Performance of a large diameter oil storage tank on improved clay deposit

Performance d'un bac de stockage de grand diamètre sur un sol d'argile amélioré

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ABSTRACT
Large diameter cylindrical welded steel tanks bearing on soft clay deposits can experience substantial settlement. Ground improvement by provision of granular piles is an effective method to strengthen the soft clay and control settlements within acceptable limits. The paper presents the installation technique together with results of load tests conducted on trial granular piles to assess the behavior of the improved ground. The tank performance, evaluated by hydro-testing the tank, compared well with the theoretical model used to compute the settlement of the improved ground.

RÉSUMÉ
Un bac de stockage en acier soudé de grand diamètre reposant sur un sol d'argile molle peut s'affaisser considérablement. L'amélioration du sol grâce à des pieux granulaires est une méthode efficace pour renforcer l'argile molle et limiter l'affaissement dans des proportions acceptables. Cet article présente la technique d'installation ainsi que les résultats des tests de charge conduits sur des pieux granulaires d'essai afin d'évaluer le comportement du sol amélioré. La perfomance du bac, évaluée par hydrotest, est conforme au modèle théorique utilisé pour calculer l'affaissement du sol amélioré.

Keywords : Large diameter steel tanks, soft clay, granular piles, settlement of improved ground, load tests, hydrotest

1 INTRODUCTION

A series of 65.5 m diameter and 15 m high oil storage tanks were proposed to be constructed on soft clay deposit in coastal Andhra Pradesh, India. The site is located close the sea shore. Due to low shear strength and high compressibility characteristics of the marine clay deposit, construction of the tank on these soils in the natural state could result in bearing capacity failure and large magnitude of settlement.
Therefore the soils beneath the tank were improved by provision of granular piles. The paper presents the case study of one of the tanks founded on treated marine clay deposit. It includes a presentation on the soil conditions together with the soil parameters selected for analysis along with evaluation of the tank settlement for the unimproved soils as well as the improved soils. The performance of the tanks on ground improved by provision of rammed granular piles is assessed through hydro-test and the settlement predictions compared with the actual measurements.

2 SUBSOIL CHARACTERISTICS & FOUNDATION ANALYSIS

Geotechnical investigations indicated the presence of very soft to soft marine clay to 5 m depth underlain by fine sand to about 13 - 14 m depth below which medium to stiff clay was encountered to 50 m depth. The water table was met at 1 - 1.5 m depth.

The tanks were planned to be placed on about 1 m thick compacted gravel pad imposing a pressure of 12.15 T/m^2 at the ground level. Based on the boreholes drilled, Fig. 1 presents the design profile used for the analysis.

2.1 *Total Settlement*

The total settlement of the tank based on the soil parameters given in Fig 1 works out to 1.431 m at the centre and 0.892 m at the edge. Applying correction due to the three dimensional consolidation (Skempton and Bjerrum 1957) the settlement at centre and edge works out to 1.216 m and 0.758 m respectively. Further, the immediate settlement values should be added to these consolidation settlement values. Thus, the total settlement will be very high and detrimental to the tank safety.

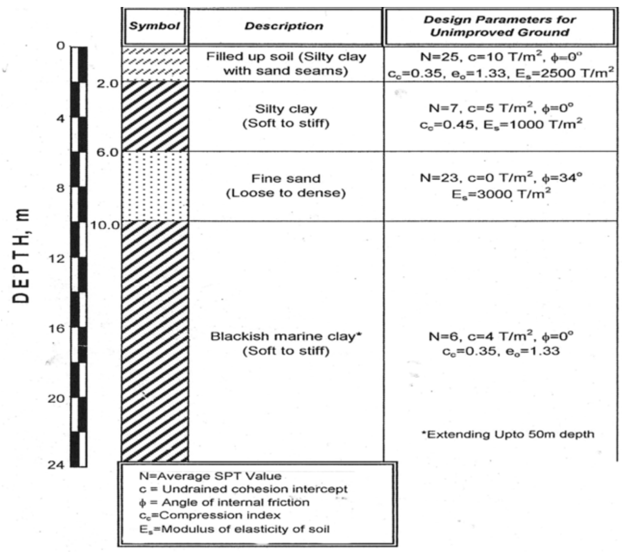

Fig. 1. Design Profile for Unimproved Ground

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2.2 *Ground Improvement*

Ground improvement is aimed at transforming weak and highly compressible subsoil into a stratum of desired strength and compressibility. The advent of more effective and fast operating machines and decades of accumulated experience have combined to trigger rapid advancement in ground improvement techniques. Better understanding of response of the improved ground has been the natural consequence (Ranjan 1989).

Reviewing the soil conditions, the use of stone columns / granular piles was selected as a technically viable and economically feasible scheme. The details are discussed in the following sections.

3 BEHAVIOR OF GRANULAR PILES

3.1 *Basic Concept*

Conceptually, a granular pile may be treated as a pile of low stiffness (Ranjan, 1989). Individually these piles are not capable of transferring the loads to deeper competent bearing stratum. However, when a large area is developed by providing several granular piles in a pre-determined pattern, the composite ground effectively supports the load.

Under actual loading conditions, the applied load is distributed between the gravel pile and the surrounding soils. The gravel pile acts as a reinforcing medium as well as a drainage medium.

3.2 *Behavior Under Load*

The load carried by the granular pile is resisted by interfacial shear between stone aggregates and soft soil and in end bearing (Ranjan, 1989, Rao, 1992). The lateral stress mobilized in the granular pile is resisted by lateral compressive strength of the soil. If the strength of the soil is less than the lateral stress in the column then the column will fail by bulging.

As per various researchers, the critical length of stone column varies from three to five times the pile diameter (Ranjan 1989). The function of granular pile is two fold i.e. to act as reinforcing media and the other as drainage media. The passive pressure developed due to loading of treated ground offers resistance to the bulging of the granular piles and thus contributes to its load carrying capacity. When the entire area is developed by providing granular piles, the soil in the inter-space also contributes to load carrying capacity.

A granular pile system derives its support mainly from the following four components –

a) resistance offered by the surrounding soil against lateral deformation (bulging) of granular pile under axial load,
b) bearing support provided by soil in between the granular piles.
c) increased resistance to lateral deformation due to surcharge on the surrounding soil,
d) increased capacity from components (a) to (c) above, resulting from dissipation of excess pore water pressure through granular piles which act as drainage paths.

3.3 *Peripheral Restrainment*

For best efficiency, peripheral restrainment (Ranjan and Rao 1986) of the granular piles is essential. This may be done by providing additional rows of stone columns outside the loaded area. An alternative method is to provide skirted granular piles.

4 DESIGN OF GRANULAR PILE SYSTEM

4.1 *Input Data*

The design profile for the unimproved ground is illustrated on Fig. 1. Modulus of pile material was taken as 5000 T/m^2. The nominal diameter (D) of the granular pile to be installed was 500 mm. The length of the granular pile was 10m. The centre-to-centre spacing of the granular piles was kept at 1.5m (=3D).

4.2 *Capacity of Granular Pile*

When a granular pile is installed by charging a borehole with crushed stone and compacted by ramming, it results in an increase in diameter. The installed pile diameter is 15 to 20% more than the borehole diameter (Ranjan 1989).

Recognizing the contribution of the load shared by the ambient clay (Ranjan & Rao 1986), the ultimate bearing capacity of a single granular is given by the following equation:

$q_{ult} = K\,(10\,c_u + q_s + 2.5\ \Upsilon_{sub}\,d)\,A_p$

where

q_{ult} = ultimate bearing capacity of single granular pile
K = non dimensional factor = 6 (Ranjan and Rao 1986)
c_u = cohesion intercept (undrained)
q_s = load shared by ambient soil
Υ_{sub} = submerged unit weight
d = installed pile diameter, taken as 15 percent more than the nominal diameter of the borehole
A_p = effective cross-sectional area of pile

According to Ranjan and Rao 1986, the load shared by the surrounding soil, q_s and q_p vary as

$$q_p = 3.33\,q_s$$

Since the pressure intensity at the ground level (q_s + q_p) as specified is 12.15 T/m^2, q_s = 2.80 T/m^2.

Substituting appropriate values in Eq.2, q_{ult} = 84.54 T, and adopting a factor of safety of 3, the safe bearing capacity of the single granular pile is 28.18 T Say 28 T.

The total weight of tank plus contents is about 53000 tonnes. Assuming a spacing of 3 times the pile diameter, the number of piles provided is 1900.

4.3 *Settlement Analysis*

The total settlement, S of improved ground reinforced with partially penetrating granular pile is given by the following equation (Rao and Ranjan 1985 and 1988):

$$S = \Delta L + \Delta H$$

where

$$\Delta L = \textit{Settlement of reinforced layer} = \sum_{i=1}^{n}\left(q_i\, m_{veqi}\, h_i\right)$$

ΔH = Settlement of unreinforced layer

m_{veqi} = equivalent coefficient of volume compressibility(m_v) of the i^{th} layer

q_i = incremental pressure at center of i^{th} layer of thickness h_i

Considering a total of 1900 granular piles installed, total pile area

= $1900 \times \pi / 4 \times (0.575)^2 = 493.13$ m^2

A circular area of about 69.5 m diameter was treated in order to strengthen the soils beneath the tank (Area = 3793 m^2).

Hence replacement factor,

$$\alpha = \left(\frac{493.13}{3793}\right) = 0.13$$

The coefficient of equivalent volume compressibility, m_{veq} , of the composite mass is given by the following equation:

$$m_{veq} = \left[\frac{1}{\alpha E_p + (1-\alpha)E_s}\right]$$

where E_p and E_s are soil modulus for pile material and soil respectively and α is the replacement factor or the relative pile area in terms of total pile area, A_p and total area of foundation, A ($\alpha = A_p/A$).

The computations of settlements are summarized below.

Sub-Soil	Layer Thickness, m	Total Settlement, mm	
		Centre	Edge
Treated	10	55	26
Untreated	15	164	75
Total	25	219	101

5 VERIFICATION THROUGH PROTOTYPE INSITU TESTS

To assess the safe capacity of installed granular piles and compare it with the analytically computed value, single granular pile and a group of three trial granular piles installed at site was load tested. The details of installations and load test results are presented in the following sections.

5.1 *Installation Technique*

The granular piles were installed in a pre-bored hole by compacting granular material by a rammer. The borehole was made using a bailer and a 3 m long surface casing. Bentonite slurry of thin consistency was used to stabilize the borehole. After drilling the borehole to the required depth, aggregate in about 1 m thick layer was placed in the bore. The aggregate used was well graded with a mix of 75 mm, 40 mm, and 20 mm size. The aggregate was compacted by a 1200 kg down-the-hole rammer, falling through a height of about 1.5 m.

The following set criterion, developed on site after installation of a few trial granular piles, was adopted -

(a) Apply a minimum of 30 blows on each layer
(b) Record the set for each of the 5 blows
(c) If the set for 5 blows is less than 20 mm in two consecutive observations the next charge is placed

After one layer of granular material is compacted, the second layer was charged and compacted using the set criterion discussed above. The process was repeated till the granular pile was completed (Gupta & Sundaram 1996).

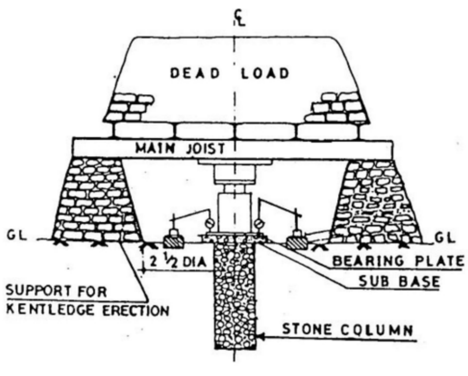

Fig. 2. Test Arrangement for Load Test on Single Granular Pile

5.2 *Load Testing*

After installing trial granular pile, these were load tested. Load tests were performed on single granular pile as well as on a group of granular piles. Fig. 2 shows a typical arrangement for single granular pile load test. The load tests were conducted after 15 - 20 days of installation of the pile. In order to simulate the field conditions in the intervening soil, a series of piles were installed as per the configuration (Fig. 3).

After the installation of the piles and the maturing period, a sub - base was prepared using aggregate and sand compacted thoroughly using a 10 T roller. Thereafter, a specially designed load frame was set up for applying the load. The load in the case of single pile was applied through a steel plate of 1 m x 1m size whereas in the case of the three pile group, the size of the plate was 2.5 m x 2.5 m. Fig. 4 is a photograph showing the test arrangement using kentledge along with a close up view of the load test arrangement for a group of granular piles.

The recording of the settlement under each magnitude of load was carried out using four dial gauges suitably mounted. The load-settlement curves are presented in Figures 5, 6 and 7.

In case of single pile load test, a glance at Fig. 5 indicates that initially as the pressure intensity is small, the corresponding settlements are small. However, as the magnitude of load intensity increases (more than 15 T/m^2) the ratio of increase in load intensity to corresponding increase in settlement increases.

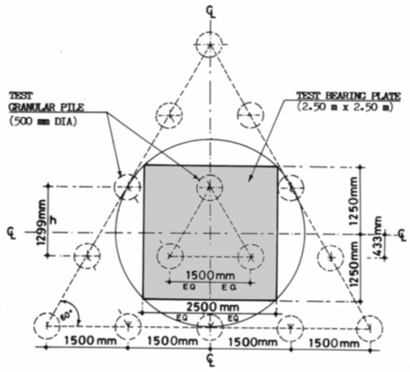

Fig. 3. Arrangement for Load Test on Group of Granular Piles

(a) (b)

Fig. 4. Load Test Arrangement on Group of Granular Piles
(a) Dead Load Kentledge (b) Close-up View of Test Plate and Dial Guage

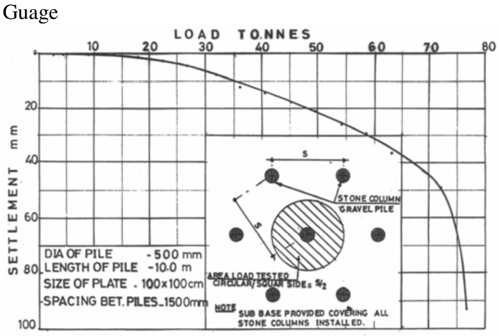

Fig. 5. Load Test on Single Granular Pile

The increase in rate of settlement with increasing load intensity further increases beyond 67.8 T/m^2. Reviewing the load intensity-settlement data, a failure intensity of 76.84 T/m^2 may be adopted which is in agreement with the theoretical estimates.

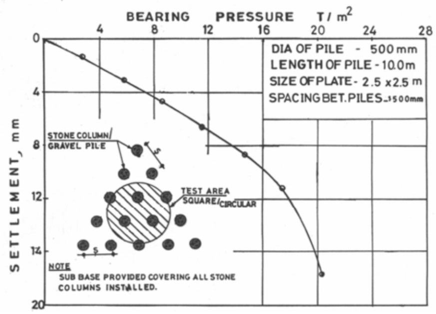

Fig. 6. Load Test on Group of Three Granular Piles

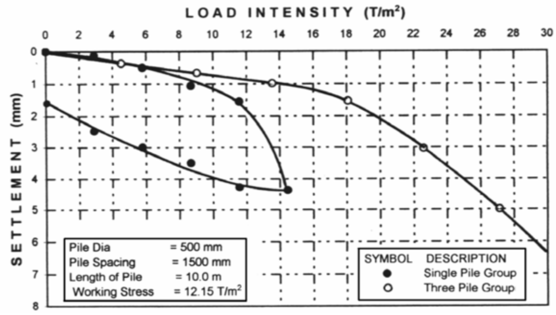

Fig. 7. Comparison of Load Test Results on Single Granular Pile and Group of Three Granular Piles

Results of three-pile group test are presented in Fig. 6. The settlement of the three pile group under the working stress of 12.15 T/m^2 is 1.7 mm. Comparing load intensity-settlement data of three-pile group test with single pile at the working stress of 12.15 T/m^2 (Fig. 8) it is observed that the ratio of settlement is not in the same ratio as the increase in the plate size but decreases. This is in conformity with the data reported in the literature (Bergado 1991).

After the testing on the trial granular piles was completed, the area was excavated to expose the granular piles installed. Fig. 9 is a photograph showing the exposed granular piles.

5.3 *Settlements During Hydrotesting*

After installing the granular piles, the tank pad was constructed and tank was commissioned. The tank was then subjected to hydro-test. After the hydro-test, the tank was filled with oil. During hydro-test and subsequent oil filling, the tank settlement tank was continuously monitored at 12 different locations (at an interval of 30°). The average settlement is plotted on Fig 10.

There are some differences in the predicted settlement values and actual measurements. The heterogeneity of soil and changes during installation are likely to influence the behavior and consequently the performance of the tank. However, the observations on settlement of tank indicate that the settlements are uniform. Further, the soil under the tank gets consolidated under repeated cycles of loading and unloading, resulting in improvement of soil characteristics. Thus with each cycle of loading and unloading the settlement of the tank decreases and stabilizes after five or six cycles (Fig. 10).

6 CONCLUSIONS

Ground improvement is being increasingly used to ensure satisfactory performance of structures. The poor subsoil treated with granular piles has been able to sustain heavy pressures due to the oil storage tank. The in-situ tests have verified the performance.

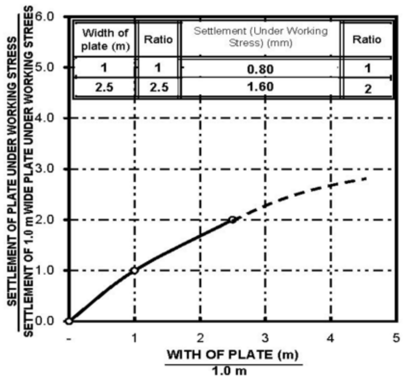

Fig. 8. Plate Size Versus Settlement

Fig. 9. View of Exposed Granular Piles

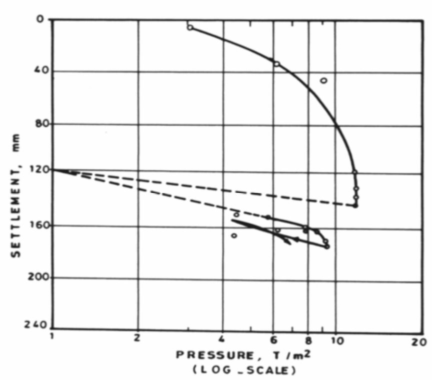

Fig. 10. Hydro-test on Tank

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