thanet Beds the strains were expected to be less than 0.1% of the excavated height. By applying the observational technique it was considered possible to adopt a strain level of 0.1%.

The soil stiffness values for the London Clay and cohesive layers of the Woolwich and Reading Beds were estimated at 0.1% strain from the small strain triaxial test results. No laboratory stiffness testing was available for the lower granular strata so design stiffness values were derived from pressuremeter testing, see Table I Soil Properties.

It was assumed that in the temporary case the cohesive soils would act in an undrained mode, and the C_u values in Table I for the cohesive strata were kept constant with depth. In the event of the soils becoming drained the wall was still designed to have an adequate factor of safety.

5 INSTRUMENTATION

The works contract provided for a considerable amount of instrumentation in and around the station box. This was intended to monitor compliance with controls on adjacent structural movements. It was also used to validate the assumptions made for the analysis of the wall. Ten torpedo read inclinometers were installed, one in each face of the diaphragm walls, four in the ground behind the walls and two to the east of the box. These last two also included ring magnet extensometers. The positions of these instruments is shown on the site plan (Figure 1). Soil pressure cells were installed at four levels on each side of the box and piezometers in the adjacent ground. These were used to assess the soil and water pressures during construction. Total and effective pressures were derived from this instrumentation. A total of 168 strain gauges were installed in the trusses. Ground surface and building settlements outside the box were recorded by precise levelling and electrolevels.

The measured movements of the wall were compared to those predicted by the analysis and the wall was back-analysed to estimate the actual values of various parameters. Had the deflections exceeded the trigger values then a temporary prop was to be installed as proposed in the original design.

The data was transferred from site to Maunsell's design office on floppy disks. This established a reliable method of transferring the data. A suitable protocol for

Table I: Soil Properties

	1 Made Ground/ Alluvium	2 Terrace Gravel	3 London Clay	4 Woolwich & Reading Cohesive	5 Woolwich & Reading Granular	6 Thanet Sands
γ (kN/m^3)	18	20	20	20	20	20
ϕ'	25	35▲	25	30	35▲	40▲
c' (kPa)	0	0	10	10	0	0
E' (MPa)	6.3	50	45	90	300	300
$K_{0\ (installed)}$	0.6	0.4	1.0	1.0	1.0	1.0
c_u	0	0	100	180	0	0
Revised E' (MPa)				Upper Beds 70 Lower Beds 500	500	500
▲ Triaxial value - higher values can be used in plane strain with corresponding changes to other parameters						

transmitting the data via e-mail could not be agreed in the short start-up time of the monitoring contract and would have been new to both parties' quality assurance systems leading to administrative difficulties. The computer resources employed to handle the data were modest by today's standards but was considered adequate. The ease of transmitting data and the computer power available have made this type of detailed monitoring practical.

6 SOIL STRUCTURE INTERACTION ANALYSIS

From a review of the soil testing a set of most probable soil properties was established, see Table I.

The analysis was carried out using the Maunsell computer program *Diana* which is based on a beam and non-linear spring model. The analysis technique uses movement of the wall to model the stress path for a soil element moving from the installed value of $K_{0\,installed}$ to a proportion of K_a or K_p depending on the movement. This model is relatively simple to use for routine design purposes.

To model the effect of diaphragm wall installation the in situ value of K_0 was reduced to $K_{0\,installed} = 1.0$ subsequent to excavation and concreting of the diaphragm wall panel, before bulk excavation was carried out. This has been taken as the starting point of the analysis.

The *Diana* model was used in conjunction with a structural analysis program to produce deflections of the wall that would be expected to occur at failure. Failure was taken as being the limit of serviceable behaviour with the predicted formation of 0.2mm wide cracks.

Diana analyses were carried out for each stage of excavation using most probable soil properties. The structural program was then used to determine the increase in load required to cause failure during the subsequent excavation stage. A combined displacement profile was then generated. Thus an envelope of expected displacements and displacements at failure was established from which appropriate trigger values were derived.

The *Diana* analysis was carried out based on undrained soil conditions for the cohesive strata. Separate analyses using drained parameters indicated that much larger displacements would be expected if drained conditions were to arise.

In the event the displacements were less than those predicted.

7 STRUCTURAL DESIGN

The walls were designed to BS 8110. Concrete with strength 40N/mm² was used and crack widths were limited to 0.2mm. Partial safety factors on the loads derived from the *Diana* analyses were 1.0 for serviceability conditions and 1.4 at ultimate limit state. The critical design parameter was the limiting crack width for both temporary and long-term conditions.

A sensitivity analysis was carried out to assess the effects of the reduction in wall stiffness due to cracking of the concrete. This indicated that the deflections would be increased by approximately 5% with possible reductions in bending moments in the construction stages of up to 30%. This reduction was not used in the design of the wall.

The trusses were designed for the same criteria as the walls. The frame model showed most variation from the measured values near the assumed supports for the inside chord of the truss. This indicates that there was more interaction between the walls and the trusses than allowed by the simplified supports of the model.

8 MONITORING RESULTS

The instruments were monitored 2 or 3 times a week, increasing to daily readings during periods of excavation.

8.1 First stage

Excavation down to the truss at 99.3mPD, install floor slab and truss.

Ground surface movements around the excavation were larger than expected. The movement near the excavation may have been due to the construction of the diaphragm walls. Movements of up to 9mm were recorded by the electrolevels.

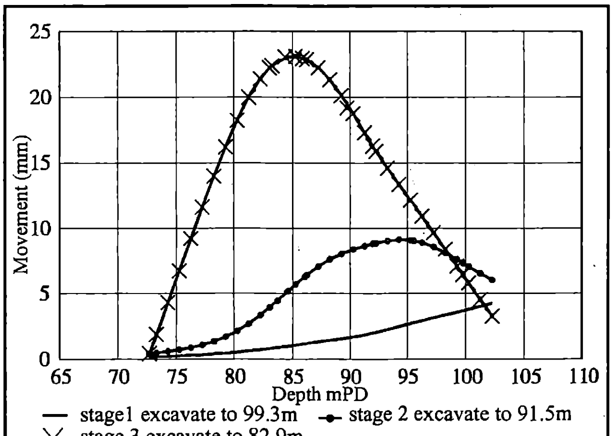

Figure 4: Predicted wall displacements

The inclinometer deflections at this stage generally show the wall to be acting as a cantilever with the maximum movement (4 to 6mm) occurring at the top of the wall. This deflection was generally as predicted by the *Diana* model, see Figures 4 and 5.

8.2 Second stage

Excavation down to the truss at 91.5mPD, install floor slab and truss.

Ground surface movements were lower than expected with very little change from the movements during the first stage.

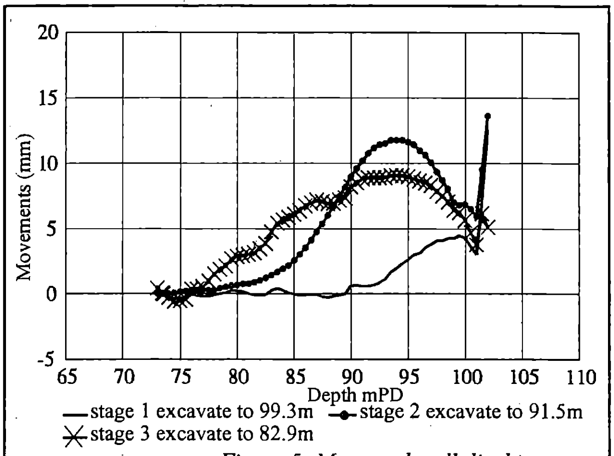

Figure 5: Measured wall displacements

At this stage the wall acts as a propped cantilever, the inclinometer deflections indicate the maximum wall movement (7 to 11mm) occurs midway between the props. The deflections show similar profiles to those predicted in the design, see Figure 4 and 5.

8.3 Third Stage

Excavation down to formation level (82.9mPD), cast lower plant room slab.

Ground surface movements around the structure were quite variable in this stage and statistically it would not be possible to say that they were significantly different from the expected movements or that any significant movement had occurred. In general the ground settlements showed small movements that were less than the values predicted by the analysis.

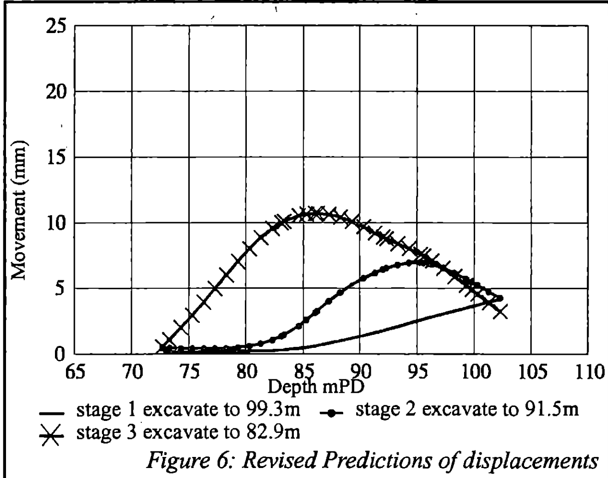

Figure 6: Revised Predictions of displacements

The wall inclinometers continued to give movements similar to each other with the point of maximum deflection moving down the wall. The maximum deflection as given by the inclinometers was much less than in the original design possibly indicating that the soil at these lower levels is stiffer than the design value. The maximum deflections between the lower plant room and basement levels were less than the trigger values and therefore no remedial action was required.

8.4 Back Analysis

After reviewing the monitoring results a back analysis using the original *Diana* model was carried out. The basic assumption was to change the model as little as possible and to restrict the deflections of the lower portion of the wall. The analysis was re-run with the following changes:

1. The diaphragm wall stiffness was increased from 25GPa to 27.5GPa.
2. The stiffness of the lower granular stratum was increased from 300MPa to 500MPa.

An additional strata equivalent to the Woolwich and Reading Beds' lower shelly clay (Ellison Classification) was added to the design. This unit was given the same stiffness as the granular strata below ie 500MPa. The inclusion of this unit was generally assessed from site visits and after reviewing all the instrumentation. The wall displacements predicted by this model are shown in Figure 6.

8.5 Dewatering

To enable safe excavation of the station box, a dewatering system was installed to lower the groundwater level to below formation level. The system comprised 8 deep wells designed to dewater the Thanet Sands, with 4 wells installed outside the box and 4 inside. The external wells were commissioned first, the internal wells being utilised when the excavation was below Lower Plant Room level.

A steady state draw down was readily achieved to 81mPD in the granular Woolwich and Reading Beds and Thanet Sands, a drop of 9m.

8.6 Earth Pressures

The pressures indicated by the earth pressure cells showed considerable variation between each of the four walls, although the shape of the pressure profile was quite consistent. The effective pressure profiles for various stages of excavation are shown on Figure 7. Also shown are theoretical pressure lines for $K_0 = 1$. The pressures recorded by the top two cells, which were located in the alluvium and at the Thames Gravel/ London Clay interface, are consistent with theoretical pressures, assuming $K_0 = 1$ for the London Clay.

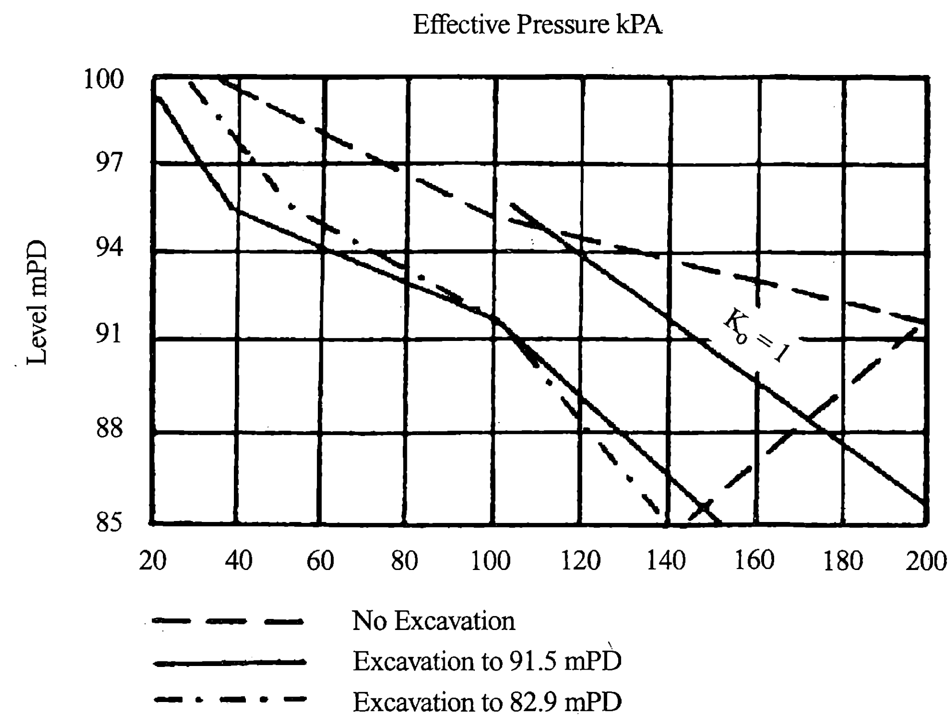

Figure7: Effective Pressure Plots

Maximum at rest pressures were recorded in the third level of pressure cells which were located in the Upper Mottled Clay of the Woolwich and Reading Beds. The recorded pressure ranged from approximately 250 to 350Kpa, these values being equivalent to $K_0 = 1.5$. Corresponding effective pressures were estimated as 190kPa and 285kPa.

The effective stresses reduced as the excavation proceeded and values approaching the active condition were achieved at the final excavation stage.

The lowest level of pressure cells recorded pressures which are consistent with a theoretical value derived from $K_0 = 1\text{-}\sin\phi'$. This is possibly coincidental, however, and the relatively low pressure is considered to be due to the stiffness of the Woolwich and Reading Beds at this level. The fact that the derived values of effective stress at this level did not change with the progress of the excavation tends to support this view, since the reduction in pore water pressures at this level would be expected to lead to an increase in effective stress.

9 CONCLUSION

The construction of the station box for Bermondsey station on the JLE was completed efficiently by using an alternative sequence of excavation which omitted a temporary prop and was justified by extensive monitoring of the structure during construction. Ground movements and building settlements were also monitored and were within acceptable limits. The soil properties were checked by back analysing the wall and for some layers were found to be stiffer than used in the original analysis.

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