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Tank Pad foundation Construction in the Niger delta, Nigeria

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Abstract

Large storage tanks are important oil and gas production infrastructure. However, due to widespread occurrence of weak and compressible soils, and the inevitable consequences of foundation failure, extreme care is exercised in the design of its foundation. More so in the Niger delta, characterized by a variety of soils including tropical lateritic soils that are known to exhibit lateral and vertical variations in strength and compressibility, as well as effects of seasonal changes in its moisture levels. The necessity for ground improvement is determined by soil peculiarity and the sensitivity of the structure to be placed on the ground. The tropical lateritic soil with seasonally dependent strength (50kN/m² - 110kN/m²) was considered inadequate for the foundation of a large storage tank with diameter ø 76m and 22m high transmitting about 10^6kN when fully loaded, because of anticipated high total settlement which is unacceptable for the rigid skid systems. This paper describes the geotechnical and environmental considerations in the design of foundations. The choice of sand replacement for the foundation of the large storage tanks and details of the procedures, particularly the quality control steps adopted to ensure a satisfactory foundation performance are also described.

Keywords: Tank pad foundation, lateritic soils, bearing capacity, settlement

1 Introduction

Storage tanks are widespread in Nigeria, especially in the Niger Delta where continuous production of crude oil and refining is taking place. However, large storage tanks with diameters in excess of 76m are rare and require highly competent load bearing layers to minimize settlement in order to secure the rigid skid system. Foundations for such sensitive structures demand a careful ground exploration and design especially in cases where the underlying soils exhibit seasonal variations in engineering properties.

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Figure 1: The Niger delta

The Niger delta (Fig. 1) consists entirely of sediments at different stages of lithification within shallow depths of engineering significance (0-40m). These sediments or soils as commonly referred to have been classified into four geomorphic units (Short and Stauble, 1967, Akpokodje, 1979) recognizing their origin, composition and geography. Apart from the Beach ridge which occupies less than 10% of the land area, the soils are inherently weak and compressible in their foundation bearing characteristics (George and Abam, 1992). Lateritic soils which falls into this category presents challenges in construction (Malomo, 1979), one of which is the capacity to lose 80% of their strength when wet (Enoch George Associates, 1993).

Ideally, the foundation of structures sensitive to settlement such as large tanks are designed on piles, especially in areas with the prevalence of thick compressible layers which are prone to settlement and exhibit engineering properties that are season dependent. However where the thickness of clay is large, the use of piles may be considered uneconomic. In many sections of the mangrove swamp, for example, piles may be considered uneconomic for foundation of large tanks, because of the very thick layer of soft compressible clay material (greater than 40m in some cases). In this case, pre-loading with sand, dredged from a near-by river estuary can be carried out (Jones D.B. et al., 1987, George and Abam, 1992) in order to consolidate and improve the bearing characteristics of the underlying layers.

The problem with pre-loading is that consolidation of the underlying layers is slow because of the unfavourable combination of thickness of the compressible materials and low permeability of the clay sediments. In order to accelerate the consolidation process, vertical drains are installed (Yeung 1997; Taube 2014). In sections of the Niger delta, this may be considered environmentally unsafe, since the highly permeable vertical drains could also encourage the ingress of saline water and petroleum contaminants into the underlying aquifers.

Lateritic soils on the other hand, sometimes referred to as coastal plain sands comprising sandy clay or clayey sand are widespread and cover more than 50% land area. This soil present ground surfaces that are mostly firm and can mobilize sufficient strength and bearing capacity to support relatively small tanks of diameter ø less than 36m as well as one or two storey buildings on shallow foundations without the need for ground improvement. However, due to the high stress induced by the large tank (220kN/m^2), these soils are considered incompetent and therefore inadequate for the foundation. This paper describes the design of ground improvement and procedures adopted, together with the quality control steps put in place to ensure satisfactory performance of large storage tanks in lateritic soils of the Niger delta.

2 Site Characterization

Since the strength and behavior most soils in the Niger delta are dependent on the presence of water, it was considered important to understand rainfall distribution and its relationship to groundwater level variations. Accordingly, a domestic meteorological station was set up to obtain rainfall data for planning. At the same time, monitoring wells were installed to observe groundwater levels continuously for a period of 18 months to provide data for planning excavation works. Also, records of over 250 geotechnical borings and 200 CPT soundings obtained at different times (in accordance with BS 5930), cutting across 3 geomorphic units, namely; mangrove swamp, lateritic soils and flood plain sediments were analyzed in order to gain insight of variabilities of geotechnical parameters in this area, and in particular to determine the soil stratigraphy and geotechnical characteristics of the sediments.

A series of classification, strength and compressibility tests along with chemical analysis were also carried out in accordance with ASTM D 698, D448, D4318 and BS 13477 (1990). Undrained shear strength for the sandy clay was determined using unconsolidated undrained triaxial compression tests. Chemical tests were performed to evaluate the reactive and corrosive characteristics of the soil and water samples. These tests included determinations of sulphate, chloride contents, conductivity and pH values.

3 Results and Discussion

The soil stratigraphy at the region was determined from the results of the borings, soil sampling, in-situ penetration test and laboratory test. The typical stratal sequence which comprise soft to very soft silty clay for the mangrove swamp (3-7m thick), soft to firm sandy and silty clays (6-12m thick) for the lateritic soil and 1-3m thick clayey sand in the flood plain area overlaying a predominantly sandy formation is essentially consistent with the immediate construction area although variations in the thickness of the layers were observed.

For the specific case study site, a firm to stiff clay stratum of more or less uniform thickness (0 -10m) underlie the site and this in turn is underlain by a prevalent deposit of medium dense to dense sand to a depth of about 42m. Beneath the sand deposit is stiff to hard clay stratum which in places grades laterally to a dense sand.

For shallow foundations, the upper clay is the most pertinent and needs to be discussed in detail. This clay layer in general has soils classification varying between CL and CH. The clay is brown to reddish brown in colour becoming yellowish towards the base of the stratum. Despite the fact that it contains a high sand content, the engineering behavior of the deposit tends strongly towards that of clay as the fine content has a dominant influence.

The rainfall distribution and groundwater response in the area were explored (Fig. 2). Daily and monthly rainfall records during the year preceding commencement of construction were obtained to plan the phasing of construction. Superimposed on the rainfall distribution is the variation of average ground water level deduced from boreholes at the site.

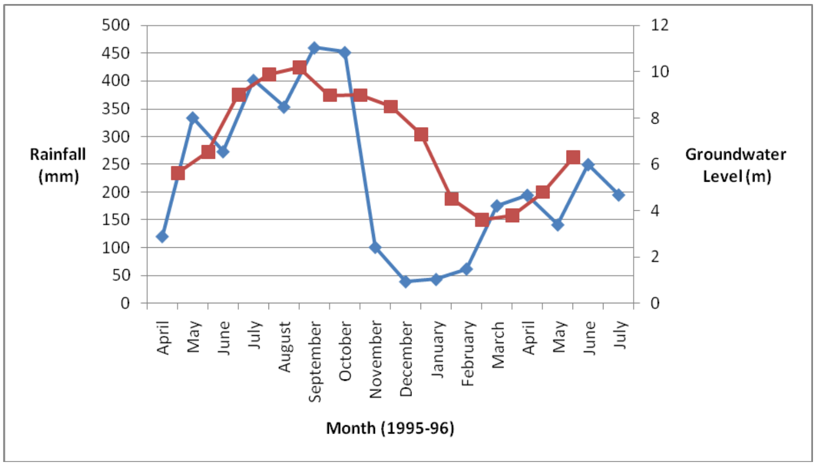

Figure 2: Rainfall distribution and groundwater response at the PortHarcourt Refinery

Comparison of the rainfall and water level trends showed that there was an initial lag in the response of the ground water level. Its peak coincided with the peak of the rainfall. This is understandable, since the rainfall at the inception of the rainy season is expected to infiltrate and make up for the moisture in the vadose zone caused by the prolonged exposure of the soil to dry season. Once saturation in the soil was achieved, soil moisture became a continuum and was therefore able to respond instantaneously to precipitation.

The maximum rise in the water table was compared with the design elevation of the excavation in order to anticipate the incidence of water related problems of drainage or uplift pressures for underground tank or concrete bases. The relatively low rainfall amount between November and April (less than 125mm) suggested the concentration of earth works involving excavations and compaction of the lateritic soil during this time window. At this time, precipitation is low and easily permits the achievement of 95% relative compaction of lateritic soil. This is because only a small range in moisture variation 15- 20% can be tolerated for the densification of the lateritic soil to meet the 95% relative compaction requirement. Besides, sediments mobilization by runoff was minimal at this time. The strict environmental control on site required that, as much sediment as possible be conserved in their in-situ position to avoid problems of contamination in downstream locations.

Since it was also of interest to the project that contaminant transport rates be estimated for spill containment planning, groundwater levels were further analyzed further to derive the flow direction (Fig. 3) and velocity. Groundwater flow velocities between 1. 086 cm/day were obtained.

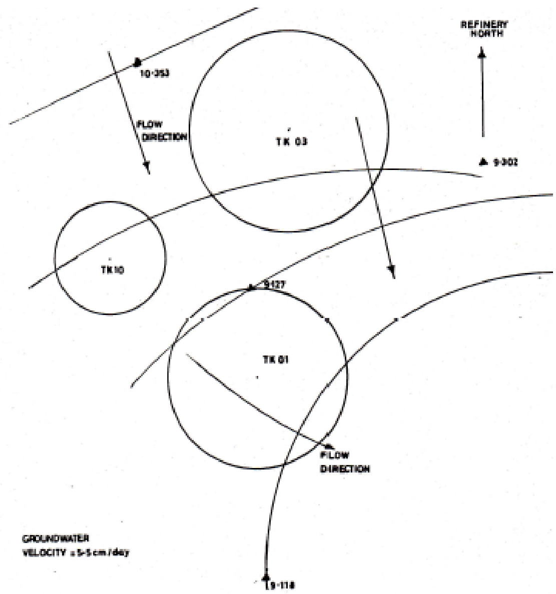

Figure 3: Groundwater flow direction at study site

The results of the unconsolidated undrained tests for the soil revealed that the lateritic soils were mostly unsaturated with angles of internal friction sometimes considerably higher than zero, reflecting the significant sand contents. For a founding depth not greater than 6.0m, shear strength parameters of Cu = 80kN/m² and $ø^1$ = 10° were considered appropriate. Coefficients of volume compressibility m_v were obtained from the results of one-dimensional consolidation test and compared with the relationship $1/m_v$. Cu = 100 which is appropriate for this soil (EGA 1992).

Using the CPT soundings, vertical profiles of undrained strength, allowable bearing pressure, California Bearing Ratio (CBR) and cumulative settlement distribution were derived based on empirical correlations (Robertson, 2009), exemplified by Figs. 4 and 5.

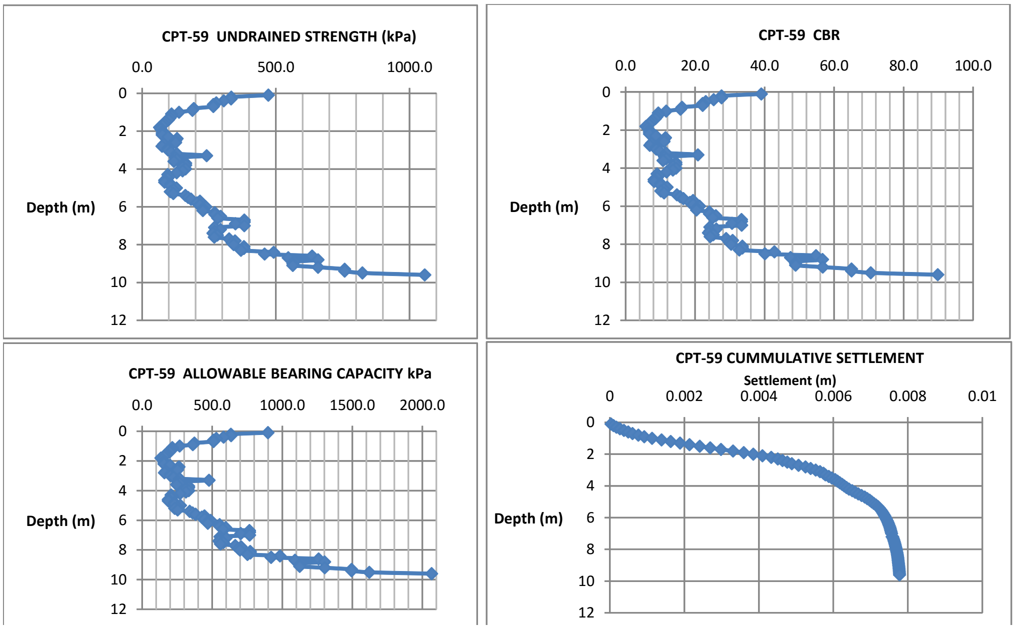

Figure 4: vertical profiles of undrained strength, allowable bearing pressure, California Bearing Ratio (CBR) and cumulative settlement distribution at location CPT59

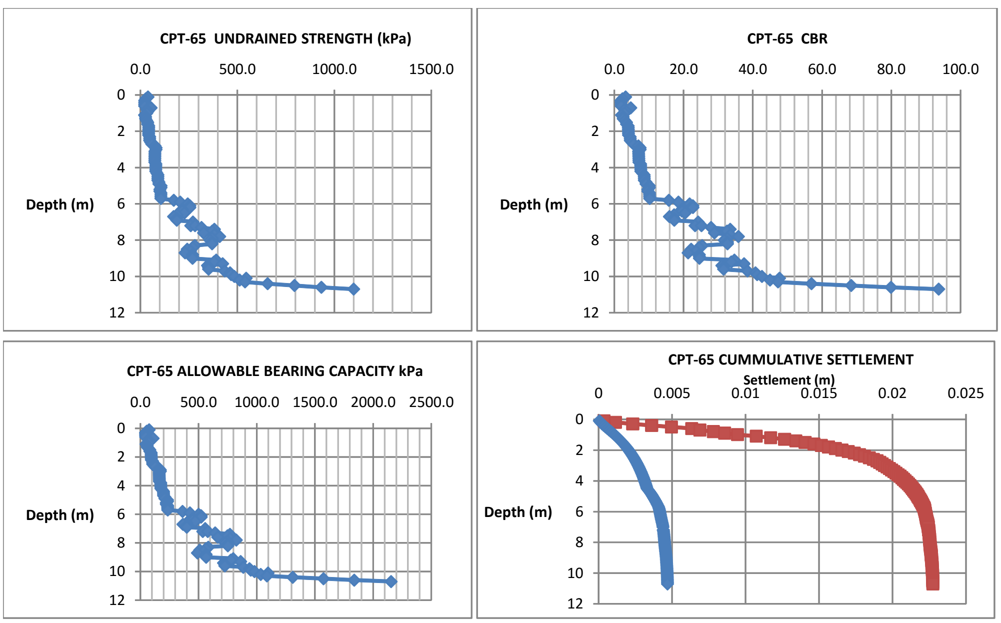

Figure 5: Vertical profiles of undrained strength, allowable bearing pressure, California Bearing Ratio (CBR) and cumulative settlement distribution at location CPT65

The cumulative settlement is only the incremental summation of settlement at intervals of the manual CPT readings of 20cm resulting in a vertical settlement distribution typified by Fig. 6 as in the case of location CPT59. This figure clearly indicates that nearly all the settlement is contributed from 0-10m depth. It also shows that significant reduction in settlement following replacement of the *in-situ* soil.

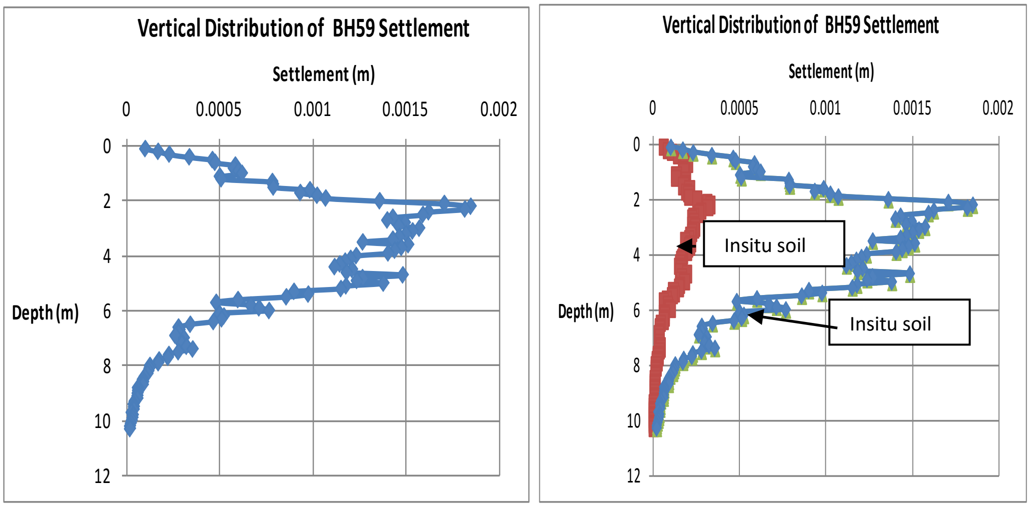

Figure 6: Vertical distribution of settlement before and after soil replacement

The broad spectrum of soils in the Niger delta exhibit different characteristics as shown in Fig. 7 for mangrove swamp, lateritic soils and floodplain sediments.

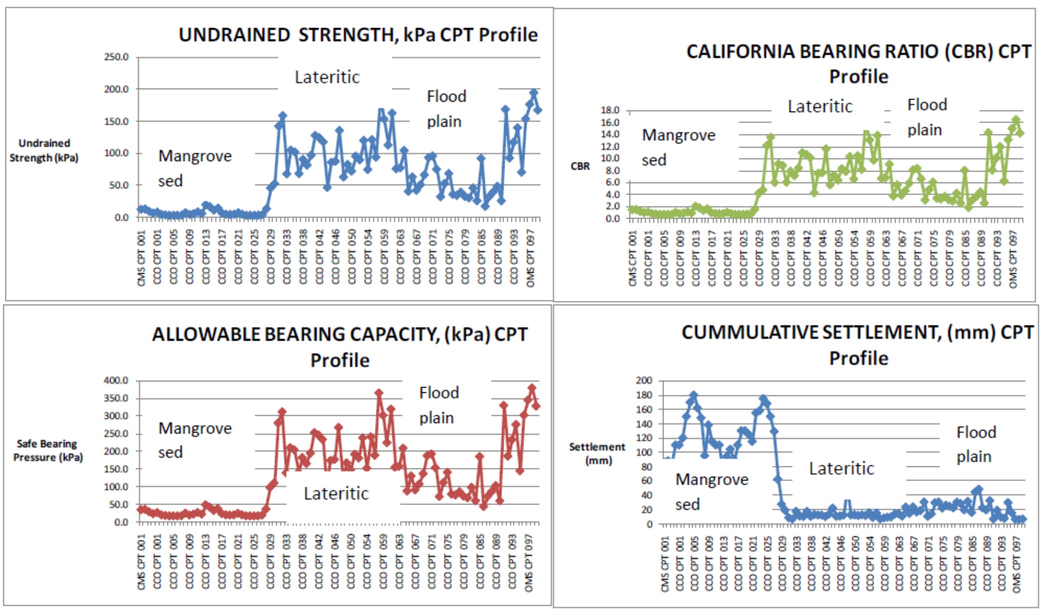

Figure 7: Geotechnical characteristics of some major soils in the Niger delta

It is evident that comparatively, the mangrove swamp soils exhibited the least desirable characteristics in terms of foundation bearing capacity, CBR and settlement, followed by the floodplain sediments, and then by lateritic soils, especially in well elevated areas.
The co-efficient of volume compressibility m_v was found to decrease from $0.3m^2/mN$ at the top of the deposit to about $0.1m^2/mN$ at 6.0m depth. Although this soil is not susceptible to significant swelling it softens rapidly when saturated with subsequent loss in bearing strength. For this reason, it practitioners recommend that provision be made to

avoid the ingress of water into the foundation soil during and after the foundation construction.
Settlement computations showed that total settlement of order of 1000mm to 1450mm could be expected for loads as small as $30kN/m^2$. Much higher total settlement values have been recorded in the mangrove swamp (TNGPL 2015). These settlement values are considered excessive for large storage tank structure because it would endanger the rigid skid systems linking the tanks and pipeline works and create resultant safety issues. This makes soil improvement designed to ensure minimum settlement and improved bearing characteristics of the foundation imperative.

4 Soil Improvement

Computation of vertical settlement distribution in most lateritic soils in the Niger delta show that a large part of the total settlement is contributed from the top 6m. At the same time, the bearing strength derivable from this upper clay is usually unsatisfactory. Consequently, it the shallow part of the upper clay layer is frequently excavated and recompacted or replaced with granular materials to improve both strength and compressibility performance. In order to determine whether recompaction would produce desirable improvements, compaction tests on this lateritic sandy clay was carried out and revealed that it was medium plastic with a significant clay content (greater than 20%) and with a sharply defined compaction curve characterized by a sharp peak. Effective recompaction of this sandy clay soil is difficult in a place such as the Niger Delta. This is because the small moisture content range over which 95% relative compaction can be attained is difficult to achieve.
The frequent heavy rainfalls, with annual average of 2500 mm (Gobo, 1990; Adejuwon, 2012) concentrated between April and September, sometimes increases the moisture content to over 70%, which will then require another three (3) days of continuous sunshine following such heavy rainfalls, to lower the moisture content to the permissible range to ensure satisfactory compaction. This invariably leads to delays in scheduled activities with all the accompanying cost penalties. The consideration of the high rainfall at this time (35mm/month) and the difficulties of construction which this would create in terms of the workability of the clay, sediment flow and moisture control in the compaction process together with the cost penalties consequent upon delay in the time achievement of set milestones, would readily justify the use of granular sand as backfill.
The use of sand has the advantage of rapid drainage, and good workability even in near saturated condition. This is made possible by the high permeability (1.5×10^3 m/sec) and the conhesionless nature of the sand. The compaction of samples of the sand from a number of burrow pits showed that it could effectively be densified over a wide range of moisture conditions as determined by Abam and Ofoegbue (2000).
Following determination of the most compressible zone, foundation excavations were commenced with the largest covering a land area of 81m diameter to a depth of 4.7m, using essentially excavators and pay loaders. An excavated slope of 45 degrees was maintained around the excavation wall, except at the ramp where an inclination of 10 degrees was used to allow for plant traffic. The excavated floor dipped mildly (1:100) towards a corner provided with a drainage sump. Two other sump were provided to cope with the occasional heavy rainfalls and consequent flooding of the excavation.

Foundation excavation was quickly followed sand backfill sourced from six burrow pits, all within 50km radius. Materials from each source were tested for compliance with the specified particle size distribution envelop. Laboratory maximum dry density and optimum moisture contents were determined for each compacted layer. Also, the pH, chloride and sulphate contents were determined to ensure that potentially corrosive sand sources were not used. Average values recorded for the area are summarized in Table (1).

Table 1: Compaction and physicochemical characteristics of some sand deposits in the Niger delta

Sources	Max.Dry Density (Mg/m^3)	Optimum Moisture Content (%)	pH	Chloride (ppm)	Sulphate (ppm)
Eyaa 1	1.805	12.0	6.7	119.8	18.6
Eyaa 2	1.740	12.4	7.0	108.2	12.9
Elelenwo 3	1.795	13.4	6.9	2.6	1.0
Choba 4	1.80	11.9	7.0	1.7	2.1
Akpajo 5	1.801	12.9	7.0	3.5	1.6
Akpajo 4	1.803	12.7	6.9	2.1	1.5
Akpajo 3	1.82	11.5	7.8	175.4	20.7
Akpajo 2	1.76	13.0	7.6	345.8	35.8

The trial field compaction was carried out to determine the suitability of the roller and the number of passes required to achieve 95% relative compaction in comparison with the laboratory maximum dry density. The trial compaction also helped to determine the optimum thickness of sand lift to ensure effective field compaction, which in this case was determined to be between 200 mm-250 mm.

The sand layers were then compacted using a vibratory Bomac roller, plying two principal trajectories (Fig. 8) to ensure uniform compaction and full coverage of the tank area. The degree of compaction was then determined in four to five positions selected by a quality inspector, using the sand cone method and level-checked with Kern automatic levels. The maximum dry density for each of the compacted layers was consistently high with values between 98.3 to 101%, well in excess of the 95% of the laboratory maximum required by specification and with corresponding optimum moisture content range between 8.9 – 14.1%.

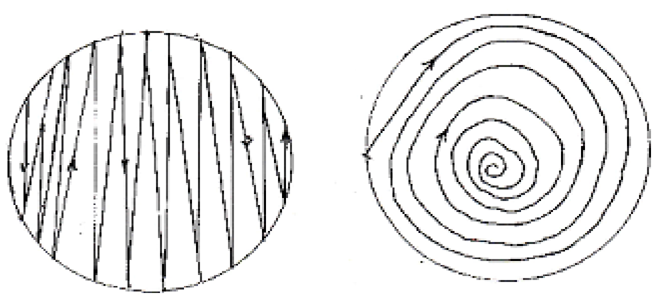

Figure 8: Two principal trajectories to ensure uniform compaction

The compaction of the sand backfill was not perceived as a problem for two reasons. Firstly, the 18% moisture continent range i.e. from 4 to 22%, over which densification of the sand to 95% relative compaction can be achieved is large. Consequently, rainfall was not likely to contribute to increments in moisture content in excess of 22%. Even when this happens, the high permeability of the sand ensured that it could be drained rapidly whenever the site experienced rainfall greater 100mm. Since the optimum moisture content is very low and was easily exceeded whenever it rained; the significance of permeability of the backfill material was regarded as crucial.

After about 21 such layers of sand, amounting to a sand thickness of about 4m, a ring wall was commenced using 0-50mm crushed stones. Again the crushed stone was, tested both at production base and on site periodically, to ensure compliance with the specified gradation envelop, degree of compaction and chemistry. The trial compaction in this case was carried out on an arc-strip of crushed stones which realistically models the path of the roller in the compaction of the ring wall. In this arrangement, the trial rolling predicts the performance of the roller more accurately. Several layers of crushed rock were compacted in lifts of 250 mm.

At an appropriate elevation below the ring walls an impermeable, 4mm thick polythene membrane was placed. Adjacent polythenes were overlapped by 0.5m and dipped (1:50) towards the perimeter. The polythene was to prevent contaminations arising from leaks in the tank to be extended to the sub-soil. Instead, such leaks are rapidly transmitted to the periphery at an estimated velocity of 2.8 x10^{-3} m/sec where a leak detection pipe is fitted. The pipe end intercepts the membrane in a gravel filter ditch created at the inner lower end of the ring wall (Fig.9). The membrane is protected from damage by a sand overlay, which is then capped with a 100mm thick 0 -50mm crushed stone covering the entire tank area. This provides a suitable surface to place and compact sand asphalt in final preparation to place the tank (Fig.10) .

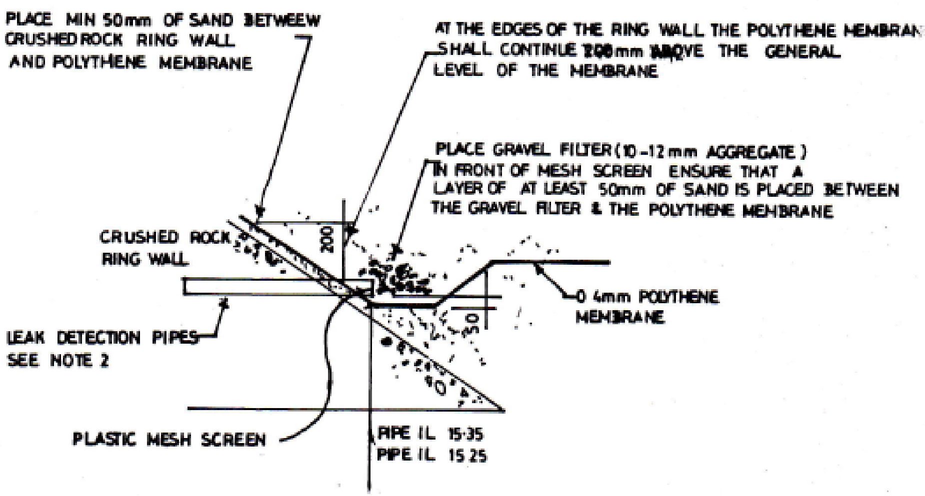

Figure 9: Details of Polythene membrane gravel filter and leak detection pipr connection

Figure 10: Completed Tank Pad foundation

5 Conclusion

Geotechnical factors affecting the design and construction of sensitive civil structures in weak and compressible soils have been described. In particular, the paper describes the procedure for foundation construction for a large storage tank involving replacement of underlying lateritic soil with cohesionless sand consistently densified to 95% of the laboratory maximum dry density in loose lifts of 200 mm.

The paper concludes that although lateritic soil may be competent for small structures such as bungalows and small diameter tanks, it is grossly incompetent for a large tank exerting 220kN/m². Planning and phasing of construction works in the tropical rain forest belt is largely dependent on the rainfall distribution while the water table may define the scope of excavation and other earth works or assist in anticipating water related construction problems. The lag between rainfall and water table response has been explained as the consequence of the combined effect of infiltration and saturation.

The careful implementation of a program of quality control is necessary to ensure that densification of sand backfill and crushed rock ring walls consistently met the specified 95% relative compaction in order to provide satisfactory foundation performance that is expected for a sensitive structure such as a large storage tank.

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