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Case history of a deep 'stepped box' excavation in soft ground at the sea front, Langney Point, Eastbourne

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ABSTRACT: An underground treatment works required 11 and 14 m free span in a severe ground and groundwater environment. Predictions of soil structure interaction were required to effect safe and economic construction. Class A predictions were achieved by continuum modelling using the Imperial College Finite Element Program.

1 INTRODUCTION

Eastbourne Wastewater Treatment Scheme requires a structure at Langney Point, Eastbourne. The process machinery for the works will be housed underground in a rectangular box some 130m by 40m in plan with a stepped base 11m and 14m below the current ground level. Plate 1 illustrates the position of the box relative to the sea, access roads, current and future housing provision and an existing small circular outfall.

The Client required the safe construction of a substantially watertight and robust sub-structure with a design life of 40+ years. Restrictions were also placed on noise, land take and disturbance to existing sea defences. Movement concerns were particularly important in the area of the limited land take adjacent to housing. Speed of construction was also a major consideration for economic and environmental reasons.

Although any structural form could be considered, the Client required maximum free standing space (i.e. minimum propping) and discouraged the use of anchorages for support or holding down. All tenderers proposed the use of diaphragm walls and construction using top-down methods.

Alternatives to the client's 130m by 40m by 14m deep box were investigated for the retaining walls. These included using a combination of walls that were circular, straight and ellipsoidal in plan covering a smaller plan area with a deeper excavation. In the event the contract agreements and superstructure requirements dictated that the rectangular shape could not be changed.

A late tender proposal for a stepped base was agreed resulting in the shape illustrated in Fig 1.

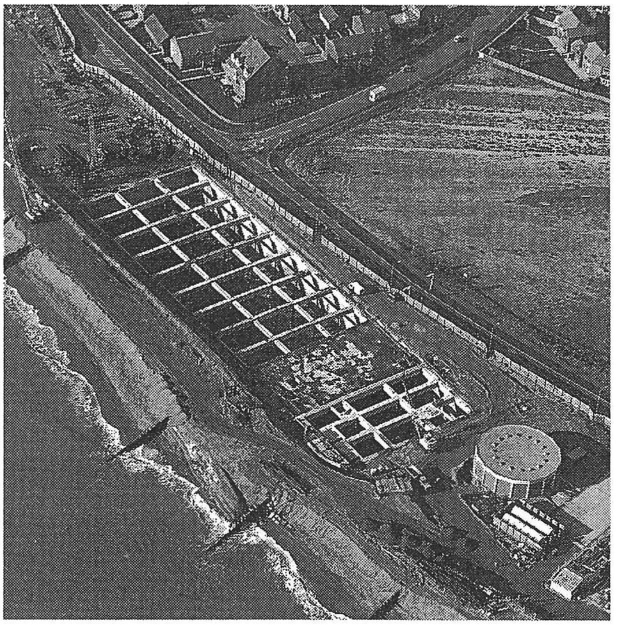


Plate 1.

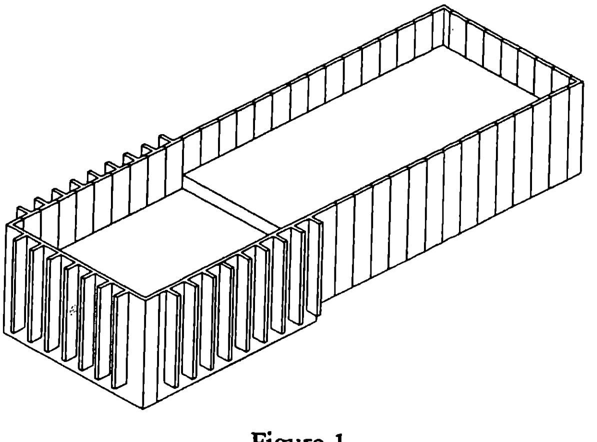


Figure 1.

It was recognised that meaningful prediction of movement and design stresses for the retaining walls could only be obtained from Finite Element Analyses (FEA). Only robust well conditioned Finite Element Method programs should be used for such predictions, particularly where economy and time dictate that sensitivity studies (FEA) could not be undertaken during the tender period. Equally importantly, experienced personnel had to be employed in 'driving' and assessing output. The authors' companies have some 10 years experience working together with one well developed program.

2 GROUND AND GROUNDWATER

The ground profile is illustrated in Fig 2 along with the design profile adopted for all of the works. The ground comprises dense beach gravel overlying a dense marine sand over a clay, fine sand, silt melange of Alluvium overlying the Gault Clay. The alluvium rises in elevation particularly towards the NE corner of the site and this 'worst case' profile was adopted for all analyses.

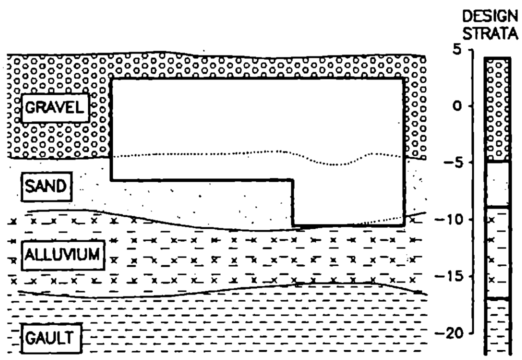


Figure 2

The highest groundwater recorded in a borehole had an elevation of +2.1mOD. However it was a design requirement to accommodate levels reaching a Highest Astronomical Tide (HAT) = +4.25 mOD. Long term allowances for tidal levels (and therefore water pressures in permeable soils) to reach +6.35mOD external to but not on top of the box were to be made. Direct hydraulic connectivity was assumed within the diaphragm wall periphery. A pictorial representation of these features is indicated in Fig 3.

Sampling, testing and characterising such a profile is never going to be easy and even the results of the standard site investigation for the Gault (e.g. Cu vs. Depth) were questionable. Fortunately some extra boreholes were carried out in which SPT's were done and it was possible to examine arisings. The SPT results allowed the data in Table 1 to be provided for general design appreciation.

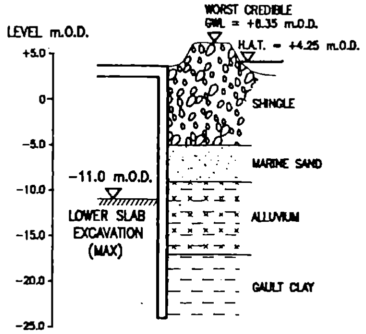


Figure 3

Table 1

	Soil Strength (N / Cu)	ϕ'	Bulk Density (kN/m^2)	k (m/s)
Gravel	30 / -	38	20	10^{-5}
Sand	35 / -	38	21	10^{-6}
Alluvium	-/25to40	30	18.5	Mixed
Gault	-/90to360	25 (c'= 10)	20	N/A

The following questions were prominent in deciding how to build and design the sub-structure :-

Gravel; could it retain bentonite and would trench support be sustained? What quality of finish could be expected on the wall (alignment and protrusion tolerances were at the limit to reduce the box size)?

Alluvium; what was the dominant material of the melange? Could it be sensibly dewatered? Would it have to be treated to prevent ground heave? What surcharging pressures would be transmitted to the base slab? How would toeing into the Gault benefit the base behaviour?

Gault Clay; does its fissuring ensure hydraulic connectivity (in the short term) whilst still not significantly reducing strength or stiffness? Does it act as a stiff clay or a weak mudstone? Is the SPT vs. Depth profile a better gauge of consistency than Cu? Do experiences of low adhesion factors on piles generally apply to this deep material?

It is a reflection on our industry that site investigation techniques are still 'out of step' with our analytical and modelling capabilities. Eastbourne illustrates vividly the need to have experienced personnel at every level of interpretation of data and in the perusal, assessment and preparation for a relatively simple problem. The parameters in the FEA were not lightly chosen and needed debate, study and the agreement of many parties within the professional team. Design Build contracts may in future facilitate this whole process by developing different philosophies and measuring/investigation techniques.

Although FEA was always seen as necessary on a complicated soil structure interaction problem such as this, 'scoping' analyses are useful to place FEA in context and assess the relevance of for example movement predictions.

Considering the flat panel condition in classic 'top-down' (i.e. cast the walls, cap the walls and support them with the top floor and excavate to the underside of the base) using limit equilibrium analyses a bending moment (BM) of about 3000 kNm/m was predicted. If this is a locked in stress and a standard load factor is applied, an ultimate BM of about 4200 kNm/m is produced. This is likely to be the design BM since other cases would argue for lower load factors. This should be compared with the 4000 kNm/m used from the FE analysis (3600 kNm/m with the 'worst credible' load factors).

Movement was predicted from an O'Rourke approach (Ref.1) for flat panels in the marine sand at excavation level and toed firmly into the Gault. A Factor of Safety against basal failure > 2 was estimated and a movement of 0.5% is calculated (i.e. 11.5 m x 0.5% = 60 mm). This compares with the FE prediction of 55 mm.

Movement is real and stress more 'abstract' and we do not know that the BM correspondence is significant, but we do know the magnitude of movement is comparable with observations. Comfortingly then, if not suprisingly, the results are comparable.

3 FINITE ELEMENT ANALYSES

Imperial College Finite Element Program (ICFEP), developed by Prof. Potts, was used in two 2D sections through the site. An indication of the mesh employed is given in Fig 4. For all soils linear elastic perfectly plastic constitutive models were used to describe material behaviour. Structural elements (walls, piles and slabs) were modelled as elastic materials. The analyses did not account for wall installation.

Previous analyses and experience indicated that the presence of a substantial number of large diameter piles (these are 1800mm and 900mm dia. piles at about 4.5m centre to centre spacing) affects the movement of the soils and that the linkage between various soils and structural elements is important. Pile behaviour approximates poorly to plane strain analyses and therefore sweeping assumptions had to be made regarding the sectional stiffness of the piles

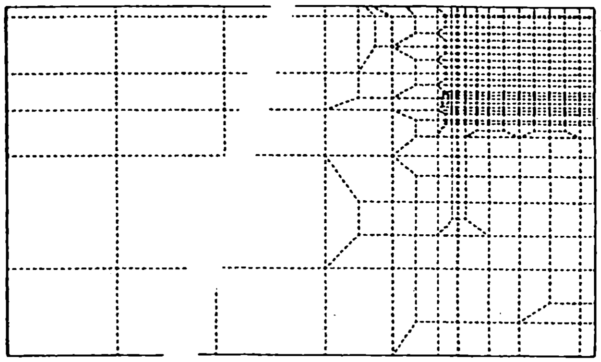


Figure 4

and the ground stiffness in the pile reinforced zone.

The properties assumed for the ground and structural members are shown in Fig 5. Assumed values of stiffness were used for the gravel and sand. The alluvium stiffness was estimated from E/Cu = 500 to give E' = 10 MN/m2. From previous experience in the Gault, the N value versus depth relationship was linked to the cohesion (Cu = 4.5 N). This gave Eu/Cu = 750 at the top to Eu/Cu = 1000 @ 30 m below ground level. A drained stiffness profile of E' = 65 + 12.4z MN/m^2 was adopted (where z = depth into the Gault Clay).

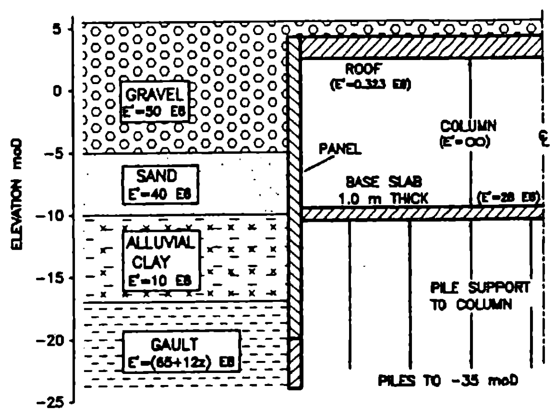
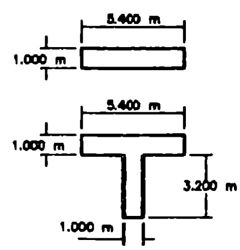


Figure 5

For the purposes of the analyses, soils were treated as 'drained', except for the Gault. The Gault was assumed undrained until the long term conditions were developed. Analyses were uncoupled, i.e. independent of time, removing the need to concentrate on permeability in absolute terms.

Diaphragm walls were modelled as plane sections 1.0m thick in both analyses. The flat panel section was given a stiffness based on Young's Modulus of concrete of 28E6 kN/m^2.



The Tee shaped section was given a stiffness of 803.4E6 kN/m^2 to -20 mOD. Below this level the back leg was curtailed in order to save material cost, but more importantly construction time.

The grillage of beams (as shown in plate 1), roof and base slabs were given appropriate compressive stiffness for an equivalent plane stain condition. Pile stiffness was modelled in two ways :-

- prior to base construction large diameter piles supporting the grillage were given compressive stiffness of concrete (Area and E per metre run);
- after base construction piles are subject to tension and given an E based on E_{steel} and steel area equivalent per metre.

The piles were assumed to be bar elements with a point of fixity at -35 mOD and attached to that point and the base slab only. In the piled zone, the drained stiffness of the Gault was enhanced to E' = 160 + 12.4z MN/m^2. It was assumed that the alluvium was not affected.

As yielding soil models were employed it was necessary to define an initial stress condition. This was generated assuming no longitudinal variation, ground level at +5.5 mOD, GWL at +4.25 mOD and that pore pressures are hydrostatic. A surcharge modelled the sea defence wall (20 kPa 8 m back from the box). The analysis then followed the construction sequence (anticipated sequence) as described below:

0. Initial stresses at GWL = +4.25 mOD.
1. Apply 20 kPa surcharge for sea defence.
2. Apply 10 kPa live surcharge.
3. Gault stiffened (piles installed).
4. Dewater inside box to GWL = -0.5 mOD.
5. Construct grillage beams and columns.
6. Dewater inside box to GWL = -4.5 mOD.
7. Excavate to -3.5 mOD.
8. Dewater to 1 m below final excavation.
9. Excavate to formation level.
10. Backfill over excavation with gravel.
11. Install piles and construct base slab.
12. Roof slab constructed.
13. Backfill over structure.
14. Dead loading applied.
15. PWP in alluvium = +4.25 mOD.
16. PWP in Gault = +4.25 mOD.
17. Final GWL = +6.35 mOD.

Stage 9 dewatering from no deeper than -3.5 mOD was introduced to ensure no hydraulic basal heave failure occurred as the overburden was removed above alluvial clay layers. Stage 15 allows hydraulic connection between the water table and the Alluvium but assumed undrained behaviour in the Gault. There seems to be evidence (Ref.2 & 3) that such an occurrence can take place in highly fissured weak rocks, where the hydraulic connectivity exists in macro scale fissures but material does not soften as quickly as in the individual clay lump. This leads to some interesting predictions (see below).

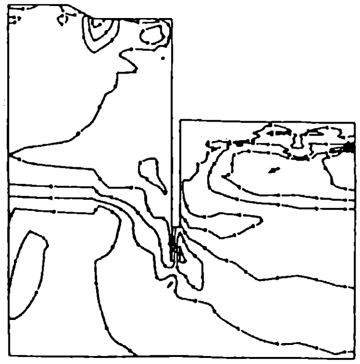


Figure 6

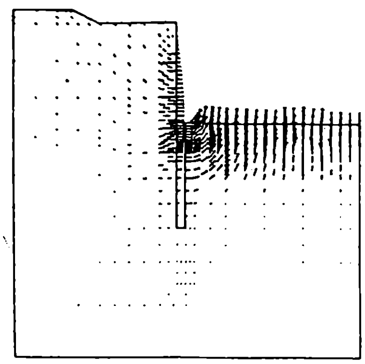
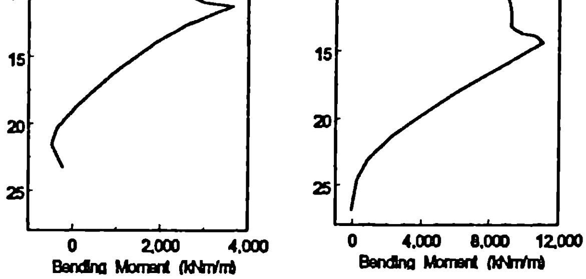


Figure 7

Figs 6 and 7 show stress contours and vector strains for the flat analyses at construction stage 9. The ability to assess stress states and movements stage by stage proved very useful in a contract where the grillage and superstructure provision had to be altered as the works progressed.

Bending moments for the runs are reproduced if Fig 8a and 8b and lateral movement plots in Fig 10a and 10b for the high level and deep level base slab (flat and tee panel) respectively. For the flat slab



(a) (b)

Figure 8

(high level) it can be seen that prior to the grillage installation (stage 5) the wall acts as a cantilever and moves a maximum of 20mm. After grillage construction and excavation to -7.5 mOD (underside of base) the movement increases to about 55 mm, with a corresponding maximum BM (short term) of 2300 kNm/m.

The analysis indicated that once the base slab was cast, upper support stiffened and dead loads applied, there was little change in wall deflection, bending moments or prop loads.

When dewatering was stopped, after completion of the base, it was assumed there would be an immediate increase in pore pressure in the alluvium. As the slab is restrained vertically by piles, with the wall acting as a membrane, the expansion of the soil pushes the wall back (apparently 8 mm). This results in a BM increase of 50%, to a maximum of 3600 kNm/m. This 'Balloon Effect' brings BM's to ultimate state using Lf =1.1 (water pressure) and 1.4 (soil pressure) close to the condition estimated from a limit equilibrium analysis but via a very different modelling process. This has significant consequences on the buildability of the structure.

Dissipation of excess pore pressure in the Gault indicates a further backward movement of the wall with negligible change in BM. The final ground water level of +6.35mOD was also applied, with little change in the BM and wall deflection. Therefore, given the adoption of lower load factors (accidental or worst credible loading) the ultimate BM's were as above. Prop forces did increase (see below).

For the tee panels (low level base) the form of behaviour was very similar, with movement being significantly smaller but BM's being much higher. Maximum wall displacement was predicted to be about 25mm and BM's showed the same balloon effect (albeit to a smaller %) of 8500 kNm/m short term to 11000 kNm/m with cessation of dewatering. There was again, little further effect with pressure dissipation in the Gault or with the increase in GWL to +6.35 mOD.

For the 'Box Effect', on global ground displacements, plane strain linear elastic analyses have limitations, not least a tendency to over predict the lateral extent of ground movement back from the wall. Again experience of the analytical output and this type of structure is necessary to provide interpretation to produce a ground surface settlement contour similar to Fig 9.

This satisfied the client that damaging ground movement would not be experienced at existing housing near the pinch point of land take at Langney Point.

The magnitude of prop forces and the design of props are a subject of some debate in the UK, even for singly propped walls. What is not in question, however, is that props in these types of structures must not fail. Considering that much of the

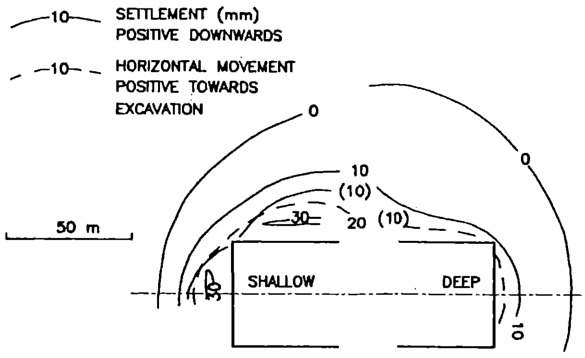


Figure 9

Eastbourne pressures are "real" in that they are derived from real water pressures, design economies were not sought in any propping arrangements.

Table 2 shows the prop forces in the grillage for both ends of the box and the base slab forces (note the sign convention and that base slab forces are reported as 'tensions').

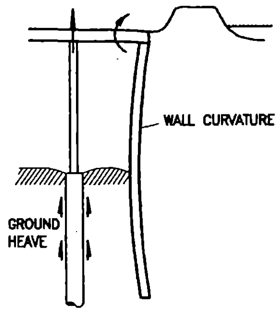
Table 2

Stage	Prop Force (kN/m)			
	Flat Panel		Tee Panel	
	Grillage/ Roof	Base Slab	Grillage/ Roof	Base Slab
9	487	-	1045	-
15	465	-539	1106	-946
17	584	-493	1168	-957

Although buildabilty is outside the scope of this paper, it can only be addressed and assessed via real modelling of behaviour. Only FEA provides this opportunity. From the above, for example, three features were addressed which may never have been fully investigated:-

- Limit equilibrium analyses would not have given the detail of prop load variation covered by the FEA. It is apparent that given up to 600 kN/m run difference in load between the two ends of the box, sway forces would effect the slender column support of the grillage and push back the flat panel wall. With the quantification, a flat plate diaphragm could be designed to enhance the grillage and take load out on the sides of the box with minimum disruption to the construction programme.
- Predictions at intermediate stages in the FEA could be examined in order to assess the effects of

transmitting ground heave through the rigid columns onto the grillage and possibly onto the wall. Steel requirements and/or curtailment plans could be modified. The heave predicted for this set of support columns was in fact 15mm.



- Assessment could be made of the 'Balloon Effect'. FEA does indicate this effect to varying degrees (dependent upon connectivity) for other deep basement structures and there has been a tendency to ignore predicted tensions in the past. However there have been recent experiences in the UK of water penetration into diaphragm wall boxes at the wall slab connection. If not the sole cause of this path, the 'Balloon Effect', at least exacerbates other problems. From this evidence it was decided to accept the possibility of the development of tension and incorporate design features within the connection (debond steel, key slab into wall, incorporate groutable tubing) to cope with predicted stresses and possible movement.

4 OBSERVATIONS TO DATE

Instrumentation, surveying and observations are not within the authors' provenance and the results below are reported with permission of the main civils contractor Biwater Europe Ltd. Fig 10a and 10b show the results of two surviving inclinometers, fortuitously in the flat and tee panel areas. The accuracy of the FE predictions is apparent. It was expected that the movements may have been closer to each other and this feature is addressed in Ref.4.

If the displacement profile is used to predict where one would expect flexural cracking (maximum wall curvature) then levels around -5 to -7mOD would be suggested. Prior to construction of the base slab some seepage was observed at these levels in the vicinity of small horizontal cracks, which were not associated with buildability concerns and were concluded to be flexural cracking (an occurrence which many UK practitioners claim not to have observed). These faces are now covered over with a structural inner lining wall.

The tops of the piles, predicted to heave 15mm by the FEA are reported to have been very accurately monitored and to have moved a maximum of 13mm.

The structure is nearing completion at this time and there is no evidence of water ingress via the base slab connection. The final stage of monitoring is yet to be undertaken and therefore there are no results to confirm the 'Balloon Effect' has occurred.

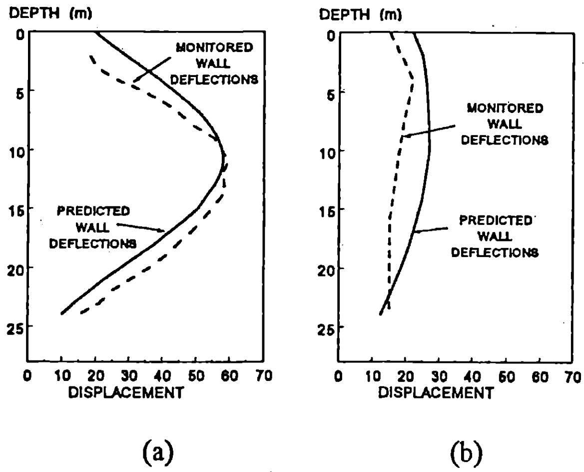


Figure 10

5 CONCLUSIONS

There is a reluctance to use FE analyses to predict 'ground behaviour' too ambitiously in soil/structure interaction. Modelling mechanisms of behaviour i.e. how works react, has generally been accepted but predicted values have not been considered absolute.

At Eastbourne adequate and apparently accurate modelling has been carried out using a linear elastic finite element program (with yield limits), perhaps helped by the fact that predicted pressures are very real (water pressures through permeable gravel).

Despite the apparent accuracy much engineering judgement was required in assessing input and output parameters. Only a robust FE program (such as ICFEP) with experienced users should be relied upon for even this simple problem.

Much insight into the soil/structure interaction was gained by studying the snapshot stages.

Current SI techniques remain inadequate in the materials of the Eastbourne profile and practice needs to change to produce sensible ground input information.

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