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Design and construction of tank facilities over near-to-shore quick sands

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ABSTRACT: The establishment of a refinery and a tanks farm over an area covered by loose fluvial quick sands and influenced by high tidal effects was feasible at a most cost - effective solution by means of geosynthetic reinforcement and soil reclamation of a minor superficial cover. The performance of the constructed installations in terms of bearing capacity and deformation criteria was excellent.

1 INTRODUCTION

At an area covering approximately more than 1000 acres extending parallel to the sea shore of the United Arab Emirate of Fujairah, a new refinery as well as oil storage tanks facilities were designed and constructed during the years 1990 to 1994, with further expansion planned for the imminent future. A large part of this area is twice per year flooded, during the high tide level periods, where the maximal sea water elevation reaches +3,10 m above the official lower sea water level observed at the area. In addition to that, flooding of a smaller part of the area of interest is remarked twice per day, also due to the daily range of the tide.

The specific area of interest presents a rather rare for the coastal areas of the Arabian peninsula phenomenon: The superficial soil layer consists of loose fluvial quick sands that have a very low deposition density and under the continuous influence of the cyclic effect of the relatively important marine tides present a high potential of liquefaction

The foreseen installations included the erection of a tank farm. The tanks presented a capacity from 13,00 t to 40,00 t, some of them being of the specific type of "floating roof", especially sensible to any deformation of their foundation.

An extensive geotechnical investigation program in two different stages was performed at the site, including sampling rotary boreholes, cone and standard penetration tests, site permeability tests, plate loading tests, as well as a full range laboratory testing program, aiming at the determination of the most cost - effective solution to follow for the safe construction and performance of the planned installation. The design life criteria of the project were set by the owner of the refinery to 50 years.

2 GEOTECHNICAL DESIGN AND CONSTRUCTION COSIDERATIONS

The area of interest consisted of outwash marine deposits with their upper horizons including sands with small lenses of silt of high plasticity and thin beds of shelly gravels. Those marine sands are presented as rather loose near to the soil surface, having an SPT blows number variable between N=1 to N=4 and with a maximum thickness of 4,00 m near to the shoreline, wedging out towards the inland to a thickness of 1,00 m approximately. Geologically they are described as "quick fluvial sands with a high liquefaction potential" and their mechanical behaviour resembles more to a fluid than to a soil layer. In fact, a quick sand is not really a type of material but a condition which can be prevented by appropriate measures. Any sand can become a quick sand and remain momentarily or continuously in that condition so long as a flow of water and a critical hydraulic gradient have been developed and are maintained by the special local conditions. This critical hydraulic coefficient corresponding to the limit equilibrium is:

$$i_{cr} = \frac{G-1}{1+e}$$

where G : absolute specific gravity of the sand and e : void ratio.

An approximate average value of the critical hydraulic gradient can be taken equal to unity and for an average value of G=2,65, the corresponding void ratio is then e=0,65 and the porosity n=39,3 %.

The loss of head along a flow line occurs because of friction between the water and the soil through which it flows. The soil skeleton resists this friction and seepage forces that represent differences of head or potential are exerted against the soil in the direction in which the water flows. Sometimes, the direction of the seepage forces is upwards (mainly if no important sandy slope exists) and then will be resisted by the unit weight of the buoyed sand. In this quick condition the sand loses its supporting power and any object placed on its surface will sink into it if its unit weight is greater than the unit weight of the fluid sand - water mixture.

Whenever this layer of quick sand is activated by a rapidly inflowing or outflowing tide and under the effect of vertical pressures, a status of liquefaction occurs and large quantities of finer grains are actively carried away from their initial position.

For the fulfillement of the design criteria, the following geotechnical considerations had to be respected simultaneously:

a) The bearing capacity of the foundation layer for every structure should not be less than 0,20 MN/m^2.

b) The allowed foundation differential settlements according to the existing regulations for circular steel oil tanks with flexible base were set to 0,003, expressed as an average slope of the profile.

c) The factor of safety against liquefaction was set at 1,50.

Taking into consideration the maximal expected tide elevation of +3,10, the final foundation elevation for all construction was set at +4,00, avoiding therefore any side effect of the tidal response. Due to the variable thickness and density of the superficial layer of quick sands and its "polluting" effect to even lower sandy layers, the area of interest was divided into certain zones of different lower limits of the quick sands. Three possible foundation proposals were considered and compared to achieve the most cost - effective one:

- Excavation, removal and replacement of the layer of the quick sands up to a certain depth, which would fulfill as foundation criteria a total cone resistance of 7,5 MN/m^2 (CPT). The embankment construction would be realised by means of a reinforcing geosynthetic and by well defined granular borrow material.
- Direct compaction on a topographically set network of a standard weight of 20 t using a free - falling adequate length between 10-40 m approximately.
- Vibroflotation of the superficial zone of 4,00 m till the desired compaction of 75% would be achieved.

The cost of the direct compaction method being estimated to reach 20 US$ per square meter of treated soil and of the vibroflotation to reach 12 US $ per cubic meter of treated soil (not including the cost of the stones and the embankment material), the foundation proposal calling for geosynthetic reinforcement was immediately elected and adopted for construction (excavation cost at 2 US $ per cubic meter and geosynthetic cost, placed on site 4 US $ per square meter).

Due to the difference of density and thickness of the superficial layer of quick sands from one location to another, the area of interest had been subdivided into five zones, according to the necessary excavation level for replacement and the granular fill compacted on site had a variable thickness (1,00 m to 3,50 m).Another important criterion for the subdivision of the area was the provided layout of the foreseen tanks farm, in order to avoid having the foundation of any tank over two different zones of variable excavation and replacement depth, that would lead to differential deformations. The maximal allowable foundation stress on the embankment was set at 0,20 MN/m^2, which assured both adequate bearing capacity and tolerable settlements and slopes. Although settlements on cohesionless soils are expected more or less to be immediate, the design proved the existence of a significant time dependent creep factor. Indeed, 15 - 20 % of the total calculated settlements were expected to occur at a time interval varying between 6 months and 1 year after application of the full load of each tank (influence of the silty - clayey lenses). To avoid secondary deformations, water tests at full capacity were designed for a period of 3 months with continuous observation of the occuring deformations, prior to the operation of the establishments.

3 EMBANKMENT MATERIAL SPECIFICATIONS

The material used for backfilling purposes in order to partly replace the quick sands was specified according to the AASHTO specifications for highway engineering design in road construction projects.

At the following Table 1, the grain size distribution imposed by the design is presented.

The main criteria for the selection of the potential areas of borrow materials were set according to the average distance from the site (less than 15 km), to the simplicity for excavation (mechanical means and no explosives), to the availability of the necessary quantities (300.000 m^3

Table 1. Embankment material specifications

Grain Diameter (mm)	Percentage passing (%)
60,00	100%
20,00	65 - 100%
10,00	32 - 80 %
2,00	20 - 50%
1,00	16 - 40%
0,50	12 - 32%
0,10	4 - 18%
0,074 (Sieve No 200)	<13%

was pre-estimated) and to the conformity of the existing materials to the specifications without major or costly intervention (such as sieving or washing off the excavation material).

Two various borrow material areas were thoroughly investigated by special pits and the materials recovered there were classified as A-1A, A-1-B or A-2-4, characterised as sands and gravels, non-plastic to low plasticity, well or poorly graded. Modified Proctor tests gave maximum densities varying between 22,2-23,5 KN/m^3 and optimum moisture content 7-9%, while CBR tests corresponding to 95% of compaction gave a varying index between 59-76%. Direct shear tests to samples compacted at 95% of the maximum density gave angles of friction varying between 43-47° and a certain "pseudocohesion" between 5-20 KN/m^2. As the materials were also completely free from any organic material content, they were directly accepted for backfilling purposes.

4 GEOSYNTHETIC SPECIFICATIONS

The geosynthetic design procedure involved two main functions of the fabric, separation and reinforcement, although filter criteria had also to be checked simultaneously. The most crucial parameter was the ultimate tensile stress of the fabric, calculated on the potential for lateral spreading during the initial fill placement under the action of rapid draw - down of the sea water level between high and low tide season. As the lower layer had a nearly sufficient bearing capacity (safety factor F=1,20 against 1,40 required as minimum), the ultimate tensile stress of the fabric was estimated at the level of 20 KN/m and its minimum weight set to 380 grs/m^2.

The selection of the polymer origin for the mostly adapted geosynthetic was directly influenced by the chemical characteristics of the environning water and its level fluctuations. Following extensive chemical analyses of the water quality, it was deduced that a certain hydrolytic degradation could be important in the future life of the fabric, if a bond breaking effect of the polymer reduces its tensile strength. The extent of this degradation could not accurately be determined because of the variety of the factors involved such as film thickness, crystallinity, orientation, relative humidity, dielectric constant, autocatalysis, type and number of ionizable groups, electrostatic effects and chain conformation (Reich, Stivala, 1971). With the additional help of a polymer expert, polyamide fabrics which are strongly polar and could easily be hydrolysed were rejected, while polyethylene and polypropylene fabrics having non - polar repeating units showed excellent resistance to hydrolytic degradation.

Because of the possible existence of some fine grained materials within the new embankment, the hydraulic filter stability function of the geotextile was also of a major importance, since it should assure that during the sea regression those fine particles could not be removed, causing thickness diminution and therefore additional instability. The uniformity coefficient c_u of the quick sands being lower that 2,0, the selected geotextile should present an effective opening size of $D_w < 0{,}11$ mm

Checking then the filter stability, the required minimum geotextile permeability was calculated as:

$$k_{g,req} = \frac{tg \cdot k_s}{5 \cdot d_{50}} = \frac{2{,}6 \cdot 10^{-3}}{5 \cdot 0{,}06} = 8{,}7 \times 10^{-3} \ \text{cm/sec}$$

The final design selection for the geosynthetic, after having checked all parameters involving permeability and transmissivity, burst strength against uplift and tear strength to avoid bad placement conditions, reached a specification for a non - wooven, needle punched continuous filament polypropylene geotextile (Polyfelt TS800). The installation guidelines adopted were conform to the manufacturer specifications and took into consideration all necessary details determined by the local conditions (especially temperature and UV radiation effect).

5 CONSTRUCTION QUALITY CONTROL AND PERFORMANCE

The ground water level was one of the most crucial parameters to be taken into consideration, in order to assure the lower possible construction cost for excavations, placement of the geosynthetic and backfilling. Periodical measurements of the water level within the placed piezometers and comparing them to the tidal variations of the sea - level, the period May - December was selected as the most appropriate for site excavation works, since the ground water was at its lower levels. Excavations were performed to the indicated levels with all intermediate slopes at 1:3 inclination. Prior to the placement of the geosynthetic, all soft pockets of clayey or silty material were carefully removed and replaced and a light compaction of the created interface was performed to 80% approximate density. The placement of the geosynthetic was performed with all necessary precautions of the specifications and the maximum delay time before backfilling (protection against UV radiation) was not longer than 24 hours.

The placement orientation of the geosynthetic was not of any importance, since the subdivided zones were almost rectangular. The geosynthetic length of the role was laid along the width of the covered area with a total recovery of the sides and overlapping (no sewing was necessary) was performed at approximately 0,40 m.

The material finally used for backfilling did not come from the specified borrow areas, as it was both cheaper and easier for the contractor to use quarry material from a near-by situated limestone quarry. As the material was brought to site, a continuous classification tests control was daily assured (classification tests, Proctor and CBR), to prove the adequacy and conformity to the initially posed specifications. Backfilling procedure consisted of layering into 20 - 30 cm thickness, moisturising to 7-8%, dynamic roller compaction till obtention of 95% density of the Modified Proctor test and regular quality control tests. For better results, the embankment's construction included the scratching of each compacted layer before the deposition of the next layer material and the recompaction of the two layers together.

The quality control of the obtained density on the various layers adopted the sand - cone method and the testing frequency was approximately 10 tests/500 m^2. At the following statistical graph, the number of occurencies for each stage of compaction in reference with the total number of occurencies is given.

Following the last layer compaction at the design elevation of +4,00, plate loading tests were performed at the center of each tank. The moduli of deformation calculated at the end of those tests allowed for an estimation of the expected settlement for each tank and its comparison to the theoretically calculated settlements at the preliminary design stage.

As soon as the tanks were ready installed on site, special water tests at specific rates of loading and unloading were performed with continuous monitoring of the developping settlements at 4 locations over the periphery of each tank, till obtention of the 90% of the maximal theoretically calculated settlements. The consolidation period was variable between 30 days and 78 days, depending mainly on the local influence of the soft clayey or silty pockets of material within the quaternary sandy deposits. Special attention was given to the fact that the water tests should include a total cycle of tide developping with continuous monitoring and control of possible lateral failures, due to an eventual outwash of the finer part of the quick sand, below the geosynthetic. The geotechnical behaviour of the reinforced embankment was rather satisfactory and the final settlements measured at the end of the final cycle of the water tests, compared to the theoretically designed ones are given at the following Table 2.

It is mostly interesting to point out that the monitored differential settlements over the periphery of each tank, are less important than expected through the theoretical calculation. This fact may be attributed to a certain beneficial effect provided by the geosynthetic that allowed an homogenisation of the behaviour of the beneath situated soil layers.

Quantitative evaluation and theoretical explanation of this beneficial effect has not yet been possible, although a special analysis on large deformations calculations using a well known finite element computer program had been performed. In fact, there is a geotechnical lack of knowledge for the behaviour of quick sand under a continuously

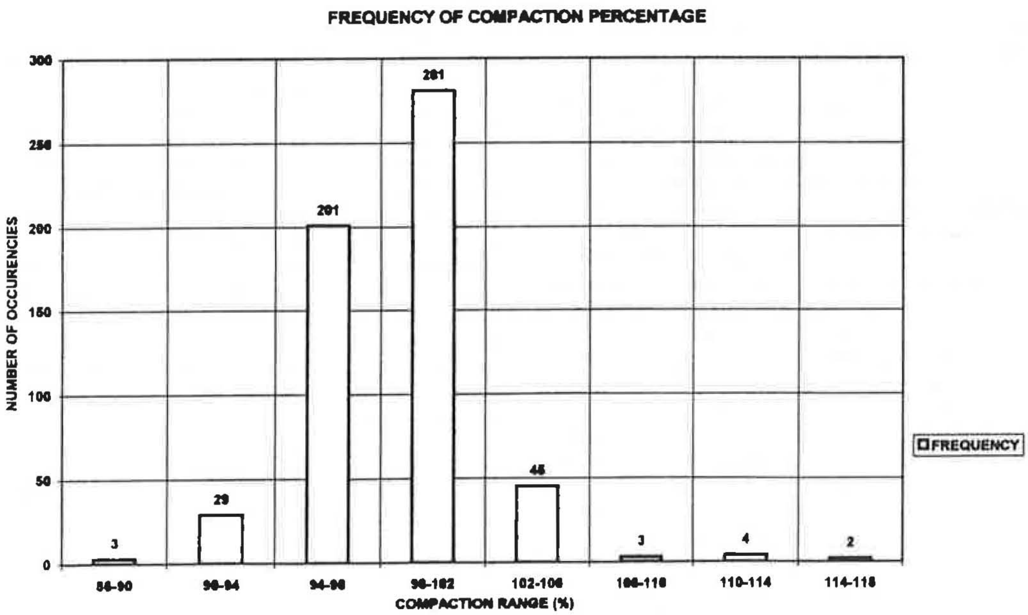


Table 2. Comparison of monitored to calculated settlements

Tank No	Total Time (Days)	Monitored Settlements (cm)	Calculated Settlements (cm)	
		Periphery	Center	Periphery
T-104	12	1,16	4,0	3,2
	42	1,57		
	72	1,90		
T-105	12	0,87	3,9	2,8
	42	1,66		
	72	2,00		
T-109	12	1,00	5,2	3,5
	42	1,44		
	72	1,70		
T-110	12	1,10	4,9	3,4
	42	1,52		
	72	1,70		
T-111	12	1,51	3,5	2,8
	42	1,77		
	72	2,10		

variable vertical pressure and the existing geological information on this soil formation is rather poor.

The settlements evolution in relation with time is given at the following figure 2.

It is mostly interesting to observe that independently to the size of the tank, more than 50% of the total observed settlement was realised upon completion of the first cycle of the water test, while 90% of it was realised within 30 days approximately of full load capacity, proving that the design principles were more or less unfavorable. The neglected during the design of the soil behaviour homogenisation, because of the presence of the geosynthetic, may be the main reason and it deserves further research in a theoretical modeling of the soil deformations in conjunction with the fabric.

Upon completion of all water tests, the tanks farm was operationally ready and fuel oil and other products started filling up the tanks. The monitoring points of observation for deformation have not yet been removed and regular survey measurements are performed at 6 months intervals approximately. According to those measurements, no practical increase of the deformation or any swelling has been observed and the geotechnical performance of the designed project is judged as rather satisfactory.

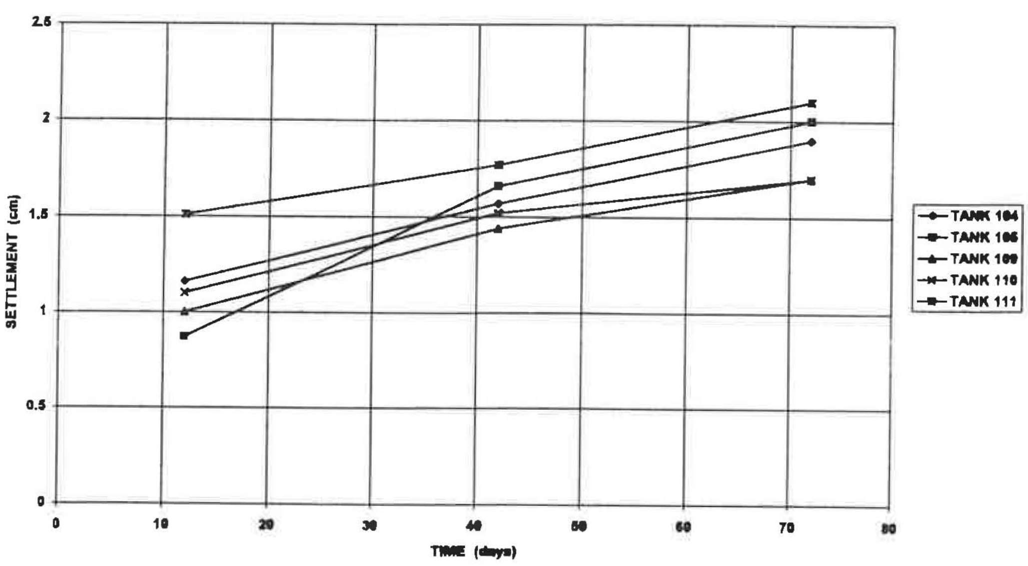
6 CONCLUSIONS

The most cost - effective solution for the construction of a new refinery and tanks facilities was designed and applied at an area superficially covered by quick sands with a maximum thickness of 4,00 m and periodically influenced by high tide effects.

The solution involved the use of an adequately compacted filling material layer and the placement of a separation - reinforcement non - wooven geosynthetic at the interface.

A strict quality control throughout the construction period, mainly involving the control of the in- situ obtained density as well as monitoring and continuous follow up of the realised

SETTLEMENTS EVOLUTION



deformations assured a nearly perfect performance of the various structures, since the designed stresses and deformations were rather realistic and the observed final values were slightly lower. The geosynthetic design procedure was rather simple and its use reduced the total cost of construction for the foundations of the major tanks by approximately 15%, without taking into consideration that any eventual deeper replacement of the superficial layer should request the use of expensive sheet pile walls. Another beneficial effect of the geosynthetic refers to the complete change of the "quick sand conditions" that remained below the treated level of backfilling, since a very different flow regime is assured and the soil skeleton preserves its original structure and resistance.

Following the end of the 1st stage of construction for the tanks facilities, a second stage of expansion of the refinery unit is faced, at a zone parallel to the coastal line and most probably the present cost - effective solution will also be adopted in the future.

REFERENCES

Edafomichaniki Ltd 1991, 1992. Refinery and Oil Tanks outside Fujairah Port, 1st and 2nd Stage of investigation.

Encyclopedia of Polymer Science and Engineering 1990. John Wiley and Sons, Vol. 17 : 796 - 798.

Reich, L., and Stivala, S.S. 1971. Elements of Polymer Degradation. Mc Graw Hill: 68 - 70.

Tschebotarioff, G.P. 1981. Foundations, Retaining and Earthstructures, Second Edition: 322-327.