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Case Study of Groundwater Control in a Power Plant in a Marginal land in the Niger Delta

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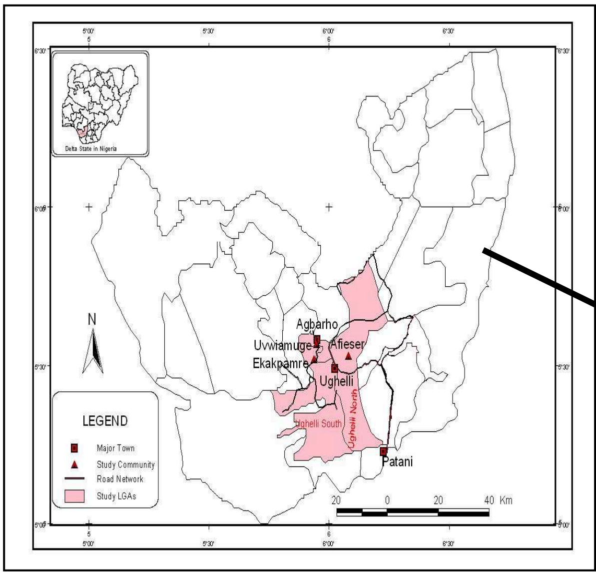
Abstract

The paper describes challenges posed by rising groundwater level in a construction site, apriori sandfilled, with installations that are sensitive to the presence of water. The geotechnical investigations, leading up to the design of a network of shallow abstraction well points are described. The permeability of the soil, which was shown to be the single most important parameter, was insensitive to the method of determinations (whether by *in-situ* or by empirical computations). By combining information on the radius of influence of the pumped well and the drawdown at the well point a network of dewatering wells were designed and constructed. The selection and operation of suitable pumps in these wells ensured that ground water level was maintained below the threshold level across the site, throughout the period of construction. The operation of pumps has ensured effective and satisfactory operation of the power plant more than 10 years after commissioning.

Introduction

A Power Plant was cited in a seasonally flooded fresh water swamp in the upper section of the Niger River flood plain (Fig.1). In order to avoid periodic floods and also create a competent bearing layer, a sandfill of up to 5m was placed over the site by hydraulic pumping from the nearby River Niger. The natural ground before the placement of the sandfill embankment, is underlain by an impermeable, stiff to hard silty clay soil (Groundscan 2002). To protect the sandfill from being washed away by runoff, a 10cm thick clay layer of approximate shear strength of 20kPa, was placed around the sandfill embankment as a liner. A sketch of the vertical section of the sandfill and embankment clay cover is shown in Fig. 2.



Construction work commenced within 3 yrs of placement of a sandfill when it was believed that primary and secondary consolidation settlements must have been completed. Attainment of a significant degree of consolidation settlement was very important, because the structures consisted of several rigid skid systems transporting gas that are intolerant of any form of settlement.

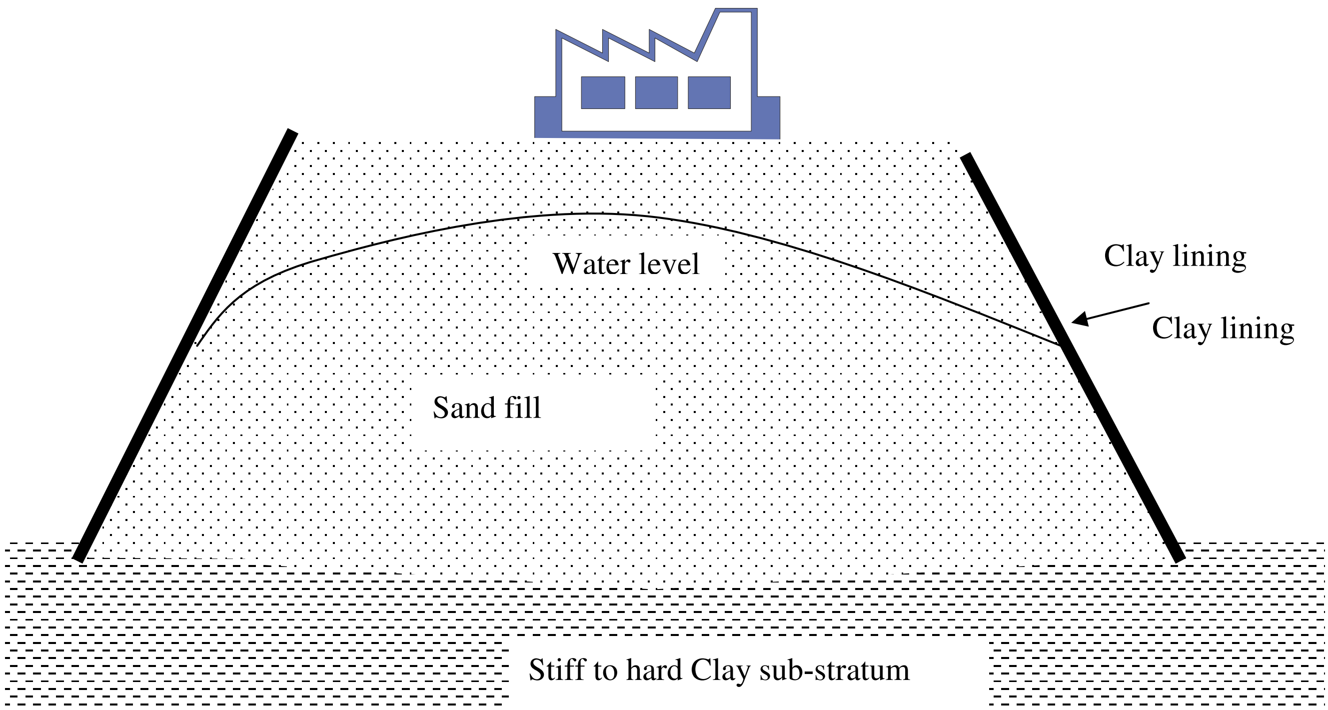


Fig. 2: Sketch of the sandfill surcharge over the clay stratum and perched water table

After construction had progressed to over 70% completion, it was realized that high groundwater would constitute a problem for the cabling and effectiveness of the clay liner protecting the sandfill. Water table rises close to the ground surface, especially during the wet season, creating challenges bordering on effective performance of the engineering structures. It was therefore imperative that a study be carried out at the Power Plant site to proffer solutions to the control of the high ground water at the site.

Methodology of Analysis

Two strategic options for solving the high groundwater level were considered: Option 1 was the drilling through of the stiff clay, creating a permeable sand column to hydraulically connect the impounded water with the groundwater. This option by-passes the impervious clay and creates an avenue for the high water level in the sandfill to drain through the underlying sand. The problem however is that it may not effectively lower the ground water level in the sandfill at peak flood season because of the negligible hydraulic head. Besides, the highly pervious stone columns may serve as preferred travel paths for contaminants to migrate into the ground water, which is a sensitive receptor. This option was therefore both ineffective and environmentally unwise.

The Second option, which is the preferred, considered the use of a network of pumped wells and availability of safe receptacle for channeling pumped water. This option satisfied the need for both effective functioning and environmental best practice.

To implement this option, a total of 5 boreholes were drilled to the depth of 7m below the natural ground surface. Four (4No.) of these boreholes (piezometers) were aligned linearly with an interval distance of about

15m, 8m and 5m from each other (to make possible, the observation/of monitoring the lateral extent of the cone of depression from the center of the well ie radius of influence, as the well is been pumped).

The 5th borehole was not cased and was drilled close to the edge of the embankment area to observe possible changes in lithology caused by seepage, arising from the flow towards the embankment and consequent blockage of flow path by fine sediments.

Borehole drilling was executed using the cable tool percussion method. Both disturbed and undisturbed samples were recovered from the boreholes. Undisturbed samples were taken with the help of U-4 tube in the cohesive soil, while the split spoon was used for undisturbed sampling in the cohesionless soils. Furthermore, eighteen (18No.) Excavations to 3m depth were executed at pre-selected points to observe the ground water level and the influence of the cone of depression in the site during pumping test. The ground water level which was about 2.5m below the ground surface was monitored for 3 days as pumping test was continued in the piezometric boreholes. The pit which was about 3 x 4m wide and about 2.5 – 3m deep was closed after the monitoring exercise.

Soil samples were collected at every 0.5m interval and whenever a change in lithology was observed. The samples were subsequently described with preliminary classifications based on colour, texture, grain density and other observations. The collected samples were also subjected to a range of laboratory analysis including; particle size analysis, permeability test, bulk density, and moisture content etc.

Well Construction:

The borehole wall was cased by a 300mm, (12" OD) steel pipes as drilling progressed in order to prevent the borehole wall from collapsing. Upon completion of drilling to required depth, a 8" ϕ, 12 bar, PVC screen (machine slotted, with 1mm slot size) and PVC casing (8" ϕ, 12 bar) were lowered into the 12" ϕ steel cased hole. The 12" ϕ steel casing were then retrieved to exposed the screen zone to the formation material. In order to minimize possible infiltration of fine particles into the boreholes, permeable geo-textile material was used to rap round the entire perimeter of the screen. Particle size analysis from a previous investigation (Groundscan 2002) indicated that between 70% and 80% of the aquifer material was less than 1mm (Fig. 5), suggesting that *in-situ* sand could pass through the screen slot in the absence of geo-textile materials.

A total of 5m of screen lengths were installed from the borehole bottom, leaving 1.5m blind casing length to the surface. The borehole annulus between the screen and the borehole wall was gravel packed from bottom to the zone above the screen level (6m from bottom). The remaining 0.5m length of borehole annulus to the ground level was sealed by a bentonite slurry. The borehole was completed with a bottom shoe and a surface casing cap. The casing protruded about 0.2 – 0.5m on the ground surface.

Well Development and Water level Measurement

Pumping test was carried out in the boreholes, with one borehole serving as pumping well, while the others served as observation wells. One Horse Power (1Hp) pump was installed complete with a flow meter and delivery unit. The pumped water was discharge through the delivery unit to a far distance (50m) to avoid recharging the ground water within the cone of depression (Plates 1 to 3 show different scenarios of Well development) .

Constant rate test (by pumping water at designated capacity to determine the time drawdown and distance drawdown) was done. Also step drawdown test was carried out (by pumping water at progressively increasing fractions of the maximum discharge capacity. The pumping rate was related by a flow value and the pumping was increased progressively when equilibrium / stability in delivery in water level measurement was obtained at each step.

Pumping was done for about 72hrs or more. Water level was measured in the pumping well and also in all the three observation wells.

Water level measurements was made at the following times:

- Immediately before well discharge was started.
- Minutes intervals until completion of about 2hrs.
- Every hours interval thereafter until the completion of ± 72 hours of pumping. Time-drawdown and distance drawdown data were generated.

The boreholes were developed by air lifting / surging using a 750cfm air compressor machine. This was done for about 1 hour, until fresh water from the borehole (formation water) is collected. Water samples were collected in a sterile bottle for physico-chemical analysis.

Results

The site is located in the flooded back swamp of the River Niger which is underlain by alluvial plain deposits, consisting of a sequence of silty clays, clays and sands. The area is regionally within the clastic sedimentary deposits of the coastal plain sands of the Niger Delta and dominated by fresh water swamp, as a consequence of being flooded in the rainy season due to impoundment of water by the presence of an impervious surface stiff-hard silty clay soil. In the dry season, water table is lowered down to about 9m below ground surface. The presence of the clay creates a disconnect between natural groundwater and the perched water above the clay. The lithological succession and engineering properties of the soil are illustrated by Fig. 3. The sandfill is underlain by stiff to hard silty clay with fibrous material and wood that is (8– 9.5m) thick.

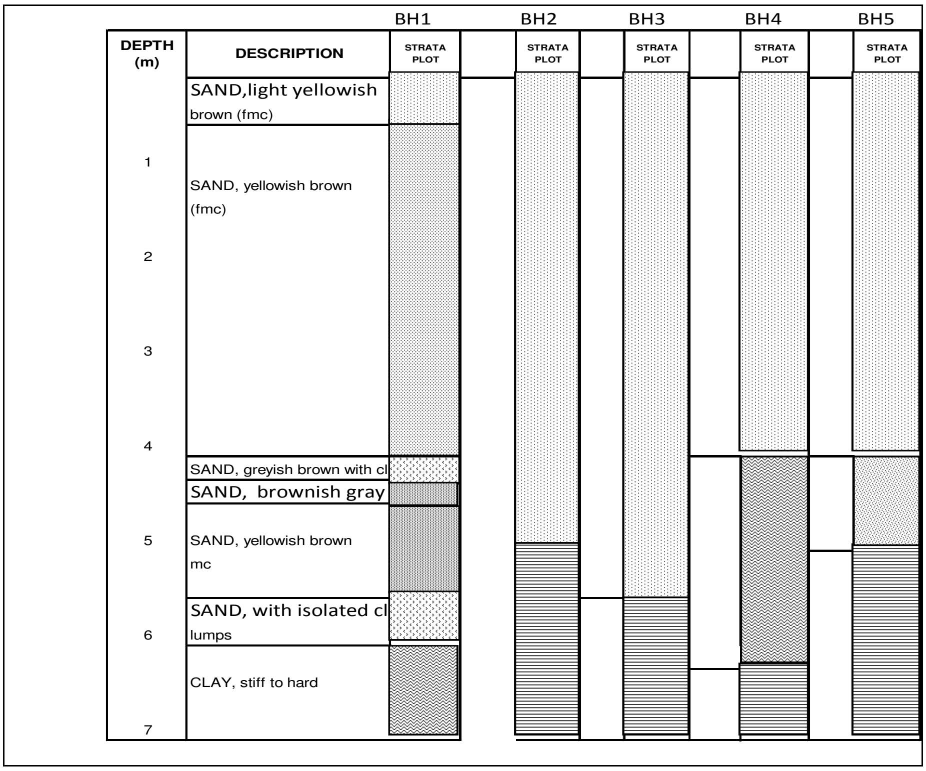


Fig. 3: Composite stratigraphy of the site from the boreholes

The integrity of the clay liner was breached at several places by the high hydrostatic pressure, resulting to seepage erosion and failure of the embankment. Comparatively, the hydrostatic pressure generated by a 2m rise in water level in the embankment was enough to breach the clay liner protecting the sand. Secondly, high water level interfered with cabling and other electrical installations.

The placement of 5m thick sandfill with equivalent vertical effective stress of 18x5 km/m², over the compressible stiff clay, caused a sagging of the base of the sandfill by way of consolidation settlement by an estimated 21mm at the centre and 14mm at the periphery. Analysis of the consolidation profiles indicate that consolidation was limited to the top 5m of the clay (Fig 4). This is because the stress increment induced by the sandfill at depths below 5m was insignificant.

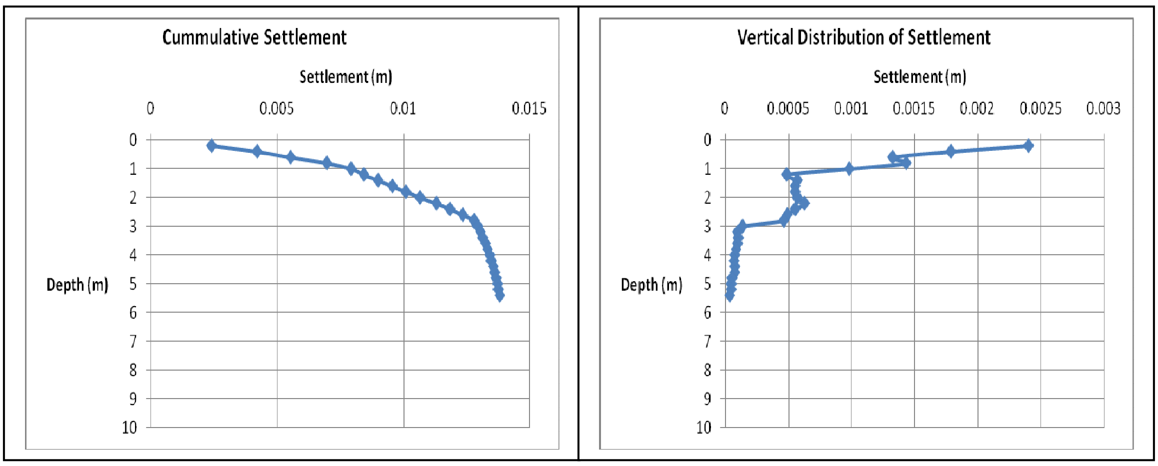


Fig. 4 (a) Total setlement profile at the periphery due to placement of sandfill over stiff clay and (b) Distribution of settlement with depth in the underlying clay

The embankment sand which was roughly 4.5 – 5.5m thick is well graded with small gravelly content, very loose – loose, brownish, fine-medium coarse (Fig. 5). The sand is mixed with clay in places. The attributes of the sand (D_{10}, D_{50}, Uniformity and coefficients of curvature) are summarized in Table 1. Empirically derived coefficient of permeability using $K = CD^{10}$ m/s is similarly presented in Table 1. Groundwater level was uniformly at 2.5m in BH1, BH 2, BH3 and BH4.

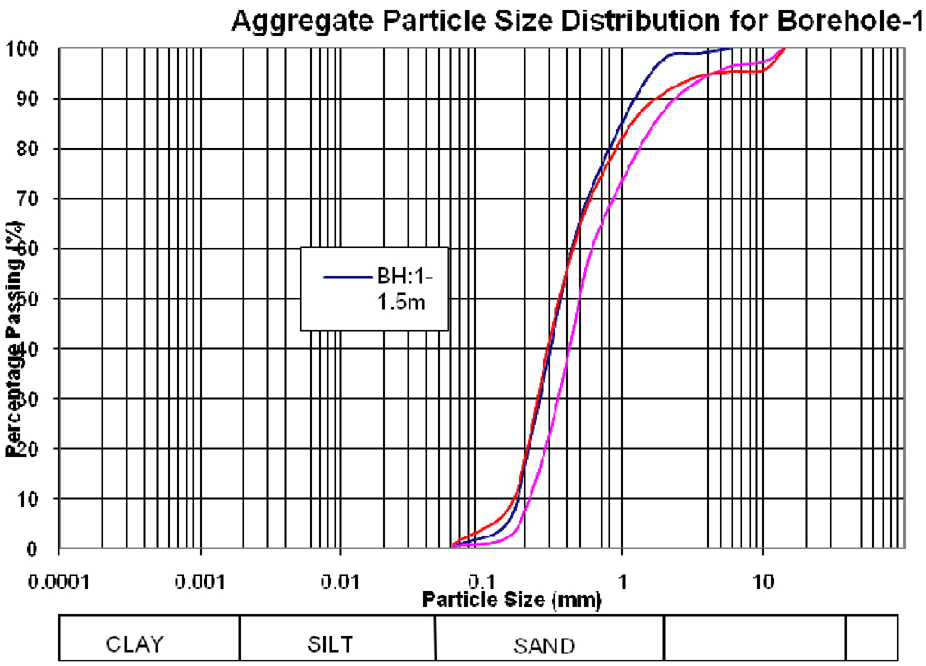


Fig. 5: Particle size distribution of the sandfill

Determination of Coefficient of Permeability in the Field

The goal of the study was to lower the groundwater level as not to interfere with construction and performance of the industrial facility. The layout of the boreholes (pumped and observation wells) facilitated determination of *in-situ* permeability. Although there are several ways of determining permeability in the field, evaluating the pumping from wells is the most commonly applied. Such tests usually involve flow over wider areas and therefore tend to be more representative (Kruseman and De Ridder 1983).

Figure 6 illustrates the well, where the permeable sandfill layer is underlain by an impermeable clay stratum. The coefficient of permeability of the top permeable layer can be determined by pumping from a well at a constant rate and observing the steady-state water table in the nearby observation wells (Kesler and Raad 1983). The steady-state is established, when the water level in the test well and the observation wells have become constant, as widely expected after 24hours of pumping in the case in question.

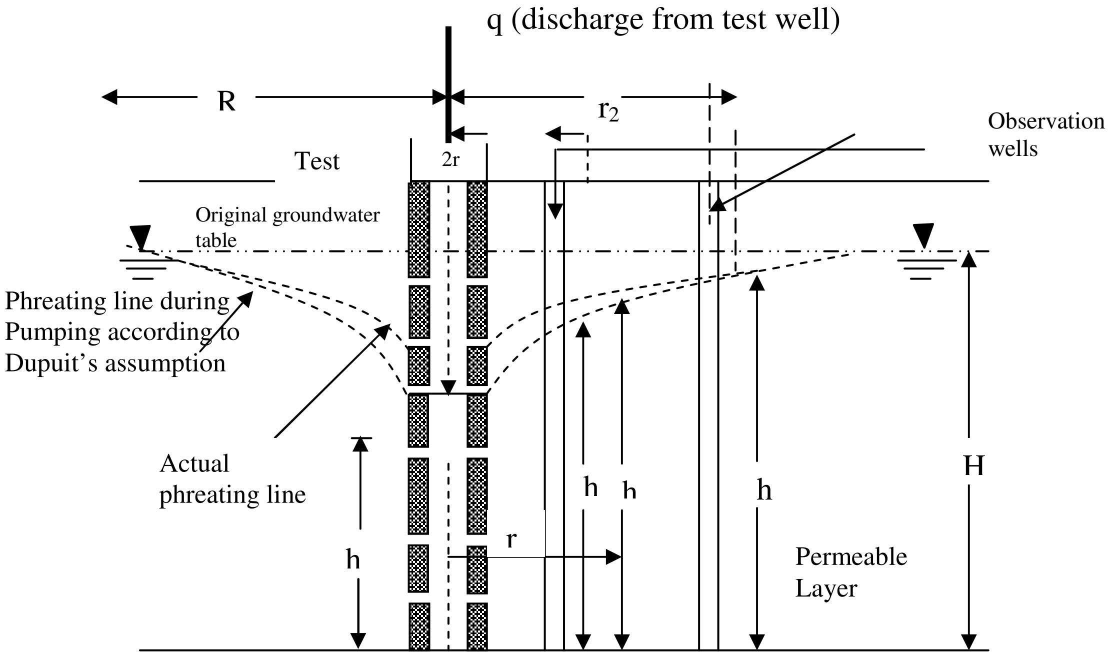


Fig. (6) Illustrating groundwater levels during pumping from well

The rate of discharge due to pumping, at steady state in the case, where the relationship for the coefficient of permeability satisfies the assumption that the well fully penetrates the permeable layer may be expressed as:

$$q = K\,i\,A \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 1$$

From Fig (6) i is approximately equal to $\frac{dh}{dr}$

while $A = 2\pi rh$;

Substituting these in the equation for rate of discharge, we obtain;

$$q = K \frac{dh}{d_r} 2\pi rh \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2$$

Re-arranging and integrating of the equation within the limits r_1 to r_2 and h_1 to h_2 would yield:

$$\int_{r_1}^{r_2} \frac{d_r}{r} = \frac{2\pi k}{q} \int_{h_1}^{h_2} hdh. \quad \ldots\ldots\ldots\ldots\ldots\ldots 3$$

A further reduction of this equation results in:

$$K = \frac{2.303\, q \log (r_2/r_1)}{\pi (h_2^2 - h_1^2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4$$

An average coefficient of permeability of 1.2 x 10^{-3} m/s was calculated from this equation, with the substitution of values of r_1, r_2, h_1, h_2, and q already determined from field measurements. Estimates of permeability were also determined from the simple empirical relationship (Freeze and Cherry 1970) $K = CD^2_{10}$, based on particle size distribution of samples from the boreholes and tabulated in Table 1. Ironically, the values obtained by both field measurements and by empirical relationship were comparable to within limits of 0.002m/s

The maximum radius of influence for draw down arising from the pumping (Bear 1979) can be given by as:

$$R = \sqrt{\frac{12t}{n}\sqrt{\frac{qk}{\pi}}} \quad \text{……………………………………………..5}$$

where n = porosity

q = discharge

R = radius of influence

t = time during which discharge of water from well has been established

If we substitute $h_1 = h_w$, at $r_1 = r_w$ and $h_2 = H$ at $r_2 = R$, then

$$K = \frac{2.303\, q\, [\,\mathrm{Log}\ (R/r_w)]}{\pi\,(H^2 - h_w^2)} \quad \text{…………………………….………6}$$

Where H is the thickness of the sandfill less the depth to water table, ie depth of the original groundwater table from the impermeable layer.

The depth h at any distance r from the well ($r_w \le r \le R$) can then be determined from equation (6) by substituting $h_1 = h_w$ at $r_1 = r_w$ and $h_2 = h$ at $r_2 = r$

Thus $$K = \frac{2.303\, q\, [\mathrm{Log}\ (r/r_w)]}{\pi\,(h^2 - h_w^2)} \quad \text{………………………………..7}$$

Which when re-arranged translates to:

$$h = \sqrt{\frac{2.303q}{\pi k}\log\frac{r}{r_w} + h_w^2} \quad \text{…………………….8}$$

Since the permeability was now determined, the radius of influence also known it becomes imperative to select a suitable pump with discharge capacity adequate to achieve a drawdown of (h) at the well point. The next discharge well point could now be located at the margin of the radius of influence, providing minimum overlap and ensuring cost effectiveness. The combined pumping in the two wells maintained the water level in the well points at levels in the order of (h), well below the zone considered dangerous to cables and the clay liner. In this way the problems associated with both water level and cabling was effectively checked.

Conclusions

This paper describes the considerations made in providing a solution to high groundwater level at a sandfilled construction site underlain by highly impervious stiff clay soil. Based on the considerations made in this paper, it is concluded that a network of pumped wells can indeed effectively control high water level. The determination of the cone of influence of a pumped well is critical to the planning and strategic location of the network of pumping wells by ensuring minimum overlap, and by extension efficiency through cost effectiveness. The values of coefficient of permeability obtained by both field measurements and by empirical relationship were comparable to within limits of 0.002m/s indicating that the flow rate in sandfilled areas may be insensitive to method of determination.

Table 1: Particle size distribution attributes

BH No	**Depth**	**D_{10}**	**D_{30}**	**D_{50}**	**D_{60}**	**$Cu=D_{60}/D_{10}$**	**$Cz=D_{30}^2/(D_{10}*D_{60})$**	**$K=C*D_{10}^2$**
	(m)	**(mm)**	**(mm)**	**(mm)**	**(mm)**			**(m/sec)**
1	1.5	0.18	0.26	0.35	0.42	2.33	0.89	0.00324
	3	0.22	0.34	0.5	0.6	2.73	0.88	0.00484
	4.5	0.17	0.25	0.35	0.42	2.47	0.88	0.00289
2	3	0.22	0.35	0.9	1.5	6.82	0.37	0.00484
3	4	0.19	0.26	0.36	0.44	2.32	0.81	0.00361
4	4	0.19	0.28	0.37	0.44	2.32	0.94	0.00361
5	3	0.17	0.24	0.31	0.4	2.35	0.85	0.00289
	4.5	0.08	0.18	0.24	0.26	3.25	1.56	0.00064
Mean		**0.1775**	**0.27**	**0.42**	**0.56**	**3.07**	**0.90**	**0.00332**
SD		**0.044**	**0.055**	**0.206**	**0.391**	**1.547**	**0.322**	**0.0013281**

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Plate 1: Well development using a compressed air

Plate 2: Two of the Well used in Pumping test analysis

Plate 3: Another two Wells drilled for the purpose of groundwater control at site