Sand Compaction Piles (SCP) for Improving Embankment Foundations
Case Study: Alhalfaya Bridge East Approach Road Embankment

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Abstract: This study investigates suitability of weak foundation soils for supporting the 12m high eastern approach earth embankment of Al Halfaya Bridge. The subsurface soils consist of soft silty clays and very loose to medium dense silty sand alluvial soils extending to 15m depth. Analysis of the results of a comprehensive geotechnical investigation carried out showed that the naturally occurring soils possess low bearing capacity and are expected to undergo excessively large settlements under the proposed embankment loads in case such soils have been treated. The soil investigation results also revealed that they have a very high potential for liquefaction in the event of an earthquake occurrence. A theoretical evaluation of the 'sand compaction piles, SCP' method for soil treatment is made in this study for improving the geotechnical characteristics to attain foundation soils adequately capable of supporting the proposed embankment safely. This has been resolved through the installation of sand piles with 1.1m diameter spaced at 2.5m interval distances in a square pattern. This design of SCP was found to improve the soil bearing capacity of originally weak foundation soils by 190%, to reduce consolidation settlement is reduced by 35%, to increase the safety factor against embankment slopes failure by 155% and significantly reduce the risk of soil liquefaction during any earthquake event.

Key words: sand compaction piles, soil improvement, high embankments, weak soils, soil bearing capacity, soil settlement, slope stability, soil liquefaction.

المستخلص: هذه الدراسة تبحث استقرارية تربة أساس ضعيفة لإنشاء رمادية ترابية بارتفاع 12 متر عدد مدخل كبرى الحلفايا الشرقي. تتكون التربة النحتية بالموقع من طبقة طينية ضعيفة وطبقة طينية رملية مهلكة إلى متوسطة الكثافة ممتدة إلى عمق حوالي 15 متر. أوضحت نتائج التحليل للبحث الجيولوجي المكشف ان قوة تحمل هذه التربة ضعيفة ويتوقع حدوث هبوط شديد نتيجة اعمال الرمادية المفترضة إذا لم يتم معالجتها. نتائج تحليل التربة أوضحت أيضا أن التربة لها خصائص التربة القابلة للانهيار في حالة وقوع هزات أرضية. تم تقييم طريقة الأعمدة
1. Introduction

The problems related to the design and construction of high earth embankments placed on weak and compressible soils i.e. low soil strength (low bearing capacity) and large settlements have lead to the development of ground improvement techniques for treating such embankment instability problems. Ground improving methods such as the surcharge loading, stone columns, wick drains, sand compaction piles and geosynthetic reinforcement, have all been used in various countries to accelerate the consolidation settlement rates and enhance embankment stability issues associated with construction on weak soils. In Sudan, most the approach earth embankments leading to the major bridge projects linking the two banks of the River Nile or its tributaries have to be placed on weak alluvial deposits that need treatment to support the overlying high embankment weights. In the research study presented in this paper an attempt is made to investigate the suitability of one technique for the treatment of a typical situation in which a high earth embankment must be constructed of relatively thick deposits of soft and loose soil deposits.

The study aims at evaluating the safety and stability of an embankment constructed on weak natural foundations soils and theoretically investigating the “sand compaction piles” method for the treatment of such a type of embankment. The site selected for the study is located at Alhalfaya area in Khartoum north town. A very high earth road approach embankment was proposed to be constructed on the eastern end of Alhalfaya Bridge built across the River Nile between Khartoum North and Omdurman to link it with Khartoum-Atbara highway.

2. Application of the Sand Compaction Piles Method (SCP) for Improving the Behavior of Weak Embankment Foundations

Sand compaction piles (SCP) of soil improvement techniques act as drains and under favorable conditions can significantly decrease the time for primary consolidation to occur. Furthermore, the
construction of sand piles reduces the build-up in pore pressure in granular layers, and hence decreases the soil potential to liquefy during an earthquake. Sand is often readily available near most construction sites, and it’s considerably less expensive than other processed materials such as crushed stone which may have to be imported from a considerable distance by a considerable cost. Either bottom feed stone column equipment or sand compaction pile equipment can be used to construct sand piles [1].

For this reasons the SCP method was selected for performing theoretical analysis required for improving the foundation soils under the high embankment. In order to apply the SCP method for the case study under consideration, it is important to set up an engineering design that fulfills the basic requirements of soil improvement. There are different procedures for the design of sand compaction piles and the design procedure adopted in this study is described elsewhere [2]. Theoretical analysis was undertaken to design and evaluate the degree of effectiveness of the SCP method for improving the weak and compressible soils behavior prevailing at the study site.

3. Geotechnical Design Aspects Related to Building High Earth Embankments on Weak Foundation Soils

The main geotechnical aspects that should normally be considered in the design and construction of high embankments to be supported on weak soil deposits such as soft cohesive soils and loose sands include the following:

- Shear Strength and Bearing Capacity of Foundation Soils.
- Soil Deformations (settlements).
- Embankment stability.
- Soil liquefaction.

Brief accounts are given below on each of the above aspects.

The stability of an embankment built on weak and highly compressible soils can be evaluated by determining the soil shear strength parameters (cohesion and internal friction angle) of the embankment and the supporting foundation soil strata using the Terzaghi’s bearing capacity. The coefficient of embankment stability should range from 1.30 to 1.50 depending on the importance of the road and the accuracy in determining the cohesion resistance $c$ of the layer [3].
The total settlement of highway embankments under its own weight and other applied traffic loads consists of a compression of the embankment material and the settlement of the underlying foundation soils. The settlement of soils typically consists of three; initial, primary and secondary. The initial settlement is the instantaneous compression that occurs upon load application on soil mass. The primary settlement is due to compression of the soil resulting from the gradual escape of water from the voids of loaded soil and accounts for the majority of the total settlement. Generally the total settlement of an embankment is taken as the summation of compression of embankment body and the elastic settlement of foundation soils. As for the compression of the embankment material, Sherard et al.[4] indicated that earth fill embankment dams will generally settle between 0.1 to 0.4% of their height and recommended that a fairly conservative approach is to assume a value of 0.5% for earth fill dams. The elastic settlement of foundation soils is calculated by the empirical equations based on field and laboratory tests using methods such as that developed by Schmertmann et al. [5].

Foundation soils and embankments provide adequate support for highway and other infrastructure if the imposed stresses do not exceed the shear strength of the embankment soils or underlying strata. Overstressing the embankment or foundation soil may result in embankment slope failures. The factor of safety which may be defined as the ratio of the allowable shear strength to mobilized shear strength is used in design to account for uncertainties with respect to about the reliability of the soil parameters that should be considered in slope stability analysis, such as the soil strength and stratification. Typically, the minimum factor of safety for slope stability of a newly constructed embankment ranges from 1.3 to 1.5 [6]. The values of factor of safety are defined considering the likely slope failure modes and shear strength of fill soils.

For the evaluation of liquefaction behavior of soils during an earthquake, a method based on cumulative extensive measured field and laboratory data has been developed and extensively applied in Japan [7]. The soil potential for liquefaction is usually assessed by the equivalent ground acceleration $a_{eq}$ and the SPT $N$ values measured in the foundation soils. A typical chart which may be used for soil liquefaction evaluation in which the equivalent acceleration is related to the equivalent SPT $N_{65}$ of a given soil layer is shown in Figure 1 where.

$$N_{65} = \frac{N_{65} - 0.019(\sigma_v - 65)}{0.041(\sigma_v - 65) + 1.0}$$  

(1)
4. Experimental Program Details

A geotechnical investigation consisting of drilling boreholes, performing cone penetration tests (CPT), and conducting laboratory tests on representative soil samples was carried out at Alhalfaya bridge east approach embankment site to determine their geotechnical properties the soils types studied.

The geometry of the proposed embankment is of 12.0m maximum height, 1:2 (vertical: horizontal) side slopes, 30.0m top width and 78.0m bottom width.

Figure 2 shows the locations of boreholes and CPT. The borehole and CPT depths reached the hard formation. Three boreholes were drilled to 14.5m to 16m depth using a continuous flighty auger Acker Co. and representative disturbed and undisturbed samples were taken from various soil depths. Standard penetration test (SPT) was performed in boreholes at 1.5m depth interval.

The level of ground water table was also recorded during drilling of boreholes. Static cone penetration test (CPT) measures two soil parameters known as the cone resistance $q_c$, and skin friction $f_s$, in the field to estimate the bearing capacity and deformations of the foundation soils. The CPT used in this study was a 20 tons capacity static machine equipped with an adhesion friction jacket cone.

### Figure 1 Assessment of ground acceleration $a_{eq}$ from equivalent $N_{65}$ values [ref]

Soil falling within zone I have a very high possibility of liquefaction.

Soil falling within zone II has a high possibility of liquefaction.

Soils falling within zone III have a low possibility of liquefaction.

Soils falling within zone IV have a very low possibility of liquefaction.
Typical borehole and CPT depth profiles of the soils tested in the study site are graphically illustrated in Figure 2. In addition to the field drilling and testing a program of laboratory testing was undertaken to classify the soil types encountered at the study site and determine the parameters required for the evaluation of their shear strength and consolidation characteristics. A concise summary of the laboratory test results obtained from the testing of embankment foundation soils is given in Table 1.

5. Analysis and Discussion of Study Results

In this section the soil profile (soil types and stratification), the safety of foundation soils and embankment against bearing capacity and settlement and slope stability and liquefaction probability are discussed for the naturally occurring soils before being subjected to any type of improvement.

5.1 Soil Profile

The borehole drilled revealed that the soil profile in the studied site comprised of a soft silty clay layer ranging from 1.0m to 3.0m in depth, underlain by a very loose to medium dense poorly graded fine to medium coarse silty sand or sandy silts extending considerably to a depth of about 15.0m. These alluvial deposits rest directly on the highly weathered Nubian sandstone formation. The water table encountered at 6.0m depth from the ground level.
5.2 Bearing Capacity of Foundation Soils

To estimate the bearing capacity of the soils on which embankment was constructed, the test results obtained from the SPT, CPT and laboratory tests were used in the analysis. Three different methods of predicting the soil bearing capacity were considered in this analysis including:

(a) Zein Method [8]
This method was developed for Sudanese fine grained clay and silt soils. The undrained shear strength, $S_u$, of these soils has been empirically correlated to the CPT cone resistance, $q_c$.

(b) Meyerhof Method [9]
Meyerhof proposed an empirical relation for the net allowable bearing capacity of foundation based on the cone penetration resistance ($q_c$).

(c) Fugro Engineering Method [10]
A CPT "simplified" method of calculation can be used for foundation design where "safe" bearing capacity can be estimated from cone resistance ($q_c$).

Figure 4 shows the variations of bearing capacity with depth for the naturally occurring sub-soils at the investigated site evaluated according to the above three different empirical prediction methods.

![Graph](image.png)

**Fig. 4** Comparison allowable bearing capacity predicted using empirical methods with embankment stresses (a) before improvement and (b) after improvement.

It may be noted that down to the depth of about 9m below ground level, the allowable bearing capacity values are much lower than the actual stresses which are expected to be imposed on the foundation soils due to the construction of the embankment. The variations of the actual stresses with depth were also plotted in solid line on Fig. 4.
5.3 Soil Settlement Due to Embankment Weight

The elastic settlement of the granular soil layers was evaluated using the semi-empirical strain influence factor method proposed by Schmertmann [3] assuming that at any depth the modulus of elasticity ($E_z$) is twice the cone resistance ($q_c$) measured in the CPT. According to the Schmertmann’s settlement calculation method, the elastic component of the settlement was found to be 248mm.

The consolidation settlement of cohesive soil layers was also computed using the test results obtained from consolidation tests of the soil samples and a maximum value of 690mm was estimated for the soil profile in BH1 which represents the worst case due to the occurrence of two relatively thick clay strata.

With regard to the embankment material compression, the 12m high Alhalfaya bridge embankment under question is expected to settle about 60mm under its own weight based on Sherard et al.[4] recommendation.

Therefore, the total settlement or the summation of the elastic and consolidation settlements in the foundation soil under the embankment plus the embankment body compression is anticipated to be 998mm which is a highly excessive value.

5.4 Slope Stability of the Embankment Section

The simple circular arc failure analysis was applied in this study and used to check the slope stability of the embankment section adopted for both homogeneous and heterogeneous soil conditions. Based on the assumption that a rigid, cylindrical block will fail by rotation about its center and that the angle of internal friction is zero in the silty clay layer i.e. the shear strength was considered to be due to cohesion only. The factor of safety was calculated by summing moments about the center of the circular surface and following the short cut procedure for the critical circle failure for the c-soils suggested by Fellenious [11]. The results of the slope stability analysis revealed that the lowest factor of safety equals to 1.43.

5.5 Liquefaction of Foundation Soils During Earthquakes

On the basis of comparison of the grain size distribution curves for soil samples taken from different borehole depths, it was found that the gradation curves for virtually all soil samples fall well in the middle of the boundaries of the envelopes specified for highly liquefiable soils based on the Japanese method [7].
For an equivalent SPT $N_{65}$ equals to 7.25 calculated from Equation (1) for measured SPT N-value of 9 blows/30cm the corresponding equivalent acceleration, $a_{eq}$ range between 0.76g and 0.95g for soils at the investigated site [12]. According to Figure 3, for $N_{65}=7.25$ and $a_{eq}=76-95$gall., the soils at the investigated site fall in zone I which indicates a very high potential of liquefaction and therefore are expected to liquefy during an earthquake if it occurs during the life time of the embankment.

5.6 Summary

Based on the above mentioned findings it was concluded that the proposed embankment will not be stable and/or safe if it has to be constructed on the soils under the naturally prevailing subsurface condition. Therefore, these soils have to be subjected to improvement or treatment such that they can support the proposed embankment safely and adequately against possible failure or risk due to inadequate bearing capacity, excessive settlement, slope instability or high probability of liquefaction.

A theoretical analysis was undertaken to evaluate the effectiveness of the SCP method for the improving of the embankment foundation soils as discussed in the following section.


6.1 Sand Compaction Piles Design

A theoretical design of the sand compaction piles (SCP) was undertaken with the intention to treat and improve the properties of the weak and compressible soils behavior prevailing at the study site as indicated in the previous section. The procedure described in the appendix to this paper was followed for the design of the required SCP configuration. The replacement area ratio $a_s$, was found to be equal to 0.15. Assuming sand piles of 1.1m diameter and 2.5m spacing, the volume of sand required to be installed per unit depth, $V$, will be equal to $0.938m^3/m$ depth for the square pattern.

6.2 Effect of SCP Installation on Soil Bearing Capacity

For the soil bearing capacity evaluation, the bearing capacities of uniform clay ground and uniform sand piles are first calculated individually, then the bearing capacity of SCP improved ground is calculated by the weighted average of the bearing capacities of all piles using the replacement area ratio, $a_s$. According to equations given in the appendix, the input data and assumptions were
made and subsequently used for the computations of the bearing capacity of the SCP improved soil strata for the particular case under consideration. The bearing capacity of natural soil layers located below 9m depth was greater than those corresponding to improved ground, thus the SCP improvement should not be extended beyond 9.0m depth at this site.

6.3 Effect of SCP on Foundation Soils Settlement
The final consolidation settlement of improved ground is calculated by multiplying the final consolidation settlement of the original ground before improvement \( S_0 = 690 \text{mm} \) for the case under consideration, with a reduction factor, \( \beta \) which a function of the soil fines content and the SPT N value. The value of the consolidation settlement reduction factor, \( \beta \) was calculated using the method described in the for the SCP treated foundation soil and was found to be 0.65. Accordingly the total consolidation settlement anticipated to occur in the modified foundation strata due embankment weight will be 448.5mm after applying a 35% reduction in the consolidation settlement after the improvement. The elastic settlement component of total settlement shall also decrease due to the increase in the CPT and SPT N-values expected after the improvement by SCP installation.

6.4 Effect of SCP on Slope Stability of Embankment Section
The stability analysis of SCP improved ground may be evaluated assuming a slip circle failure analysis in which shear strength of composite soil is incorporated [5]. Such an analysis was carried out for an assumed improved soil foundation and the factor of safety (FS) against sliding was found to be 3.64 according to the calculation method adopted. This value of safety factor is much greater than that computed for the untreated soil foundation (FS=1.43) indicating a 155% degree of improvement in the slope stability of the embankment.

6.5 Effect of SCP on Soil Liquefaction Potential
The value of the modified equivalent SPT \( N_{65} \) was calculated according to equation 1 for the case of SCP improved soil profile conditions. According to the modified N-value of \( ((N_{65}=29 \text{blows/30cm}) \) and the corresponding \( \alpha_{eq} \) value [12], the improved soil falls in zone IV of Figure 1 indicating that it has a very low potential for liquefaction during an earthquake event. This may be compared to the natural state soil conditions i.e. before treatment by SCP method identified as falling within zone I which has very high possibility of liquefaction.

7 Conclusions
This study investigates the suitability of the “sand compaction piles, SCP” method for improving the geotechnical characteristics of weak alluvial deposits under a high earth embankment without experiencing failure or undergoing successive soil deformations.

A suitable SCP design was undertaken to improve the originally weak foundation soils properties through estimating the replacement area ratio, \( r_{w} \), required. It was found that a system of SCP of 1.1m in diameter spaced at 2.5m for square pattern are capable of producing significant improvements in the stability and the anticipated future performance of the 12m high embankment of Al Halfaya bridge eastern road approach.

On the basis of the theoretical analyses carried out to compare the subsurface conditions and characteristics of the embankment foundation soils prevailing at the studied bridge site before and after SCP installation it was concluded that:

i. The bearing capacity of foundation soils after the improvement was found to be 250 kN/m\(^2\), and the degree of improvement according to Zein’s method [8] developed for local soils ranged between 30% and 370% for the different soil layers with an overall average of 190%.

ii. The consolidation settlement improved ground, decreased by 35% after SCP treatment. Moreover, the elastic settlement shall also be decreased due to the expected increase in the CPT and SPT N-values after improvement.

iii. The factor of safety for embankment side slopes stability of the improved ground increased by 155% of the value computed for natural subsurface conditions.

iv. The soil potential for liquefaction has been reduced considerably due to SCP installation such that the composite soils changed from highly liquefiable to that of a very low potential for liquefaction during an earthquake.

References


Appendix

(A) Allowable Bearing Capacity of Natural and SCP Improved Soils

The allowable bearing capacity of clay ground is given by the following expression:

\[ q_{ac} = \frac{1}{F_s} c \cdot N_c \]

The allowable bearing capacity of sandy ground is given by the following expression:

\[ q_{as} = \frac{1}{F_s} \cdot \frac{1}{2} \cdot E \cdot y_s \cdot N_y \]

Allowable bearing capacity of improved ground:

\[ q_a = (a_s \cdot q_{as} + (1 - a_s) \cdot q_{ac}) \]
(B) The SCP Design Procedure

1. Obtain target SPT N-value according to SPT N-value of original ground and condition of superstructure

   (procedure A)  

   (procedure B)  

   (procedure C)  

   (procedure D)

STEP 1

1. Obtain required as value from Figure 3.7 in the case where SPT N-value between compacted sand piles is the target value.

2. Obtain required as value from Figure 3.8 in the case where average SPT N-value is the target value:

\[ N = (1 - \alpha_s) N_0 + \alpha_s N_0 \]

STEP 2

1. Draw straight line of relationship between \( D_s \) and \( \varepsilon_s \) in Figure 3.10a according to \( \varepsilon_{\text{min}} \) and \( \varepsilon_{\text{max}} \)

2. Obtain increment of void ratio for original and target SPT N-values according to Figure 3.10a, then obtain required as value by

\[ \alpha_s = \frac{\Delta V}{1 + \varepsilon_0} \]

STEP 3

1. Obtain relative density, \( D_r \), and initial void ratio, \( \varepsilon_0 \), according to SPT N-value of original ground and effective overburden pressure:

\[ D_r = \frac{1}{21} \left( \frac{100N_0 - 70 + \alpha_s}{70 + \alpha_s} \right) \]

\[ \varepsilon_0 = \varepsilon_{\text{max}} - D_r (\varepsilon_{\text{max}} - \varepsilon_{\text{min}}) \]

STEP 4

1. Obtain reduction factor, \( \beta \), for increment of SPT N-value:

\[ \beta = 1.05 - 0.51 \cdot \log F_r \]

2. Calculate SPT N-value for non fines contents soil by taking into account \( \beta \)

STEP 5

1. Obtain \( \alpha_s \) by substituting \( N_0 \) into \( N_0 \) in Equation in STEP-2.

2. Obtain as for e_s and e_r by

\[ \alpha_s = \frac{\Delta \varepsilon}{1 + \varepsilon} \]

STEP 6

1. Obtain sand pile interval by following equations:

\[ d = \frac{A_s}{\alpha_s} \text{ (square pattern)} \]

\[ d = \frac{2}{\sqrt{3}} A_s \text{ (triangular pattern)} \]

STEP 7

1. Obtain relative density, \( D_r \), and initial void ratio, \( \varepsilon_0 \), by substituting \( N_0 \) into \( N_0 \) in Equation in STEP-3:

\[ D_r = \frac{1}{21} \left( \frac{100N_0 - 70 + \alpha_s}{70 + \alpha_s} \right) \]

\[ \varepsilon_0 = \varepsilon_{\text{max}} - D_r (\varepsilon_{\text{max}} - \varepsilon_{\text{min}}) \]

STEP 8

1. Obtain replacement area ratio, \( \alpha_s \), for target SPT N-value:

\[ \alpha_s = \frac{\Delta \varepsilon}{R_e(1 + \varepsilon_0)} - \frac{\varepsilon_0 - \varepsilon_1}{R_e(1 + \varepsilon_1)} \]

STEP 9

1. Obtain sand pile interval by following equations:

\[ d = \frac{A_s}{\alpha_s} \text{ (square pattern)} \]

\[ d = \frac{2}{\sqrt{3}} A_s \text{ (triangular pattern)} \]