Exercise on calculation of stone columns — Priebe method and FEM

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ABSTRACT: The results of an international class-A prediction of an embankment on vibro stone columns are presented. The settlements of the unimproved and the improved case should be calculated with different methods. At first the analytical Priebe method was applied to estimate the final settlements and the Balaam-Booker method was used to calculate the time dependent settlements. Additionally, a finite element model was used with the conventional Mohr-Coulomb soil model. Settlements from calculations are compared, yielding very similar results close to the measured values.

1 INTRODUCTION

Improvement of soft soils with stone columns becomes an everyday practice in the construction industry. The vibro-replacement method is well established and suited for a wide range of soils. It has several positive effects on the treated soil, namely an increase of strength, stiffness and permeability.

The design methods for vibro-replacement are mainly semiempirical. They are based on analytical solutions of simplified geometry and idealized material properties of the column and surrounding soil. The overall trend for the application of numerical methods offers an alternative approach for the design. However, the comparability of numerical simulations with traditional methods should be guaranteed for standard situations. Only such a basis allows for extrapolations towards unknown conditions.

The following paper presents class-A predictions submitted in the framework of an international exercise on calculation of stone columns, organized in Paris, 2004. The settlements below an embankment on both, unimproved and improved soil were calculated with different methods. Moreover, time effects due to consolidation had to be taken into account.

2 PREDICTION EXCERCISE

2.1 *Input data*

For the Symposium ASEP-GI in Paris, 2004, a prediction exercise on the behaviour of an embankment on a soft soil improved by stone columns was organized. The geometry of the embankment and of the improved zone is shown in Fig. 1.

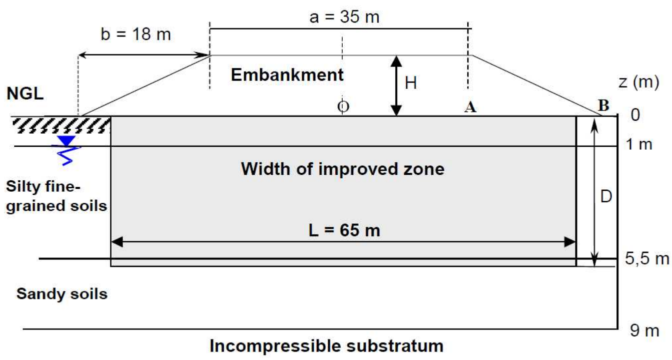

Figure 1: Embankment and subsoil geometry (ASEP-GI, 2004)

The deformation properties of the subsoil, consisting mainly of silty fine-grained soils, were described by pressuremeter moduli E_M. These were accomplished by a few index properties, compressibility and swelling indices (C_c and C_s) of the fine-grained soil, undrained shear strength c_u and consolidation coefficient c_v, see Fig. 2.

The prescribed geometrical setup of the stone columns and their parameters are summerized in Fig. 3.

The construction procedure included four steps: after the initial state (0), the embankment reached the height of 6 m during 40 days (1), after a rest period of 120 days (2), next 3 m of the embankment were constructed during 40 days (3). The final calculation

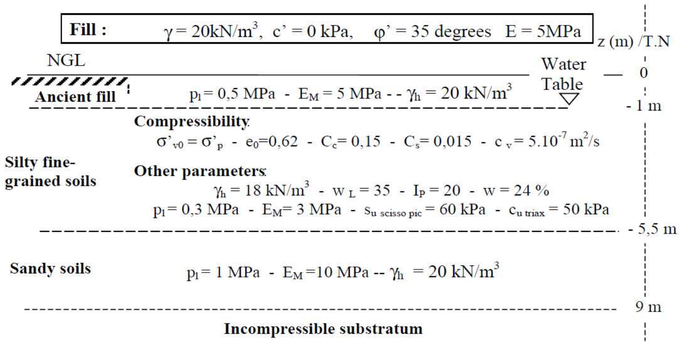

Figure 2: Subsoil properties (ASEP-GI, 2004).

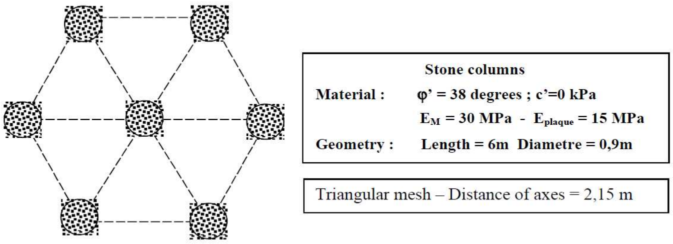

Figure 3: Stone columns parameters (ASEP-GI, 2004).

results were asked for 10 years after the construction beginning (4). Prediction outputs consisted of values of the settlements at the points O, A and B (see Fig. 1) for both, unimproved and improved ground.

2.2 *Priebe method*

A widely used design method for the vibro-replacement was proposed by Priebe, 1976. The basic version combines several simplifying assumptions: stone columns are in the active limit stress state, keeping their volume constant and being supported by the earth pressure at rest of the surrounding soil. Thus, the vertical deformation of the columns corresponds to their radial expansion and the load is distributed between soil and columns according to their area ratio. More details and design charts can be found e.g. in Dhouib et al (2004).

Priebe method was applied for the calculation of final settlements (software GRETA). Pressuremeter moduli were transformed into constrained (oedometric) moduli using the following scheme for the ratios $E_M/E_{oed} = \alpha$: 0.5 for normally consolidated soils, 0.33 for sandy soils, 0.5 for normally consolidated fill and 0.25 for gravel.

The time-dependent behaviour due to consolidation effects is not covered by the Priebe method. One can use solutions for the radial flow to wells published by Barron (1948). His approach originates from the application of the Terzaghi theory for this particular problem case. Several simplifying conditions are assumed: (i) all loads are initially taken by the excess pore water pressure, (ii) all deformations take place only in vertical direction and (iii) water inflow to the drain is radial, (iv) soil and column have the same stiffness. As extension of this theory, effects of a remoulded zone at the column margin and effects of the column permeability can be treated.

For the prediction exercise, time effects were taken into account by the theory of Balaam and Booker (1981). It originates from the Biot's consolidation theory and, contrary to the Barron's solution, different stiffnesses for soil and column are considered. Nevertheless, the mechanical behaviour is described only by elasticity.

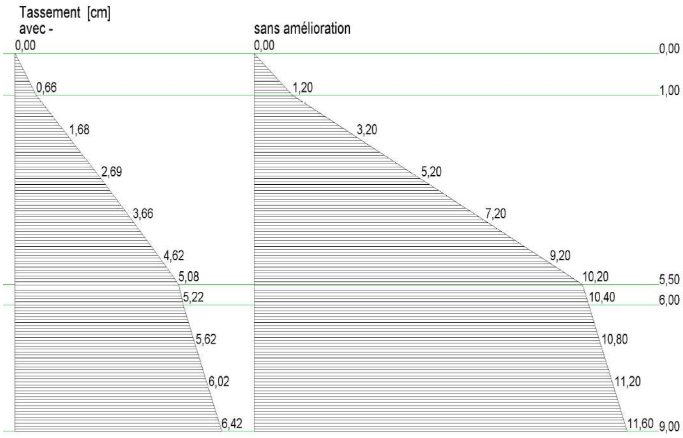

Figure 4: Settlements at the end of the 2nd stage calculated by the Priebe method.

The distribution of the calculated settlements at the end of the second construction stage (embankment height of 6 m, rest period) is shown in Fig. 4. The values reached 7 cm with and 12 cm without stone columns. For the final stage, 10cm with and 18cm without stone columns were calculated, see Fig. 5.

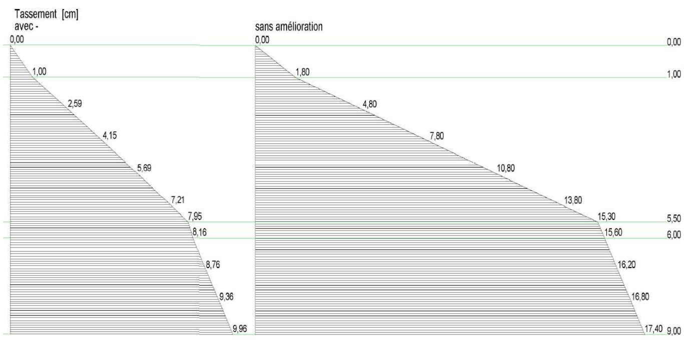

Figure 5: Final settlements calculated by the Priebe method.

The results of this standard calculation procedure show that the consolidation is already finished after 16 days. Therefore, the values of settlements of the 6 m high embankment after the first stage (40 days) and after the second stage (160 days) are equal. The same observation applies to stages 3 and 4 in case of the 9 m high embankment. This conclusion coincides well with the experience that the consolidation time with stone columns is usually less than 1-2 months.

2.3 *FE calculation*

Numerical simulations of the calculation exercise were performed with the finite element code Plaxis, Version 7. Plane-strain 6-node elements were used in the stepwise coupled analysis with closed consolidation boundaries at the mesh edges. In agreement with the prescribed material parameters, linear-elastic perfectly-plastic constitutive model (Mohr-Coulomb) was considered for both, soil and column material (Bazgan, 2004). The finite element mesh is shown in Fig. 6.

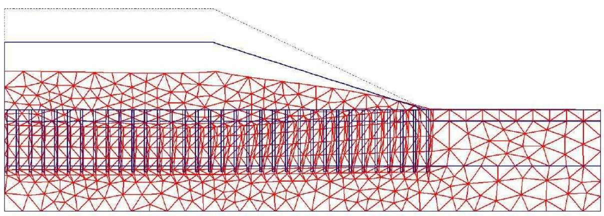

Figure 6: Finite element mesh.

Due to plane strain conditions, isolated stone columns were replaced by continuous stone walls. Their width of 0.2 m was obtained from the condition of keeping the volume of the improved soil unchanged.

A typical distribution of vertical displacements in case of the unimproved subsoil is shown in Fig. 7. It may be noticed that the largest deformations are concentrated below the embankment edge and not, as could be expected, in the middle of the fill. This can be related to the stress state in the subsoil which is close to the limit one and extensive plastic zones develop.

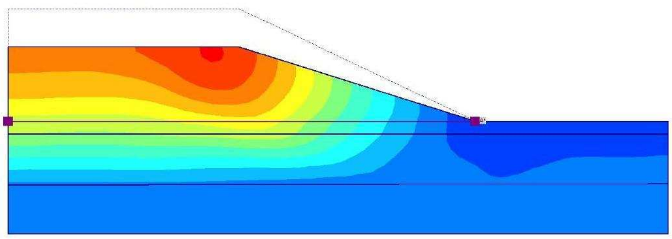

Figure 7: Distribution of vertical displacements at the 2nd stage (unimproved subsoil).

Calculated displacements for the unimproved ground are summerized in Tab. 1.

Table 1: FE results — unimproved ground.

Stage	t (days)	H (m)	O (cm)	A (cm)	B (cm)
1	40	6	9.8	11.2	-1.2
2	160	6	10.8	12.4	-1.2
3	200	9	15.6	16.4	-1.0
4	3650	9	16.5	17.5	-1.0

The behaviour of the embankment on unimproved soil (Fig. 7) can be compared with the case including stone columns. Fig. 8 shows vertical deformations for the embankment height of 9 m in the final stage after 10 years consolidation. The distribution of the deformations is more homogeneous in the latter case. Moreover, the maximum settlements are almost 40% smaller, reaching 11 cm.

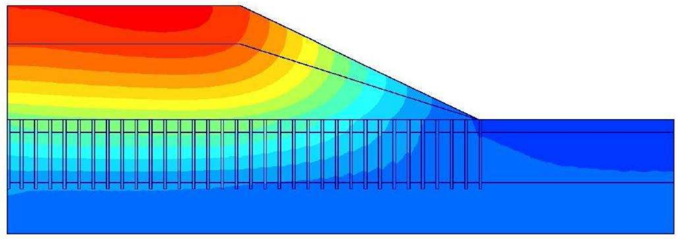

Figure 8: Distribution of vertical displacements at the final stage (improved subsoil).

An overview of the calculated settlements in case of improved ground is given in Tab. 2.

Table 2: FE results — improved ground.

Stage	t (days)	H (m)	O (cm)	A (cm)	B (cm)
1	40	6	6.8	7.6	-0.2
2	160	6	6.8	7.6	-0.2
3	200	9	10.2	11.0	-0.2
4	3650	9	10.3	11.0	-0.2

2.4 *Discussion of the results*

In case of improved soil, a comparison of the calculation results by the Priebe and the FE method in Tab. 3 reveals strikingly similar values (the settlements in point B at the end of the embankment cannot be calculated by the Priebe method).

Table 3: Comparison of Priebe and FE method for improved soil (Priebe/FEM).

Stage	O (cm)	A (cm)	B (cm)
1	6.4 / 6.8	6.4 / 7.6	– / -0.2
2	6.4 / 6.8	6.4 / 7.6	– / -0.2
3	10.0 / 10.2	10.0 / 11.0	– / -0.2
4	10.0 / 10.3	10.0 / 11.0	– / -0.2

The embankment of the calculation exercise was constructed after the submission of the predictions. The measured settlements reached settlements close to the ones of Tabs. 1 and 3 (see the prediction of the authors, No. 8, in Fig. 8 in Mestat et al., 2004). It should be noted that many of the international experts had difficulties already to estimate the settlements in the unimproved case without stone columns. This was a quite surprising result of this class-A prediction.

It seems that the particular task chosen for the calculation exercise fulfils two important assumptions of the Priebe method which are common with the FE simulations. First, the material behaviour is elasto-plastic. The crucial is the choice of an appropriate soil stiffness and column strength. Second, time effects fade away fast, thus having almost no influence on the calculated outputs.

One can consider some additional benefits from the application of a more powerful FE method. E.g. not only an overall distribution of the vertical displacements but also of the horizontal ones can be obtained, see Fig. 9.

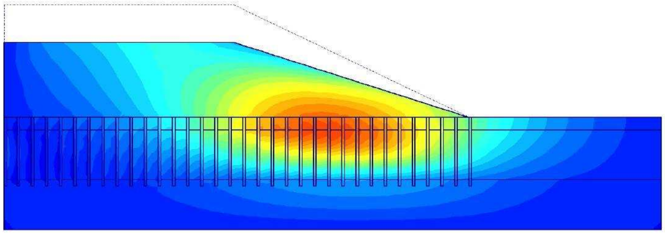

Figure 9: FE calculation, final stage: Horizontal displacements.

However, advantages of FE simulations are counterbalanced by the complexity of the method. Leaving aside many difficulties related to advanced constitutive models, the role of purely numerical parameters is often unclear to many users. They mostly rely on defaults values of these parameters offered by the program. Changing them can cause unexpected results. For instance, if decreasing the standard tolerated error from 3% to 1%, equilibrium for the first stage in case of unimproved subsoil could not be reached (Plaxis version 8 was used for this investigation). The reason was an embankment failure along a shear surface, see Fig. 10, which however did not occur in reality.

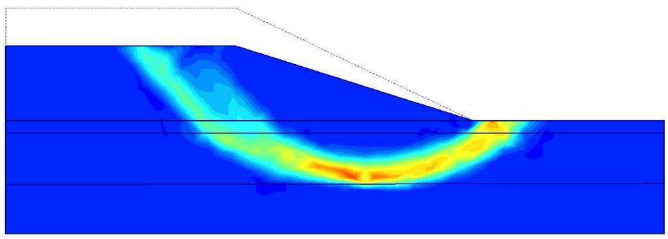

Figure 10: Incremental shear strain in the first calculation step (unimproved subsoil) — standard tolerated error 1%.

3 CONCLUSIONS

A comparison of the Priebe method and a FE analysis for the calculation of settlements of an embankment on the ground improved with stone columns yielded a close agreement between both methods. If no additional information (e.g. horizontal deformations) is needed, the Priebe method combined with the Balaam/Booker procedure is more favourable because

- FE method is much more time consuming and
- the risk of input errors in FE calculations is much larger.

In practice, other questions may be even more crucial for the calculated results. The highest uncertainty is usually related with the material parameters:

- The most important parameter for the consolidation time is the c_v-value, see Fig. 2. If "undisturbed" soil samples are used to determine the c_v-value in the laboratory or the parameter is just estimated, this may result in unrealistic consolidation times. It is strongly recommended to use in situ cone penetration tests (CPTU) including dissipation tests in different depths to achieve realistic results.
- Another value which has to be estimated (using a table quite common in France) is the value $\alpha = E_M/E_{oed}$. Users from other countries who are not familiar with pressiometer tests may not know this factor at all or make considerable mistakes.
- In the soil profile of Fig. 1, an incompressible stratum was assumed at 9 m depth. This assumption is only valid if hard rock is present. Otherwise an approach for calculating the limit depth, e.g. according to DIN 4019, should be used.

If the three above parameters are not accurate, a factor of two between measured and calculated results for final settlements and consolidation time may easily be reached.

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