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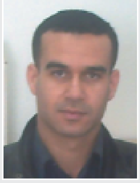
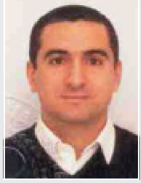

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On settlement of stone column foundation by Priebe's method

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Several contributions have been suggested to estimate the assumed linear elastic settlement of foundations on columnar reinforced soils. A number of authors have considered the so-called Priebe's method, which has been extensively used worldwide, and they have made suggestions especially for soft clays reinforced by stone columns. This paper first offers a critical analysis of the semi-empirical Priebe method by pointing out some inconsistencies related to assumptions made and the theoretical derivation of the settlement formula. A discussion of the limitation of Priebe's method for settlement estimation of foundations on soft soil reinforced by stone columns, with respect to other methods is included. Second, a comparison has been undertaken between predictions by Priebe's method and other design methods for three stone column projects in which some in situ data were recorded. On the basis of the studied case histories, it is concluded that recourse to other available and simple methods of design is more suitable than the use of Priebe's method.

NOTATION

A	grid area
A_c	stone column area
K_{ac}	coefficient of active earth pressure for column material
$\boldsymbol{P}_c$	pressure within the column along depth
$\boldsymbol{\gamma}_s$	total unit weight of initial soil
Δd	depth of subsoil layer from ground surface
ν_s	Poisson's ratio of native soil.
φ_c	friction angle of column material

1. INTRODUCTION

Evaluation of the stability of foundations resting on weak soil layers requires prediction of settlement. It is well known that stone columns represent one of the popular soil improvement techniques used to increase the bearing capacity and to reduce the settlement of foundations constructed on soft soil layers extending to a depth of 20 to 30 m. Other advantages in the use of stone columns are acceleration of primary consolidation settlement and the prevention of liquefaction (Adalier and El Gamal, 2004).

Various methods to estimate the settlement of reinforced soils by stone columns have been proposed in the past, and in most cases the linear elastic behaviour for reinforced soil constituents was assumed. These methods have included empirical methods (Greenwood, 1970), semi-empirical methods (Priebe, 1995), analytical methods (Balaam and Booker, 1981; Bouassida *et al.*, 2003; Normes Françaises, 2005) and finally numerical methods. In addition, other contributions have also been proposed based on the composite cell model as considered by Poorooshasb and Meyerhof (1997).

The first part of this paper presents an evaluation of the semi-empirical Priebe's method, namely the steps adopted for design and the assumptions for the estimation of settlement as detailed in Priebe (1995). Furthermore, referring to several publications which have addressed Priebe's method (Dhouib *et al.*, 2004; Dhouib and Blondeau, 2005; Priebe, 2004), it appears that some relationships have not been thoroughly written in comparison with the original paper (Priebe, 1995).

The second part focuses on a comparison between settlement predictions by Priebe's method and other methods programmed in Columns software. This comparison was related to three case histories of soft soils reinforced by stone columns.

2. PRIEBE'S METHOD

2.1. Priebe's method: basic assumptions

The basis of Priebe's method was developed some 30 years ago and then published (Priebe, 1976). Subsequently, several adaptations, extensions and supplements have been incorporated and these modifications justify a new and comprehensive description of the method which refers to the improvement effect of stone columns in a soil which is otherwise unaltered in comparison with the initial soil.

In a first step an improvement factor, denoted by n_0, is introduced by which stone columns improve the mechanical characteristics of the initial soil in comparison with its pre-treatment properties (without columns). According to this improvement factor, Young's modulus of the composite system is increased and, subsequently, the settlement is reduced. All further steps of design refer to this basic principle.

Reinforcement by stone columns only allows a more or less accurate evaluation for the well-defined case of an unlimited load area subjected to an unlimited column grid. In this case a unit cell with area A is considered, which consists of a single column with cross-section A_c and the tributary area of the surrounding soil, so that the improvement area ratio is given by A_c/A.

Furthermore the following idealised conditions are assumed.

(*a*) The column is based on a rigid layer (end-bearing).
(*b*) The column material is non-compressible.
(*c*) The unit weights of column material and initial soil are neglected.

2.2. Original version of Priebe's method

Based on Priebe's reasoning, the column can never fail as end-bearing capacity element and any settlement of the load area leads to a bulging of the column although the volume remains constant over its entire length. The improvement of a soil achieved in such conditions by the installation of stone columns is evaluated based on the assumption that the stone column material shears from the beginning whereas the surrounding soil reacts elastically. Furthermore, the soil surrounding the column is assumed to be already expanded during the stone column installation to such an extent that its initial resistance corresponds to the liquid state, namely the coefficient of earth pressure amounts to $K = 1$. The result of such an evaluation is expressed by means of the basic improvement factor n_0 as

$$n_0 = 1 + \frac{A_c}{A}\left[\frac{\frac{1}{2} + f\left(\nu_s, \frac{A_c}{A}\right)}{K_{ac} f\left(\nu_s, \frac{A_c}{A}\right)} - 1\right] \quad (1)$$

$$f\left(\nu_s, \frac{A_c}{A}\right) = \frac{(1-\nu_s)(1 - A_c/A)}{1 - 2\nu_s + A_c/A} \quad (2)$$

$$K_{ac} = \tan^2\left(45 - \frac{\varphi_c}{2}\right) \quad (3)$$

where φ_c denotes the friction angle of column material.

Adopting, for the native soil, a Poisson's ratio of $\nu_s = 1/3$, which is adequate for the state of final settlement in most cases, leads to the simple expression

$$n_0 = 1 + \frac{A_c}{A}\left[\frac{5 - \frac{A_c}{A}}{4K_{ac}\left(1 - \frac{A_c}{A}\right)} - 1\right] \quad (4)$$

2.3. Taking account of the compressibility of column material

Consideration should also be given to the column backfill material which is now assumed compressible. Therefore, any load causes settlement which is not connected with the bulging of the columns.

Accordingly, in the case where the improvement area ratio tends to $A_c/A = 1$, the actual improvement factor does not achieve an infinite value of n_0 as determined theoretically from Equation 1 or Equation 4 for non-compressible column material. Rather, it coincides at best with the ratio of the constrained, or oedometric, moduli of column material and initial soil.

It is relatively easy to determine the improvement area ratio of column cross-section and grid size $(A_c/A)_1$ for which the basic improvement factor corresponds to the ratio of the constrained (oedometric) moduli of column and initial soil D_c/D_s. As it is assumed that $\nu_s = 1/3$, the lower positive result of the following expression (with $n_0 = D_c/D_s$) delivers the required improvement area ratio $(A_c/A)_1$. Thus

$$\left(\frac{A_c}{A}\right)_1 = -\frac{4K_{ac}(n_0 - 2) + 5}{8K_{ac} - 2} \pm \frac{1}{2}\sqrt{\left[\frac{4K_{ac}(n_0 - 2) + 5}{4K_{ac} - 1}\right]^2 + \frac{16K_{ac}(n_0 - 1)}{4K_{ac} - 1}} \quad (5)$$

As an approximation, the compressibility of the column material can be considered by using a reduced improvement factor n_1, which can be computed on the basis of the modified area ratio $\overline{A_c}/A$ given by:

$$\frac{\overline{A_c}}{A} = \frac{1}{A_c/A + \Delta(A_c/A)} \quad (6)$$

where the additional amount $\Delta(A_c/A)$ of (A_c/A) is determined by

$$\Delta\left(\frac{A_c}{A}\right) = \frac{1}{(A_c/A)_1} - 1 \quad (7)$$

Subsequently, the modified improvement ratio n_1 can be computed based on the following formula

$$n_1 = 1 + \frac{\overline{A_c}}{A}\left[\frac{5 - \frac{\overline{A_c}}{A}}{4K_{ac}\left(1 - \frac{\overline{A_c}}{A}\right)} - 1\right] \quad (8)$$

2.4. Taking account of depth

Using Priebe's method it is assumed initially that unit weights of stone columns and the native soil are neglected. Then, the initial pressure difference between the column and the soil depends solely on the distribution of foundation load and it is constant over the entire length of the column.

Nevertheless, because of the additional effect of the weight of the soil and of the column, respectively, the pressure difference decreases asymptotically with depth. A depth factor f_d is then introduced as the ratio between the initial stress difference at the surface of the reinforced soil ($z = 0$) and stress difference at a given depth ($z > 0$). Based on this, the improvement factor is increased from n_1 to $n_2 = f_d\, n_1$.

It is suggested (Priebe, 1995) that the depth factor f_d is computed based on the following formula

$$f_d = \frac{1}{1 + \frac{K_{ac} - 1}{K_{ac}} \frac{\sum(\boldsymbol{\gamma}_s \Delta d)}{\boldsymbol{P}_c}} \quad (9)$$

where Δd in Equation 9 is referred to as improvement depth (Priebe, 1995), for which the vertical stress $\sum \boldsymbol{\gamma}_s \Delta d = \boldsymbol{\sigma}_v$ is computed as detailed in Dhouib *et al.* (2004). The main concern is which depth of improvement should be used to compute factor f_d?

In parallel, after Priebe (1995) the limitation of depth factor has been mentioned as follows

$$1 \leqslant f_d \leqslant \frac{D_c / D_s}{p_c / p_s} \quad (10)$$

where p_c is the pressure within the initial soil.

In relation to Equation 10, Dhouib and Blondeau (2005) stated that for rigid foundation then, since equal settlement is assumed, the required value is $f_d = 1$. In the same publication (Dhouib and Blondeau, 2005) for a case history which dealt with a circular rigid raft foundation in an axisymmetric condition, the depth factor considered for the design, as deduction after factor n_2, has an upper value $f_d = 2{\cdot}98$. This fact well illustrates how the design procedure is misunderstood, regardless of the background of the suggested formula, for settlement prediction by Priebe's method. This situation is very important since the best settlement is predicted by the use of depth factor.

3. CRITICISM OF PRIEBE'S METHOD

3.1. Concerning the design steps

As detailed by Dhouib *et al.* (2004), the theoretical framework of Priebe's method adopted the composite cell (CC) model. However, some inconsistencies are noted, based on assumptions considered for this model. Indeed, in a first step, Priebe considers a cylindrical cavity subjected to lateral expansion during which zero vertical deformation is assumed in order to give a solution expressed in plane stress condition.

In a second step, this solution is incorporated in the CC model for which there is a distribution of vertical stress generating non-null vertical deformation, and, consequently, the settlement is assumed constant. It is noted that Priebe's method is carried out by using the same CC model for two different problems (from the loading condition point of view): first the lateral expansion of a cylindrical cavity, and second a vertical loading. Then, the combination using the principle of superposition of the two state of stress solutions is not obvious (Salençon, 1988).

According to the calculation given in Dhouib and Blondeau (2005), the design formulae appear of complex use, especially, because of the semi-empirical character of this method as stated by Dhouib (2006). For estimating factor n_1, the assumption to take $n_0 = D_c/D_s$ is not clear, and no explanation is provided concerning how to derive Equation 8 starting from Equation 4. This may explain the improvement given by factor n_1 in comparison with n_0. Furthermore, Equation 9 shows that the depth factor depends on the unit weight of initial soil, whereas it was assumed earlier that unit weights of reinforced soil constituents are neglected. Then, the use of factor f_d, which further improves the reduction of settlement, needs to be clarified. In conclusion, the steps of the design procedure given by Priebe's method are not connected mathematically and they use different simplifications and approximations.

3.2. Concerning published papers

The expressions of n_0 in Equation 4, $(A_c/A)_1$ in Equation 5 and n_1 in Equation 8 are completely different in the references of Priebe (1995) and Priebe (2004). In the publications by Dhouib and his co-workers (Dhouib *et al.*, 2004; Dhouib and Blondeau, 2005) concerning the studied application dealing with a circular raft in axisymetric conditions, the settlement of the non-reinforced soil is predicted in linear elasticity, but on the other hand, the settlement of the reinforced soil is estimated in a particular manner. In addition, in these last two reference citations, the oedometric modulus is written

$$E_{oed} = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \boldsymbol{E} \quad (11)$$

which corresponds to the denoted modulus D_s (Dhouib *et al.*, 2004) but it is different from the actual oedometric (constrained) modulus expressed by Salençon (1988)

$$E_{oed} = \frac{(1-\nu)}{(1+\nu)(1-2\nu)} \boldsymbol{E} \quad (12)$$

Thus, Equation 11 needs to be corrected in the above references.

3.3. Comparison between Priebe's method and design methods incorporated in 'Columns 1.0' software

The French recommendations (Normes Françaises, 2005) and Balaam and Booker's methods consider the unit cell model as assumed by Priebe's model to predict the settlement of reinforced soil, whereas the same methods, which can be classified as rational methods, adopt the linear elastic characteristics, namely Young's modulus and Poisson's ratio as soil parameters.

However, for Priebe's method, only a Poisson's ratio of $\nu = 0{\cdot}3$ is considered.

In parallel the design method of Bouassida *et al.* (2003) incorporated in Columns 1·0 software has the advantage of considering a three-dimensional model for reinforced soil but still uses the linear elastic model, as in the French recommendations and Balaam and Booker's methods.

The semi-empirical Priebe's method uses, in addition, the internal friction angle of stone material as a parameter for settlement prediction. The friction angle as a failure characteristic, which is quite different from elastic characteristics, is not considered by the methods of design incorporated in the Columns 1·0 software, which all use the linear elastic characteristics.

4. STUDIED CASE HISTORIES

In the following sections, three case histories are examined to compare the methods of settlement prediction incorporated in Columns software with Priebe's method. The latter uses the settlement reduction factor defined as the ratio of foundation settlements for original ground conditions to final settlements after improvement. Then, for all studied case histories, a comparison between settlement predictions is presented in terms of the settlement reduction factor.

4.1. Oil storage tank at Zarzis

This is a 54 m diameter tank built at Zarzis terminal (Tunisia) on an area reclaimed as embankment (Solétanche-Bachy, 1990). The working load of the tank was approximated as a quasi-uniform stress of 120 kPa, which largely exceeded the allowed bearing capacity of the initial soil. In order to increase the bearing capacity and to reduce the settlement of the tank at allowed values, reinforcement by stone columns was adopted to ensure the stability of the tank was guaranteed. The reinforcement was executed along an average depth $H = 7$ m with a nominal diameter of columns equal to 1·2 m installed in a triangular pattern. The soil was improved on a circular area, the radius being the radius of the tank + 4 m, which implies an improved area ratio of approximately 32%. The characteristics of the native soil and constitutive column material are given in Figure 1.

The settlement of non-reinforced soil was estimated to be about 23 cm underneath the centre-line of the tank, whereas the settlement at the edge of tank was 6 cm.

In the calculations, the settlement of reinforced soil is predicted along a small depth H compared to the tank diameter: $H/2R = 7/54 = 0{\cdot}13$. Consequently, it is reasonable to neglect the horizontal displacement of the reinforced soil, especially at the centre-line of tank. Moreover, since the reinforced soil area is greater than the tank area, the assumption of null horizontal displacement becomes more realistic.

It is assumed that the tank is carrying uniform load $\boldsymbol{q}$. This type of loading yields, on the soil surface, an excess of vertical stress denoted by $\Delta\boldsymbol{\sigma}$, which varies with the distance from the tank centre-line. Indeed $\Delta\boldsymbol{\sigma}_{\text{centre}} = \boldsymbol{q}$, and $\Delta\boldsymbol{\sigma}_{\text{edge}} = 0{\cdot}48\boldsymbol{q}$. The settlement predictions at the centre-line of the tank and its edge that were obtained using the software Columns (Bouassida *et al.*, 2009) and Priebe's method are presented in Table 1.

In fact, using soil characteristics and under the surcharge of 120 kPa, the settlement prediction was deduced by different methods. By using the unreinforced soil's settlement values of 23 cm at the centre-line of the tank and 6 cm at its edge the value of the settlement reduction factor was deduced.

It is noticed that the reinforcement by stone columns led to significant settlement reduction. For the majority of methods, this reduction was about three to five times the settlement of reinforced soil. Furthermore, for this project, by comparing

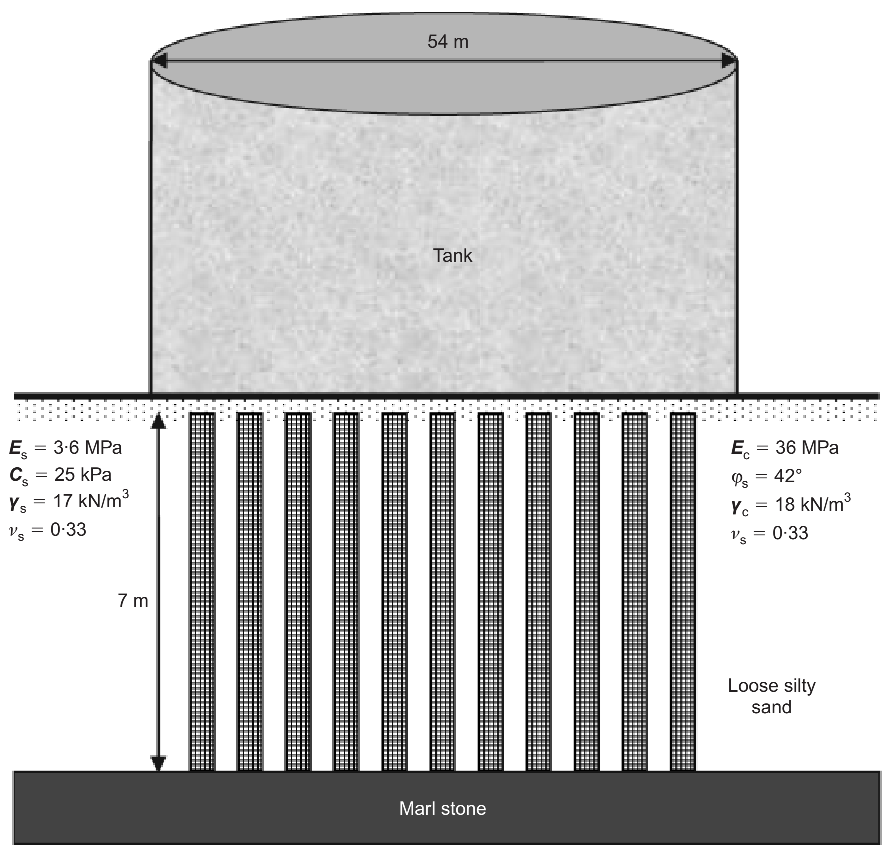

Figure 1. Characteristics of the native soil and constitutive column material

Methods	Settlement of reinforced soil at centre-line of tank: cm	Settlement of reinforced soil at edge of tank: cm	Settlement reduction factor at centre-line of tank	Settlement reduction factor at edge of tank
Recorded	–	3·0	–	2·00
Bouassida *et al.* (2003)	5·8	2·8	3·96	2·14
French standard (Normes Françaises, 2005)	5·5	2·6	4·18	2·30
Balaam and Booker (1981)	5·1	2·4	4·50	2·50
Priebe (1995)	6·1	2·1	3·77	2·85

Table 1. Comparison between predictions of tank's settlement using Columns software and Priebe's method

predictions an agreement has been shown between the variational approach and the French recommendations (Normes Françaises, 2005), despite the difference between the models adopted by these methods. In fact, the predicted settlements by the two methods appear the closest to recorded ones at the edge of the tank.

Using the improvement factor n_2, the predicted settlement by Priebe's method at the centre-line of the tank appears comparable with the predictions by other methods (variational (Bouassida *et al*., 2003), French recommendations (Normes Françaises, 2005) and Balaam and Booker (1981)) programmed in 'Columns' software. For the predicted settlement at the edge of the tank, Priebe's method underestimated the settlement according to the recorded settlement (averaged value).

4.2. Damiette project

The data considered are from a stone column project studied by Vibroflottation (2000). The soil profile was formed by a soft clay layer, 12·25 m thick, covered by a sandy layer 12·75 m thick. The soft clay was reinforced with vibro-displacement stone columns, 1·1 m diameter and with 2·7 m triangular grid spacing, implying an improvement area ratio of 15%. The parameters of the two soil layers and column material are summarised in Table 2.

To study the behaviour of reinforced soil the 'composite cell' model depicted in Figure 2 was considered (Guetif *et al*., 2007).

Parameters of soils	E': kPa	ν'	φ': degrees
Sand	25 000	0·33	35
Soft clay	4 000	0·10	21
Column material	32 000	0·33	38
Compacted sand	50 000	0·33	38

Table 2. Parameters of the reinforced soil system (Damiette project)

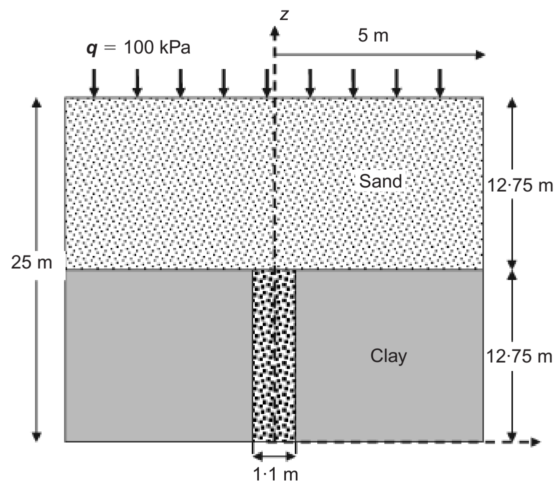

Figure 2. The 'composite cell' model

For this case history, the upper sand layer was assumed to behave like a surcharge applied to the reinforced soft clay. Settlement prediction presented in Guetif *et al.* (2007) and those provided by Columns software were compared with all predictions using Priebe's method. The overall results are summarised in Table 3.

The value of settlement factor reduction was deduced from the value of settlement of non-reinforced soil. In fact, for the different methods, namely those incorporated in Columns software and in Priebe's method, the linear elastic behaviour was used to calculate the unreinforced settlement which was about 63 cm. In contrast, by using Plaxis software, the predicted settlement of unreinforced soil settlement was equal to 46 cm.

For this axisymmetric model the predicted settlement derived using Plaxis software was underestimated in comparison with

Methods	Settlement of reinforced soil at centre-line of tank: cm	Settlement reduction factor
Plaxis software	23·0	2·00
Bouassida *et al.* (2003)	31·7	1·98
French standard (Normes Françaises, 2005)	31·7	1·98
Balaam and Booker (1981)	30·8	2·04
Priebe (1995) (with n_0)	36·8	1·71
Priebe (1995) (with n_1)	38·5	1·63
Priebe (1995) (with n_2)	26·2	2·40

Table 3. Comparison between predicted settlements with several methods (Damiette project)

the settlement estimations by other methods. These latter methods, which were programmed in Columns software gave almost the same prediction of settlement. On the other hand, the predicted settlements by Priebe's method largely varied from one condition to another, namely when using either the n_0 or n_1 or n_2 factors. To conclude, it is difficult to decide what is the appropriate Priebe's prediction to adopt in this example?

4.3. Tank foundation at Canvey Island (Priebe, 1995)

A 36 m diameter tank was subjected to an applied load of about 130 kPa. The tank was founded on a covering layer of 1 m thickness spread above reinforced soil by stone columns, 10 m long and set in a triangular pattern with spacing of 1·52 m and an average diameter of 0·75 m, as recorded at the surface after installation. The improved strata comprised some 0·4 m of top soil and consisted of a 9 m depth of silty and clayey soil with the presence of occasional pockets of peat. Columns were embedded in medium dense silty fine sand which was assumed to form a rigid stratum. Referred to depths, the characteristics of the initial soil are summarised in Table 4. Linear elastic characteristics $\boldsymbol{E}_c = 100\boldsymbol{E}_S$ and $\nu_c = 0{\cdot}25$ were adopted for the material of the stone columns. The improvement area ratio was about 22%. The settlement at the centre-line of the tank before reinforcement was estimated to be 40 cm.

Predictions of the settlement of the reinforced soil by several methods are presented in Table 5. According to the predictions obtained using the methods of settlement calculation incorporated in Columns software (variational method (Bouassida *et al.*, 2003), Balaam and Booker (1981), Chow (1996) and the French recommendations (Normes Françaises, 2005)) it can be seen that the Priebe method grossly overestimated the settlement of reinforced soil by stone columns.

Further, a remarkable difference between the values of factor n_2 taken from the chart and those calculated by $n_2 = f_d n_1$ is questionable.

Studying Priebe's example has highlighted the very conservative prediction of the settlement reduction factor by Priebe's method, especially when a very high Young's modulus of stone column material was considered, namely $\boldsymbol{E}_c = 100\boldsymbol{E}_s$ with 22% as improvement area ratio. As a matter of fact, by all methods using the linear elastic framework, a reduction of settlement of nine times was expected whereas the prediction by Priebe's method approaches three times the settlement reduction. This example demonstrates the semi-empirical nature of Priebe's method which should be handled carefully.

5. CONCLUSIONS

Several papers have described Priebe's method thoroughly by illustrated examples. In this paper, several aspects of the methods have been critically evaluated and it has been proved that the assumptions and theoretical derivation of the settlement formula are restrictive for stone column reinforcement and, sometimes, are not clear. Furthermore, some formulae are different from one publication to another, and so an adjustment is needed to obtain the correct expressions.

Priebe's method has been used worldwide for projects related to the reinforcement of soil by stone columns. However, they are aimed essentially at the prediction of settlement and are solely devoted to the stone columns technique. In the present study, Columns software has been used as a tool for comparison between settlement predictions given, on the one hand, by Priebe's method and, on the other hand, by other linear elastic models. From three analysed case histories, global agreement was found between predictions by the linear elastic models, whereas a difference, either insignificant or much different was noted when Priebe's method was used for predictions.

Layer *n*	Nature	Thickness: m	ν	***E***: kPa
1	Soft soil	0·4	0·33	1 350
2	Very soft soil	1·2	0·33	675
3	Very soft soil	6·6	0·33	6 749
4	Firm soil	0·8	0·33	13 499

Table 4. Characteristics of soil (Priebe's example)

Method	Settlement of reinforced soil: cm	Settlement reduction factor
French standard (Normes Françaises, 2005)	4·27	9·36
Bouassida *et al.* (2003)	4·52	8·84
Balaam and Booker (1981)	4·13	9·68
Chow (1996)	3·84	10·41
Priebe (1995) (with n_0)	17·02	2·35
Priebe (1995) (with n_1)	17·40	2·29
Priebe (1995) (with n_2 from curve)	12·62	3·16
Priebe (1995) (with n_2 from formula)	13·60	2·94

Table 5. Predicted settlement with several methods (Priebe's example)

REFERENCES

Adalier K and El Gamal A (2004) Mitigation of liquefaction and associated ground deformations by stone columns. *Engineering Geology* **72(3–4)**: 275–291

Balaam NP and Booker JR (1981) Analysis of rigid rafts supported by granular piles. *International Journal for Numerical and Analytical Methods in Geomechanics* **5(4)**: 379–403.

Bouassida M, Guetif Z, De Buhan P and Dormieux L (2003) Variational approach for settlement estimation of a foundation on soil reinforced by columns. *Revue Française de Géotechnique* **102(1)**: 21–29 (in French).

Bouassida M, Jellali B and Porbaha A (2009) Limit analysis of rigid foundations on floating columns. *International Journal of Geomechanics, ASCE* **9(3)**: 89–101.

Chow YK (1996) Settlement analysis of sand compaction pile. *Soils and Foundations* **36(1)**: 111–113.

Dhouib A (2006) About the paper: Recommendations for the design, calculation, installation and control of stone column foundations under buildings and structures sensitive to settlement. *Revue Française de Géotechnique* **117(4)**: 63–66 (in French).

Dhouib A and Blondeau F (2005) Méthodes de justification. In *Colonnes Ballastées*. Presses des Ponts et Chaussées, Paris, pp. 142–144 (in French).

Dhouib A, Wehr J, Soyez B and Priebe HJ (2004) Priebe's method: origin, developments and applications. *Proceedings of International Symposium, ASEP-GI, Paris, 9–10 September*. Presses des Ponts et Chaussées, Paris, pp. 131–143 (in French).

Greenwood DA (1970) Mechanical improvement of soils below ground surface. *Proceedings of the Conference on Ground Engineering*, Institution of Civil Engineers, London, paper II: pp. 11–22.

Guetif Z, Bouassida M and Debats JM (2007) Improved soft clay characteristics due to stone column installation. *Computers and Geotechnics* **34(2)**: 104–111.

Normes Françaises (2005) Recommendations for the design, calculation, installation and control of stone column foundations under buildings and structures sensitive to settlement. *Revue Française de Géotechnique* **111(2)**: 3–16.

Poorooshasb HB and Meyerhof GG (1997) Analysis of behaviour of stone columns and lime columns. *Computers and Geotechnics* **20(1)**: 47–70.

Priebe HJ (1976) Estimating settlements in a gravel column consolidated soil. *Die Bautechnik* **53(5)**: 160–162 (in German).

Priebe HJ (1995) The design of vibro replacement. *Grounding Engineering* December: 31–46.

Priebe HJ (2004) The design of stone columns. *Proceedings of International Symposium, ASEP-GI, Paris, 9–10 September*. Presses des Ponts et Chaussées, Paris, p. 2 (in French).

Salençon J (1988) Continuum mechanics – one-dimensional media. In *Elasticité–Milieux Curvilignes*. Editions Ellipses, Paris.

Solétanche-Bachy (1990) Foundations on reinforced soil by stone columns, oil storage tank at Zarzis. In *Project MARETAP, Tunisia*. Solétanche, Aix-en-Provence, France.

Vibroflottation (2000) *Geotechnical Report of the Damiette Project, Egypt*. Solétanche, Aix-en-Provence, France.