Groundwater control in estuarine deposits

Contrôle de la nappe phréatique dans des dépôts d'estuaires

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SYNOPSIS The paper describes the groundwater control necessary in order to excavate a basement in the Estaurine Deposits typical of the Durban area. It focuses on the importance of geology and the derivation of material permeabilities using various methods of interpretation of a pumping test. The estimated flow and concommitant changes in pore pressure are compared to the measured field data.

INTRODUCTION

A basement has been constructed as part of the new Natal Playhouse, Durban, South Africa. The floor level is 4.5m below the water table and 9m below ground level. Prior to excavation of the basement, contracts for demolition, piling, underpinning of adjacent buildings and construction of an 800mm thick diaphragm wall forming the basement were executed.

The excavation proceeded part way down until a temporary steel bracing grid was installed (Figure 1a and 1b). With the excavation complete, the permanent structure within the basement was constructed prior to removal of the steel grid. Temporary dewatering was carried out from within the excavation, while pore water pressures were monitored.

GEOLOGY

The site is underlain by alluvial and Estuarine Deposits, typical of the Durban area. These are silty and clayey fine sands of medium density which were deposited in the geologically recent past. The thickness of these Harbour Beds is about 32m at this site, at which depth the Cretaceous Beds occur. An upper clayey layer was encountered at between -7m and -9m mean sea level (MSL) and a less defined lower clayey layer between -12m and -13m MSL (Figure 2). The aquifer is generally a fine sand but in places becomes clayey, but can also contain coarse sand or fine gravel. The clayey pockets are attributed to in situ weathering of reworked alluvial and Estuarine Deposits. The groundwater level lies at about +1.5m MSL.

PUMPING TEST

The purpose of the pumping test was two-fold. Firstly, it was necessary to establish the response of piezometers installed beneath the upper clayey layer and compare their response with standpipes installed in the overlying sands. The difference in response gives an indication of the likely effect of the clayey layer in restricting flow into the excavation during construction. Secondly, it provides the necessary data for a more accurate determination of the bulk permeability of the aquifer beneath the toe of the diaphragm wall, and hence enable the inflow and drawdown to be estimated.

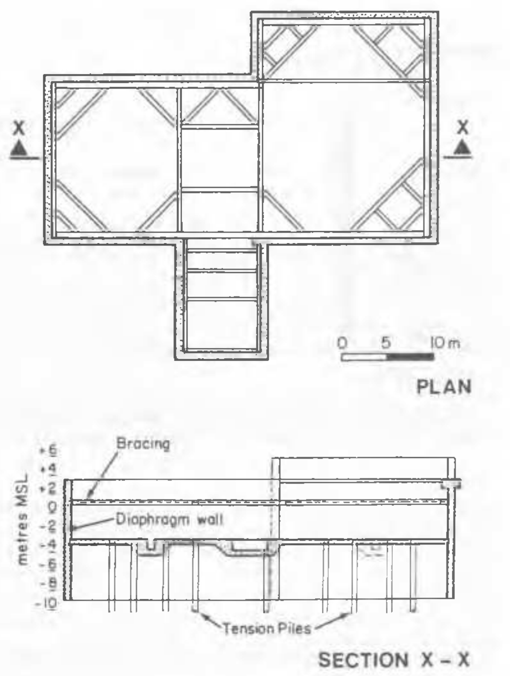

Figure 1(a) Basement Plan and Section

Figure 1(b) Bracing Grid Temporarily Supporting Diaphragm Wall

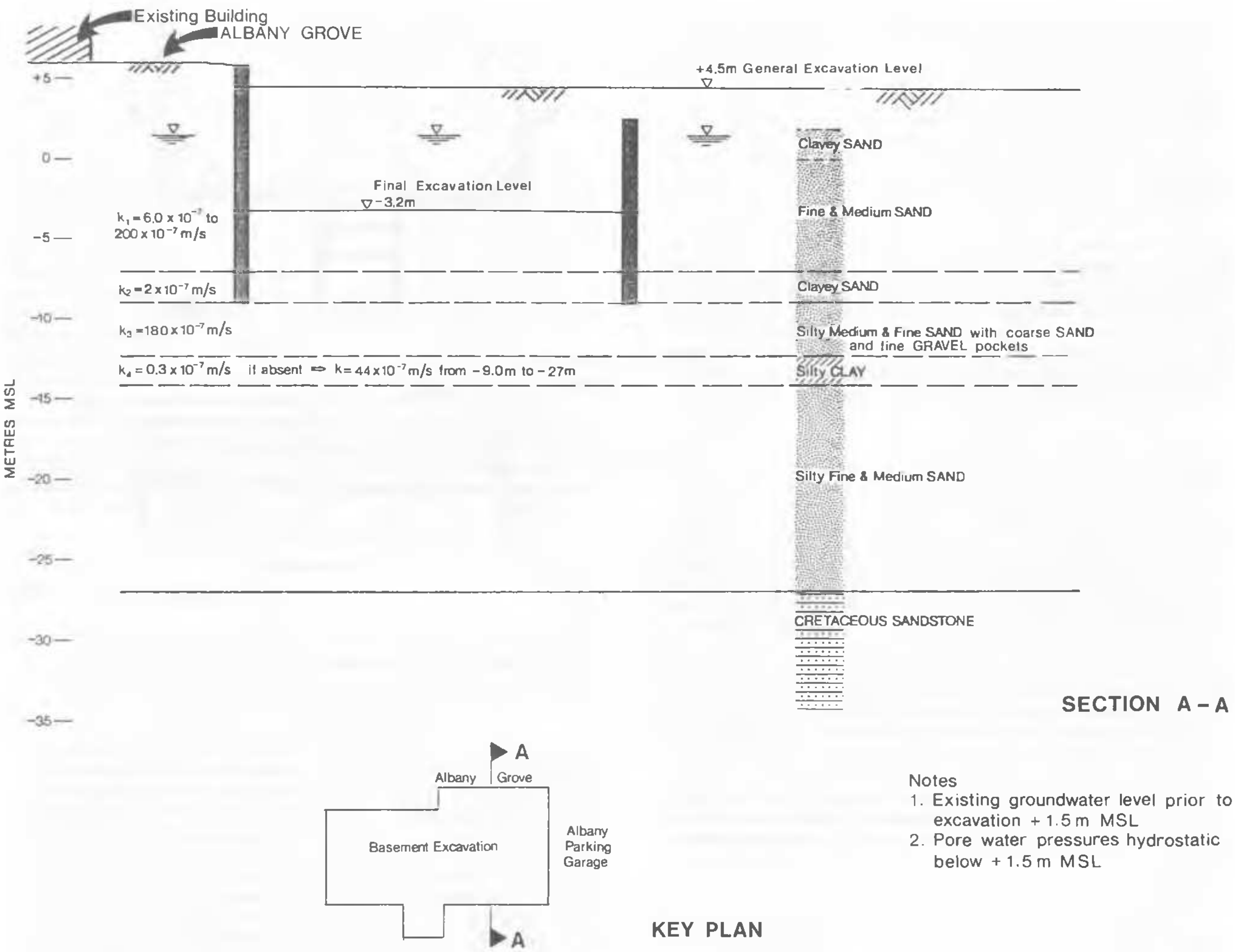

Figure 2 Section Through Basement Showing Assumed Geology and Permeabilities

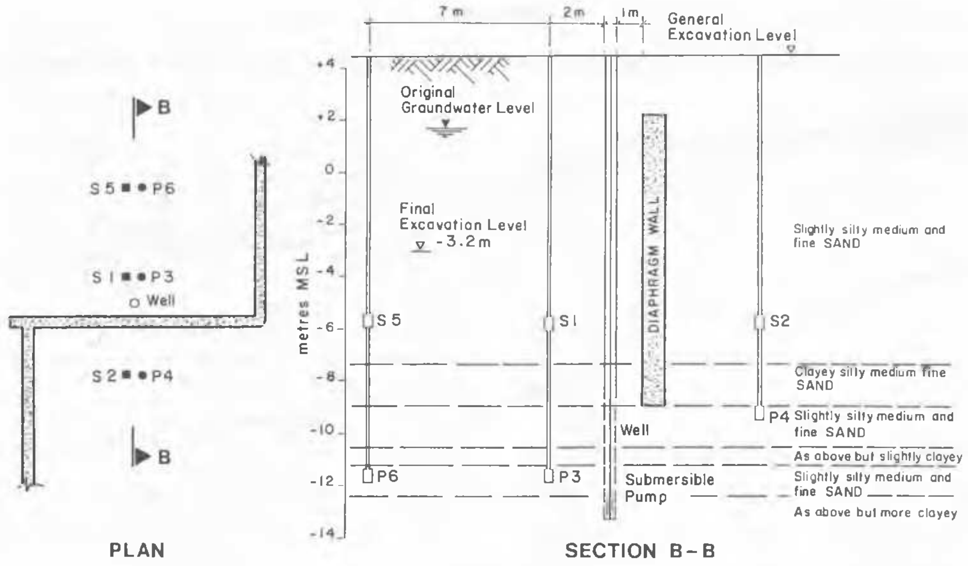

Figure 3 Plan and Section of Pumping Test

The test comprised the installation of a well, three piezometers and three standpipes in the positions and elevations shown on Figure 3. The well was positioned 1m from the diaphragm wall with a 4m screen length below the toe of the wall from -9m to -13m MSL. The well was formed by drilling a large diameter hole using a washboring technique and revert as a drilling fluid. A 200mm diameter PVC tube with 6mm holes drilled at close spacing over the bottom 4m, surrounded by stainless steel screen, was inserted in the hole. A sand filter was placed around the screen and a bentonite seal above to ensure that there was no leakage between the overlying sands and the aquifer. The piezometers were installed at elevations of between -9.5m and -11.5m MSL to measure the pore pressures within the aquifer during pumping. The standpipes were installed at about -6m MSL to measure the change in water level in the overlying sands.

Initially, a constant discharge was achieved during the removal of the well and storage water but, after a few minutes, it was found that a constant rate of pumping was not possible as the yield from the well was small relative to the capacity of the pump. It was decided to pump at regular intervals so that the water level did not rise above the top of the well screen. A plot of flow rate against time is shown in Figure 4, and it can be seen that the average for the three days of pumping was about 8 litres/minute.

The plots of drawdown versus time for S1, P3, S5 and P6 are shown in Figure 5. The local fluctuations in readings are probably due to inconsistencies in measurement, fluctuations in pore pressures or the influence of local variations in the groundwater regime affected by the pumping procedure.

The response of piezometers and standpipes at the same radii from the well were different, indicating that the upper clayey layer partially restricts the flow of water from the overlying sands to the aquifer. The difference between S5 and P6 was very small and the difference between P3 and S1 was about 0.5m. The piezometric data suggest that the aquifer behaves either semi-confined or 'leaky' where only vertical flow groundwater would occur in the semi-pervious clayey layer or as semi-unconfined where some horizontal flow occurs within the clayey layer.

For the purpose of analysing the pumping test data, the aquifer was assumed to be semi-confined and that steady state conditions prevailed during pumping. Two methods of analysing the pumping test in a semi-confined aquifer were used, namely De Glee's method and the Hantush-Jacob method (Bulletin 11, 1970). An aquifer thickness of 4m equivalent to the full well screen length was assumed

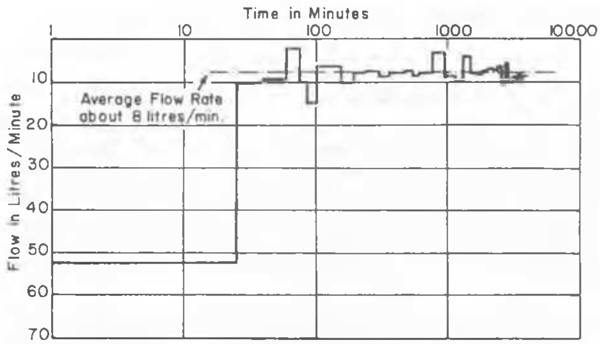

Figure 4 Pumping Rate Versus Time

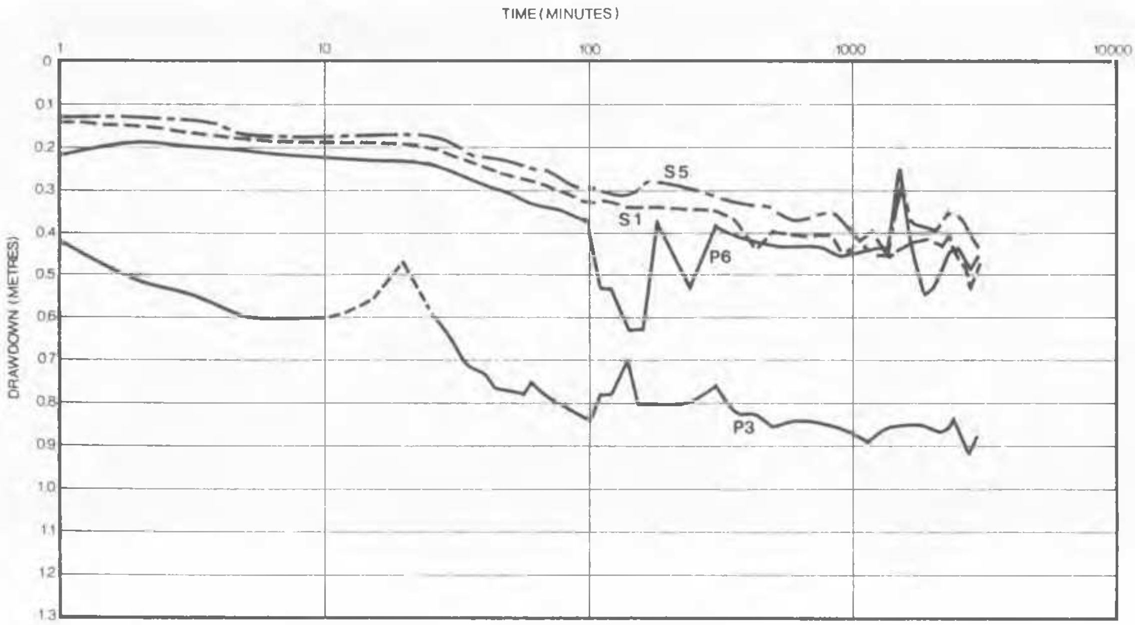

Figure 5 Drawdown Versus Time During Pumping

which implies that the lower clay acts as an impermeable boundary. For an average flow of 8 litres/minute the aquifer permeabilities were calculated as k = 190 x 10^{-7}m/sec and k = 170 x 10^{-7}m/sec for the De Glee and Hantush-Jacob methods respectively. The well can also be considered to be partially penetrating assuming the Cretaceous Beds at -27m MSL act as the impermeable boundary. Using Huisman's correction method the aquifer permeability was calculated as k = 40 x 10^{-7}m/sec. All these calculated values are considerably higher than those obtained from rising head tests in the piezometers installed within the aquifer, namely 9 to 11 x 10^{-7}m/sec. This is attributed to the presence of coarse sand and fine gravel affecting the mass permeability.

Values of permeability for the various materials were measured during the site investigation by performing falling head tests in the boreholes and also rising head tests were carried out in standpipes and piezometers and from this information average permeabilities were assigned to the various strata as shown in Figure 2.

EXCAVATION DEWATERING

From the assumed section of the average thicknesses and permeabilities of the various materials across the site shown in Figure 2, calculations of inflow into the excavation, drawdown outside and base stability of the excavation during construction were carried out.

The flow into the excavation is controlled by the permeability of the aquifer beneath the toe of the wall around the perimeter of the excavation and the permeability of the upper clayey layer within the diaphragm wall. Various methods of estimating the total inflow of water were used to obtain approximate upper and lower bound quantities. In carrying out these analyses various assumptions were made.

1. The pumped water level within the excavation is - 4 m MSL
2. Initial water level outside the excavation is + 1.5 m MSL
3. Steady state conditions prevail

The first method of analysis assumed the simple analogy that the excavation is equivalent to a large well 37 m in diameter. Two conditions were considered: firstly, the unlikely situation where the upper clayey layer has no effect and the ground around the excavation behaves as an unconfined aquifer and secondly, where the clayey layer confines the aquifer below the toe of the wall. For both cases, the analyses ignore the effect of material inside the excavation and are, therefore, conservative in this regard. The estimated total flow for the unconfined case was 34m^3/hour and 5.3m^3/hour for the confined case. By ignoring material inside and assuming that the clayey layer has no effect, the unconfined analysis is probably too conservative and the inflow should be closer to the smaller quantity.

The second method of analysis assumed that no head loss occurs beneath the wall and the full head loss of 5.5 m occurs within the material between -9 m and -4 m MSL. This method gives a lower bound flow, one which is unlikely to occur, as the clayey layer is probably not continuous and some head loss outside will occur. The calculated total inflow into the excavation was 1.1m^3/hour.

In addition to these hand analyses, a finite element program was used to model the groundwater flow. The program analyses axially-symmetric steady state flow conditions and permits different layers with various permeabilities and a row of wells to be specified. An equivalent excavation diameter of 37m was assumed which has an approximately equivalent perimeter to the actual perimeter excluding the stairwell. For the values of permeability given in Figure 2 the calculated flow was $0,9m^3$/hour, indicating that the inflow is controlled by the upper clayey layer. Any discontinuity of this layer will therefore lead to increased flows as shown by the hand analyses. The anticipated range of inflow was therefore from about $1m^3$/hour to $20m^3$/hour.

The dewatering was carried out using a series of well points. The pumping rate from the excavation was monitored during construction and was initially about $5m^3$ per hour during the period of basement excavation and rose to an average of $10.8m^3$ per hour.

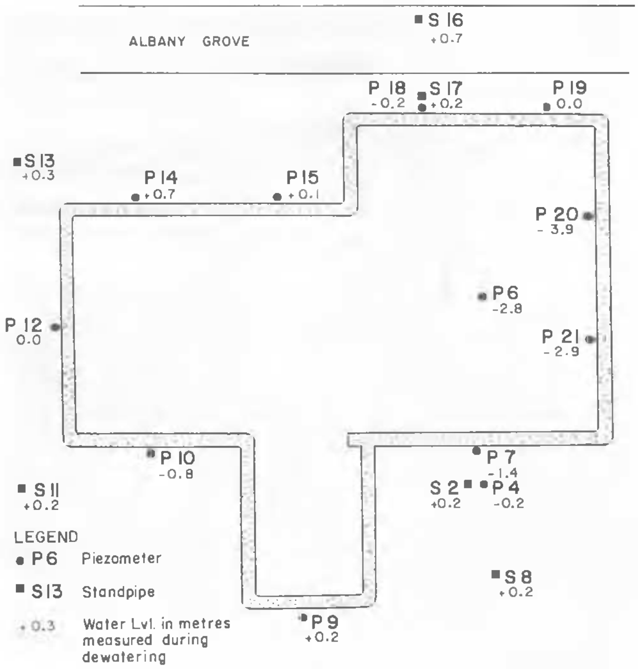

Figure 6 Layout of Piezometers and standpipes

DRAWDOWN

The drawdown in the aquifer outside the excavation is related to the total inflow into the excavation, calculated for the assumed permeabilities. The equivalent well analogy was applied for different distances from the wall and the drawdown was determined from the finite element analyses. The former is a relatively crude approach and although wide variations can be obtained for example by varying the assumed radius of influence the impact of permeability variations can be quickly assessed. Table I below shows the calculated head in the aquifer outside the excavation at various distances from the wall.

In order to monitor pore water pressures and to confirm stability during basement excavation, standpipes and piezometers were installed in the positions shown in Figure 6 and read on a daily basis. A typical section is shown in Figure 7. The drawdown in the aquifer was slightly more than given by the finite element analysis with $q = 5.8m^3$/hour and this reflects the higher actual measured flow, namely $10.8m^3$/hour. In order to obtain close matching of the piezometric elevations, calculated by the finite element program, the average permeability of the upper clayey layer has to be slightly in excess of 2×10^{-6} m/sec. This is ten times the permeability measured during the site investigation and highlights the wide variations which can be obtained from these Estaurine Deposits.

TABLE I

Calculated Piezometric Elevation in the Aquifer

Method of Calculation	Total Inflow m^3/hour	Piezometric elevation in MSL		
		Distance from Diaphragm Wall		
		0m	5m	10m
Equivalent Well	5.0 to 15.0	-0.1 to -3.7	+0.2 to -3.0	+0.5 to - 2.4
FE Analysis	0.9 5.8	+0.8 -0.5	+0.95 -0.1	+1.1 +0.6

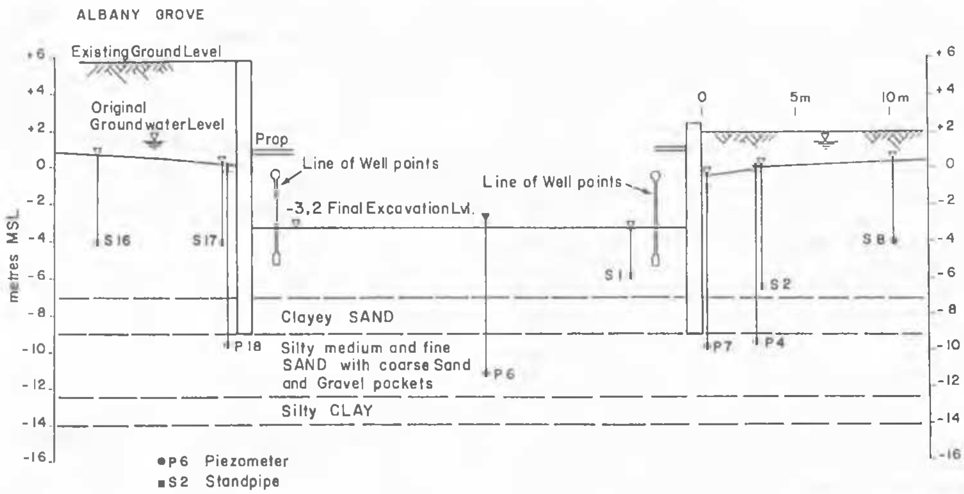

Figure 7 Drawdown During Dewatering

STABILITY

During construction the stability of the base of the excavation must be ensured. Failure by piping which would cause instability of the diaphram wall must be avoided and this mechanism can be created by two processes, namely piping due to heave and piping due to sub-surface erosion.

To check the possibility of heave failure, the calculated excess hydrostatic pressures acting at the base of the clayey layer at -9m MSL were compared to the effective weight of the soil. Relatively low factors of safety were obtained in the range 1.3 to 1.5. From the pumping test the response of the piezometers installed above the clayey layer indicated some head loss occurs through the layer. Consequently, this layer does not provide a continuous seal and indicates that this method of failure is unlikely.

A piping failure due to sub-surface erosion could occur if the excess head is sufficient to produce critical seepage velocities in the sands i.e. zero effective stress. The sands below final excavation were generally medium dense to dense (N = 20 to 30) and according to Navfac DM7 (1971), the factor of safety against piping is about 3.5.

CONCLUDING REMARKS

The experience gained from the site investigation, the pumping test, and the construction dewatering indicates that there are considerable variations in the vertical and horizontal continuity of the materials comprising the Estuarine Deposits. This makes the prediction of water inflow and drawdown difficult. It is unlikely that the Estaurine Deposits comprise a series of discrete layers but rather that it includes pockets of weathered reworked material. Thus, local permeability measurements via piezometers are likely to give a wide range of values. Although the measured inflow was within the anticipated range, an improved prediction would be difficult. This is because the variability of the upper clayey layer is not easily determined. Hand analyses provide a very suitable means of assessment of inflow and drawdown.

ACKNOWLEDGEMENTS

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