The paper was published in the proceedings of the 17th African Regional Conference on Soil Mechanics and Geotechnical Engineering and was edited by Prof. Sw Jacobsz. The conference was held in Cape Town, South Africa, on October 07-09 2019.

Optimized foundation design in geotechnical engineering

M. Bouassida
University of Tunis El Manar, Tunis, Tunisia

S. Ellouze
University of Sfax, Tunis, Tunisia

ABSTRACT: The design of foundations constitutes a major step for each civil engineering structure. Indeed, the stability of those structures relies on cost effective and adequately designed foundation solution. To come up with an optimized design of a foundation, the geotechnical study passes by several steps: the geotechnical survey including in situ and laboratory tests, the synthesis of geotechnical parameters to be considered for the design and the suggestion of foundation solution avoiding over estimated cost and ensuring suitable method of execution. In this paper, the three currently practiced categories of foundation are briefly introduced. Then, two illustrative Tunisian case histories are analysed to explain, first, when the practiced foundation solution was inadequately chosen how a non-cost-effective solution can be avoided. Second, why an unsuitable foundation solution can lead to the stopping of the structure functioning and, then, how to proceed for the design of retrofit solution to be executed for restarting the functioning of the structure?

1 INTRODUCTION

Optimized foundation design (OFD) first passes by an adequately planned geotechnical survey which usually includes boreholes, in situ tests and laboratory tests. Such a program is decided on the basis of the structure dimensions (or area), load intensities and soil conditions. A well-planned geotechnical survey (number, location and depth of boreholes and in situ investigation) is followed by a suitable synthesis of geotechnical test results allowing the adoption of realistic geotechnical soil parameters. The latter constitutes the best starting point to think about an optimized design of foundations.

Three categories of foundations are currently practiced for civil engineering structures namely: shallow foundations, deep foundations and intermediate foundations related to reinforced or improved soils (Das 2017). For each category, the adequate type of foundation is decided on the basis of an optimized solution, e.g; cost effective and acceptable time of execution.

Shallow foundations include the following types: isolated footings; strip footings; crossed strip footing (either in one or in two directions) and rafted foundation.

Deep foundations also comprise a big variety of pile types (bored, driven, etc), the optimized solution rather relies on the installation method of pile to warrant a reliable and cost-effective solution.

Ground improvement techniques represent the third category of foundation which can be considered in between the shallow and deep foundations. Several techniques can be adopted depending on the accorded priority for the project, i.e. to increase the bearing capacity and/or to reduce the settlement, or to accelerate the consolidation of compressible soils: stone columns, rigid inclusions, etc (Indraratna et al. 2015).

In this paper, it is intended to highlight either the benefits or the disadvantages that can result from well planned or unsuitable geotechnical survey that can also lead to adequate or inadequate foundation solutions. Two Tunisian case histories are presented in detail to capture the learned lessons in regard to unsafe design in terms of non-cost effective or unsuitable foundation solution.

2 FOUNDATION OF POST OFFICE IN TUNIS CENTER: CASE STUDY N°1

The design of foundation of the post office in Tunis City, a ten-floor building (basement, ground floor, eight floors) is discussed. Due to the existence of deep soft clay layer the Tunisian Ministry of equipment decided the execution of 1 m diameter bored piles reaching 52 m depth. The foundation cost was approximately 40% of that of the whole project. Such an expensive solution was dictated by unacceptable long-term settlements compromising the stability of

the structure. However, the applied load by the building is equivalent to quasi-uniform vertical stress of about 90 kPa. As such, in view of studying a foundation solution at a reasonable cost, the stone column and the sand compacted piles reinforcement techniques have revealed potential cost-effective solution (Datye & Nagaraju 1981). Those techniques, however being adopted only for some oil tank projects in Tunis City, leads to a significant reduction in settlement to admissible limit. From the geotechnical survey which included boreholes and pressuremeter tests up to 40 m depth it was concluded that crossed soft soil layers are under-consolidated silt-clay with limit pressure less than 1200 kPa.

Consideration of rafted foundation (Fig. 1) leads to admissible bearing capacity equals to 100 kPa estimated from the pressuremeter method (French method 2011). It follows that the admissible bearing capacity of the oil tank complies with the applied load of 90kPa (Bouassida & Guetif 2000).

Using Terzaghi's method, the long-term settlement of rafted foundation resting on the unreinforced soil was of 45 cm at the axis and 16.3 cm at its border.

Given these non-admissible values of settlement, the building tilting is with high risk such it has been observed for several other buildings in the same area of Tunis City like for six floors building located at the street Zaghloul which became out of service after about ten years (Bouassida & Klai, 2016). Therefore, before studying a piled foundation solution which is not cost-effective, it is obvious to analyse the reinforcement option using either stone columns or sand compacted piles. In fact, the reinforcement using by stone columns will contribute in settlement reduction to comply with admissible values and also to accelerate the consolidation of reinforced soft layers (Bouassida, 2016).Accordingly, it is recommended to cover the surface of reinforced soil, before the execution of raft, by a blanket layer made up of a draining material preferably the same as that of columns, to evacuate the water which results from the consolidation of reinforced soil. Two models of reinforced soils are adopted to predict the ultimate vertical stress of reinforced soil. The ultimate vertical stress q_{ult} applied at the head of an isolated column is determined from Equation 1.

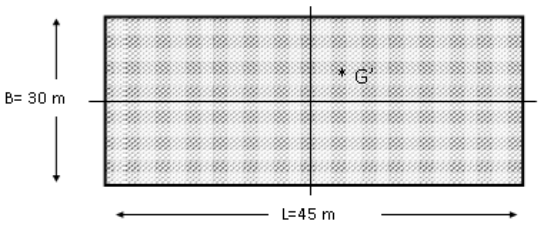

Figure 1. Geometry of rafted foundation (G' barycentre of applied loading)

$$q_{\text{ult}} = \sigma_h tg^2\left(\frac{\pi}{4} + \frac{\varphi_c}{2}\right) \quad (1)$$

For the purpose of conservative design, it is recommended to consider the horizontal stress $\sigma_h = p_l^*$;

p_l^*= limit net pressure recorded from the pressuremeter test.

σ_h= 230 kPa was considered.

φ_c=friction angle of the constitutive column material, its value is limited to 37° depending on the value of admissible stress of reinforced soil predicted by the French standard DTU 13.2, which are:

$$q_{adm} < 2\sigma_h \quad (2)$$

$$q_{\text{adm}} < 800kPa \quad (3)$$

$$q_{adm} < 2q_{ult} \quad (4)$$

Referring to the French method (2011) the admissible stress of the reinforced soil is equal to: $q_{adm} = q_r/2$ = 460 kPa. It is noted that this bearing capacity estimation does not take into account the contribution of initial soil. Using the group of columns model, the ultimate bearing capacity of reinforced soil is estimated as follows (Bouassida, 2016):

$$q_r = \eta\sigma_c + (1-\eta)\sigma_s \quad (5)$$

where η= substitution factor; σ_c= ultimate vertical stress of the columns and σ_s= ultimate vertical stress of the initial soil

$$\sigma_c = \sigma_h K_p + 2C\sqrt{K_p} \quad (6)$$

$$\sigma_s = \sigma_h K_{ps} + 2C_s\sqrt{K_{ps}} \quad (7)$$

$$K_p = tg^2\left(\frac{\pi}{4} + \frac{\varphi_c}{2}\right) \quad (8)$$

$$K_{ps} = tg^2\left(\frac{\pi}{4} + \frac{\varphi_s}{2}\right) \quad (9)$$

$C_{(s)}$ and $\varphi_{c(s)}$denote the cohesion and friction angle of columns material and initial soil, respectively. For conservative design it is assumed C = 0. The value of undrained shear strength of soil C_s is estimated when the net limit pressure (p_l^*) is less than 300 kPa, from the correlation proposed by Amar & Jézéquel (1972):

$$C_u = \frac{pl*}{5,5} \quad (10)$$

The admissible stress of reinforced soil is:

$$\sigma_{adm} \leq \eta\frac{\sigma_c}{2} + (1-\eta)\frac{\sigma_s}{3} \quad (11)$$

From Equation 11, consider an allowable vertical stress equals 120 kPa the corresponding minimum substitution factor is $\eta_{\min} = 0.13$. The variation of admissible stress of reinforced soil versus the substitution factor is shown in Figure 2.

The suitable value the substitution factor will obviously be depending on the admissible settlement of reinforced soil. However, the columns' length should

not exceed 15 m, based on considerations related to the calculation of bearing capacity of soil reinforced by floating columns (Bouassida et al. 2009). For settlement estimation, a 10 m column length is adopted.

The thickness of the compressible layers (with pressuremeter modulus E_M < 3MPa) under the foundation of the building is about 40 m. As a result, settlement of the foundation should be calculated as the sum of two terms: s_r settlement of reinforced soil and s_{ur} settlement of unreinforced soil (Bouassida & Hazzar 2012).

The settlement of rafted foundation is estimated by the pressuremeter method, and other methods assuming linear elastic theory like the variational approach which rather uses the group of columns modelling of reinforced soil.

Combining the verifications of bearing capacity and settlement a substitution factor equals 0.3 was adopted. The columns' installation of diameter of 1 m in square mesh with an axis to axis columns spacing of 1.6 m was suggested.

The total settlement of unreinforced soil after the pressuremeter method is equal to 16.3 cm. Whereas the settlement of reinforced soil s_r is respectively 3,5cm, 2,9 cm and 2,8 cm as obtained by the French recommendations, the homogenized Young modulus and the variational method (Bouassida et al, 2003).

The executed piled foundation necessitated the installation of 69 bored piles of length 54 m with total cost of 1 Million US $. Whereas the installation of 486 columns would cost about 600,000.00 US $. In case the reinforcement using stone columns was agreed about 37% reduction in the cost of the foundation and 11% in the total cost of the post office building were affordable (Bouassida & Guetif 2000).

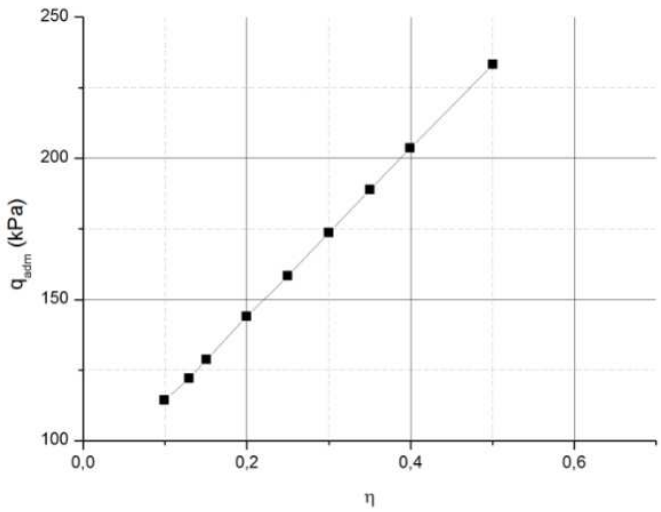

Figure 2. Variation of admissible stress of reinforced soil versus the substitution factor

3 OIL TANK FOUNDATION AT RADES: CASE STUDY N°2

It is a cylindrical steel tank of 33 m diameter built in the early nineties at the oil products storage area of the National Petroleum Company (SNDP) located in Rades in the Southern suburb of Tunis City.

The tank foundation was improved by sand piles of 0.6 m diameter and length of 6m overlaid by a mattress layer of 1.5 m thickness. Few years after the commencement of tank operations, its cylindrical shell suffered severe buckling deformation. The settlement due to the primary consolidation of compressible layers was estimated at 20 cm after fifteen years of reservoir operation. The study of this pathological case makes it possible to draw some lessons about the design of foundation on ground reinforced by floating sand compacted columns.

3.1 *Geometry and foundation of the tank*

The working vertical load of the tank is q = 100 kPa. The tank is constituted by a steel shell resting on compacted blanket layer of thickness 1.5m overlying a series of compressible sands to highly compressible layers. The main role of blanket layer is to make the settlement of tank as much as uniform and, therefore, to minimize the risk of differential settlements. The execution of tank is preceded by a reinforcement of the ground, on the first six meters by sand compacted columns well known in Tunisia as sand piles.

Based on collected data from three boreholes executed up to 40 m depth, and results of laboratory tests on several intact samples taken from 6m to 40m depth, the soil profile is subdivided into six layers (Figure 3).

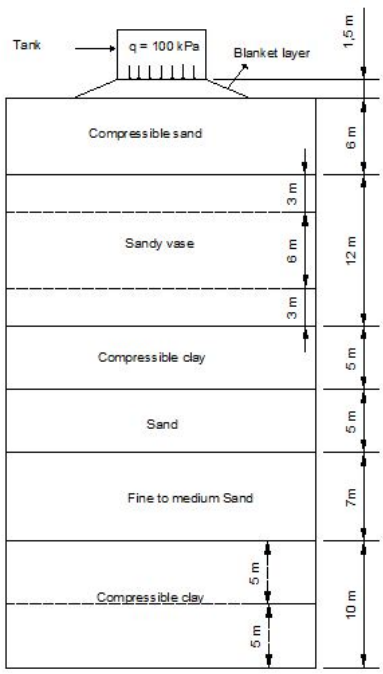

Figure 3. Adopted geotechnical profile (Kanoun & Bouassida, 2008)

Geotechnical parameters of soil layers were adopted from existing geotechnical survey previously carried out for existing similar tank projects near by the studied tank herein.

Geotechnical investigations have revealed that the layer between 2m and 30m deep has low strength characteristics. Therefore, reinforcement of those weak layers revealed necessary. The executed reinforcement comprised the installation of 481 sand columns, of 6m length and 0.6 m diameter, in non-regular pattern. The corresponding substitution factor was 11%. This solution essentially aimed to the acceleration of consolidation settlement in the upper layer of thickness 6 m.

The prediction of consolidation settlement was predicted using Equation 12 of unreinforced compressible layers, all assumed normally consolidated, by Terzaghi's method, up to 28 m depth:

$$s = H\frac{c_c}{1+e_0}log\left(1+\frac{\Delta\sigma'}{\sigma'_{v0}}\right) \quad (12)$$

where s = consolidation settlement; c_c = compression index; e_0 = initial void ratio; $\Delta\sigma'$= excess of vertical stress due to the tank's load and σ'_{v0} = effective overburden stress.

Within the first six meters depth (layer n°1) the long-term ground surface settlement is 95.1 cm which corresponds to approximately 50% of the total consolidation settlement. It should be emphasized that the long-term settlement of layers 2, 3 and 4 is approximately 75 cm, over a thickness of 12 m. Thus, the improvement by sand piles of 6m length will reduce the predicted settlement by about 44%, which is close to that of the upper layer. The total short-term settlement of unreinforced soil was estimated using a linear elastic calculation programmed in the software Columns 1.0 (Bouassida & Hazzar, 2012).

It is noted that the short-term settlement, which occurs over the first six meters depth, is of the order of 30% of total predicted settlement. Assuming that short-term settlement occurs at the end of tank construction, the long-term residual settlement is approximately 35 cm, which is not acceptable for the tank stability.

3.2 *Reinforced soil by sand columns*

Verification of sand columns reinforcement was conducted by using software "Columns 1.01" (Bouassida & Hazzar, 2012). Consider the area ratio equals to 11% the admissible bearing capacity was verified for the executed reinforcement. The settlement of reinforced soil was estimated assuming linear elastic behaviour; obtained results are given in Table 1.

The improvement by sand columns over the first 6 m depth, allowed limited settlement reduction, and its acceleration just in a few weeks. This settlement only represents about one-third of the total settlement and about 50% of that occurring in the highly compressible silt layer located between 6m and 18m depth. Thus, the proposed design for settlement reduction (sand piles of 6m length) is largely underestimated.

Table1. Estimation of linear elastic settlement (Columns 1.01 software)

Layer n°	Settlement of unreinforced soil (cm)	Settlement of reinforced soil (cm)
1	23.9	3.0
2	47.8	47.8
3	3.8	3.8
4	1.4	1.4
5	5.3	5.3
6	3.0	3.0
Total settlement (cm)	85.2	64.3

Indeed, over fifteen years of tank service, the visual observed settlement is of the order of 15 to 20 cm. This post-construction settlement corresponds to that of highly compressible layer between 6 m and 18 m depth for which the degree of consolidation after fifteen years is of about 46 %. While consolidation settlement of the reinforced layer was resorted after the proof water test of the tank.

3.3 *Retrofit technique: Micropile reinforcement*

The appropriate solution is to minimize or even to eliminate the estimated residual primary consolidation settlement in the highly compressible silt layers (over 12 m thickness) underlying the first reinforced layer by sand piles. Two retrofit solutions were suggested to achieve this objective:

- Micropile reinforcement (MP) of length reaching the top side of sand layer of thickness 5m (Fig. 4).
- Reinforcement by inclined rigid inclusions (IRI).

This retrofit solution requires the disassembly of entire tank's shell for the installation of the micropiles in a mesh of increased spacing from the centre to the border of tank. The micropile heads are connected by reinforced concrete beams are embedded ensuring uniform distribution of loads throughout the tank. The micropiles of 30 cm diameter are assumed to react only by shaft resistance. A total of 64 micropiles was estimated of total length 1,600 lineal meter (Fig. 4). This shaft capacity of micropiles was estimated based on pressuremeter data selected from the Rades-La Goulette bridge project (Kanoun & Bouassida 2008), due to the lack of specific geotechnical investigation for the oil tank.

Although reinforcement using micropiles constitutes non-cost-effective retrofit solution, it warrants the long-term tank stability without risk against residual settlement.

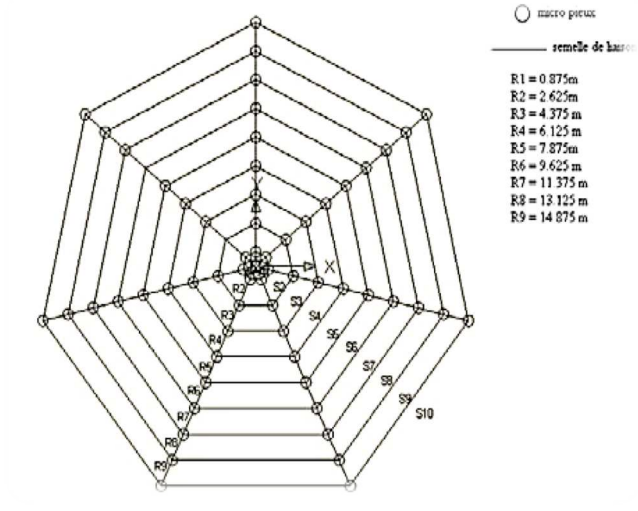

Figure 4. Layout of reinforcing micropiles embedded in reinforced concrete connecting beams (Simpro 2010)

3.4 *Reinforcement by inclined rigid inclusions*

To avoid the entire disassembly of the tank and to proceed for repairing only the affected areas by buckling, installation of Inclined Rigid Inclusions (IRI) embedded within the sand layer at 23 m depth can be designed (Fig. 5). The carrying shaft load by IRI corresponds to limited proportion of total weight transmitted by the tank structure. Therefore, it will be necessary to estimate the total allowable shaft load to be balanced by the IRI and to deduce the required number of inclusions to the proportion of carried tank load. Consider this latter estimated as 67 %, the remaining 33 % of tank load will be balanced by the consolidated first layer which degree of consolidation increased by around 50 % during the fifteen years of tank service. Since IRI are covered at the top by reinforced concrete slabs, an enhanced load concentration is afforded, therefore the number of IRI is less than that of the classical micropiles. From an economical point of view, the reinforcement using IRI is then less expensive than that of micropiles connected by reinforced concrete beams.

4 RECOMMENDATIONS

In order to repair the tank, a reinforcement using micropiles or rigid inclusions is required. Main objective of such reinforcement aims the neutralization of consolidation settlement of compressible layer located between 6 m and 18 m depth.

Due to the lack of pressuremeter data for the project, and to assess the predictions, it is strongly recommended to carry out pressuremeter tests up to 30 m depth. From recorded updated pressuremeter data the partial consolidation during fifteen years of tank service can be estimated.

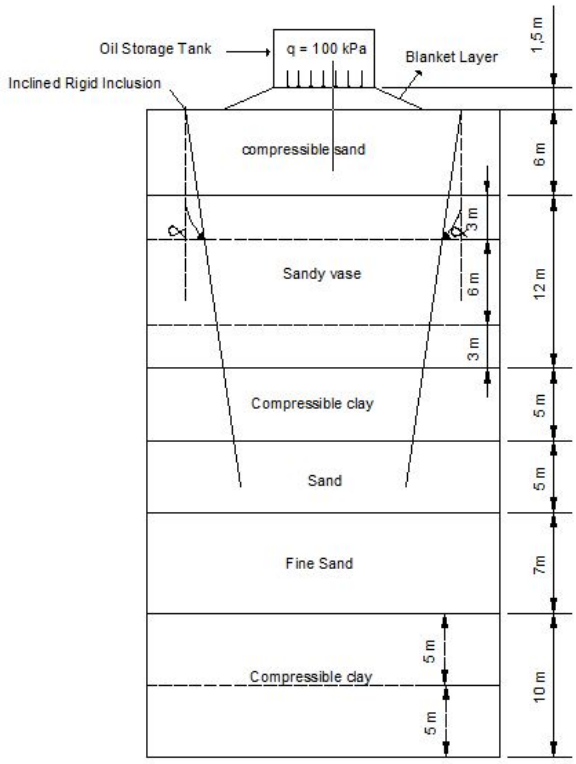

Figure 5. Reinforcement using inclined rigid inclusions

On the basis of these data, an adequate design of reinforcement solution will permit a realistic determination of the number of vertical inclusions of total length from 27 m to 30 m with 2 m embedment within the dense sand layer located approximately from 23m to 28 m depth.

Last, it should be reminded that a specific geotechnical survey reveals necessary to avoid consideration of inadequate geotechnical data even if adopted for the design of neighbouring projects.

5 CONCLUSIONS

This paper discussed the optimization of design of foundations from the analysis of two Tunisian case histories.

First, the execution of piled foundation for the post office building in the centre of Tunis City revealed non-cost effective. It has been proven that feasibility and benefits of reinforcement using stone columns would be preferable to that of pile foundation which remains non-cost-effective solution. The effectiveness of reinforcement by stone columns relies both on significant reduction and accelerated long term settlement so that stability of post office building in post construction phase will not be affected by differential settlement, as confirmed by experiences in several countries.

Second, non-successful reinforcement with sand piles was experienced for an oil tank project due to the lack of geotechnical survey and well-designed reinforcement characterized by too short sand piles of

length 6m. In fact, because the thickness of Tunis soft soil extends up to 25 m depth non-admissible consolidation settlement affected serviceability of the oil tank ceased after 15 years. Hence, reinforcement using micropiles of length 25 m depth was necessary to stop the remaining long-term settlement in initially unreinforced soft layers overlaid by the improved sand pile layer.

From the two investigated Tunisian case studies an optimized foundation design is, first, never warranted without a well-planned geotechnical survey followed by a meaningful data synthesis for selecting the suitable geotechnical soil parameters. Second, cost-effective foundation alternatives can be thought using well designed ground improvement techniques. The latter requires careful analysis especially related to the long-term behaviour of unreinforced soft soils which can be affected by the induced stress of the projected structure. As such, high costed retrofit solution can be avoided whenever serviceability of the structure is affected.

6 REFERENCES

Amar, S. & Jézéquel, J. 1972, Essais en place et en laboratoire sur sols cohérents, comparaison des résultats. Bull. Liaison L.C.P.C., n° 58: 97-108.

Bouassida, M. & Guetif, Z. 2000. Etude comparative "pieux-colonnes" Cas du siège des chèques postaux à Tunis. Actes du séminaire "Le renforcement des sols - état de l'art et perspectives en Tunisie". Hammamet 19-20 Octobre: 65-78.

Bouassida, M. & Hazzar, L. 2012. Novel tool for optimised design of reinforced soils by columns. Ground Improvement: Proc. ICE, London 165, Issue 1: 31-40.

Bouassida, M. Guetif, Z. de Buhan, P. & Dormieux, L. 2003. Estimation par une approche variationnelle du tassement d'un sol renforcé par colonnes. Revue Française de Géotechnique. n ° 102: 21-29.

Bouassida M, Porbaha, A. & Jellali, B. 2009. Limit Analysis of Rigid Foundations on Floating Columns. International Journal of Geomechanics, ASCE. Vol. 9, n°3 May-June, pp 89-101.

Bouassida, M. 2016. Design of Column-Reinforced Foundations. J. Ross Publishing (FL, USA), September. 224 pages. ISBN: 978-1-60427-072-3.

Bouassida, M. & Klai, M. 2016. Caractérisation et étude du comportement de la vase de Tunis. IVème Séminaire International "Innovation et Valorisation en Génie Civil et Matériaux de Construction. INVACO'2016; 15 & 16 décembre, Hammamet (Tunisia): 12 pages.

Das, B.M. 2017. Principles of Geotechnical Engineering, 7th Edition. Mac Graw Hill.

Datye, K.R. & Nagaraju, S.S. 1981 Design approach and field control for stone columns. Proc. 10th I.C.S.M.F.E., Stockholm, vol 3: 637-640.

French method. 2011. Documents Techniques Unifiés 13.2 (1983). Travaux de fondations profondes pour le bâtiment. Edit. CSTB (France). Fascicule 62, titre V, 1994.

Indraratna B. 2015. Ground Improvement Cases Histories, Embankments with Special Reference to Consolidation and Other Physical Methods. Vol. 1 & 2, Editors, Butterworth Heinemann publications. 2015 Elsevier, ISBN: 978-0-08-100192-9.

Kanoun, F. & Bouassida, M. 2008. Geotechnical aspects of Rades La Goulette project (Tunisia). ISSMGE Bulletin, 2 (3): 6-12.

Simpro. 2010. Cas pathologique du réservoir n°16 de la SNDP (AGIL) de la zone d'hydrocarbures à Radès. Rapport géotechnique.www.simpro-tn.com.