Deformation and strength properties of the stone columns (crushed stone piles)

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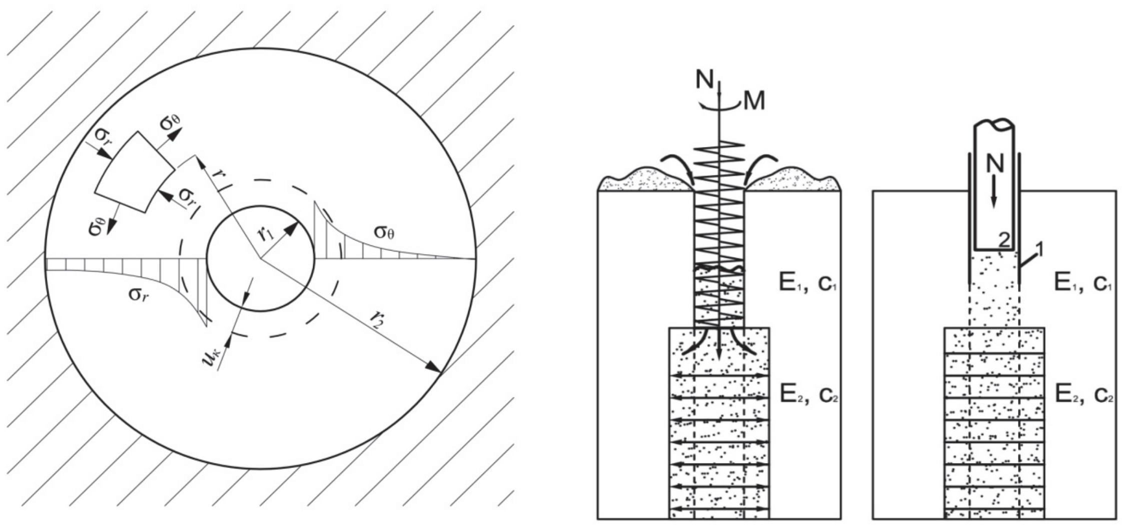
During construction on the soft clayey soils with limited thickness, bored cast in-situ crushed-stone drain piles (stone columns) are used often. They serve as drains during the preliminary deep (radial) compaction, and later on, together with compacted surrounding clayey soil (cell) serve as bearing member inside the individual cell as a part of the pile-raft foundation.

Diameter of cell and crushed-stone as well as the distance between the piles, arranged in a staggered manner, are selected in accordance with the load on a slab (raft) and physical and mechanical properties of the cell soils after their preliminary compaction. Experimental researches of the compacted soils around the columns and stone column show that the column is also getting compacted and strengthened including due to the intrusion of the part of surrounding clayey soil into its pore space (mud injection). In this case the column deformation modulus is getting increased and reaches up to 100 MPa. At the same time the deformation modulus of the compacted soft clayey soil (5 - 10 MPa) is increased up to 20 MPa, in this case the reduced cell deformation modulus in general is increased up to 30-40 MPa. The load on a cell is transferred through the raft (slab), distributed between the drain pile and surrounding soil over time due to the elastoviscoplastic soil properties. Depending on the correlation of the column cross section and surrounding soil is $(\omega = a^2 / b^2)$.

Obviously, it is necessary to perform the quantitative evaluation of total load distribution between the column and surrounding soil taking into account the deformation and strength properties of the column and surrounding compacted soil, as well as diameter of the column and cell (a, b). The column diameter depends on its production process at deep compaction, i.e. on the expansion of the leading bore-hole and its filling with crushed stone (fig. 1).



a) b)

Fig. 1. The expansion diagram of the leading bore-hole (a) diameter and its expansion technology for making the stone column (b).

It is known, that in case of the linear deformation of the column and surrounding soil the total stress on the cell (p), being transferred by the raft is distributed as followed:

$$P = \sigma_k \omega + \sigma_г (1-\omega) \tag{1}$$

where $\omega = a^2 / b^2$, at that

$$\sigma_r = p \frac{mk}{m_г \omega + m_k (1-\omega)}; \quad \sigma_k = p \frac{m_r}{m_г \omega + m_к (1-\omega)} \tag{2}$$

and the reduced factor of the relative compressibility and cell deformation modulus ($m_я \cong 0{,}8 / E_я$) have the following appearance:

$$m_я = \frac{m_н m_г}{m_г \omega + m_к (1-\omega)} \text{ or } E_я = E_k \omega + E_{г(1-\omega)} \tag{3}$$

This implies the importance of determination of E_k and E_r after making of the column and compaction of the surrounding soil.

2. Determination of the compacted soil deformation modulus

The compacted soil deformation modulus very tentatively can be determined through volumetric deformation of the compacted soil on the basis of compression curve. For weak soils at direct squeezing of the compacting unit the leading bore-hole is formed, to which the certain portion of crushed stone is filled and compacted by the same unit. In that case the volumetric deformation of the compacted soil will be equal to:

$$\varepsilon_r = \Delta V / V = (a^2 / b^2) K_{com} = \omega K_{com} \tag{4}$$

Where $K_{com} < 1$, the compaction coefficient that accounting for the partial loss of the compacted soil, which loss is squeezed out to the porous space of crushed-stone pile during the soil intrusion into the leading bore-hole walls. Therefore, the crushed-stone column with the diameter 2a appears inside the weak soil, which column takes the volume $\pi a^2 \times 1(\text{м}^3)$ at unit length. In case of presence of the leading bore-hole, being formed by a gate road, the formula (4) shall be amended taking into account the radius of leading bore-hole, i.e. we get:

$$E_r = [(a^2 - a_0^2) / (b^2 - a_0^2)] K_{com} \tag{5}$$

where a_0 is the radius of the leading bore-hole.

3. Determination of the stone column deformation modulus

Deformation properties of the stone column (crushed-stone pile), which pores are filled with clayey soil during the process of its making (mud injection) as a result of crushed stone pressing into the walls of leading bore-hole according to gate road or pressing technology (fig. 1). It should be determined, assuming that the stone column represents an aggregate consisting of the crushed-stone framework and compacted soil filling its pores, which provides the coherency to the crushed-stone pile.

4. Interaction of stone column with raft and surrounding compacted soil comprising the pile-raft foundation.

The first chapter of the present article provides the results of solution of the column interaction with surrounding soil as per the “point-bearing pile” scheme, when the compression of column and surrounding soil takes place in the linear arrangement. The solutions of problem taking into accounting fot the viscoelastic behavior of the surrounding soil and elastic-plastic behavior of the stone column are given below.

4.1. The column strain-stress state analysis with surrounding soil as a part of the pile foundation under the conditions of compression without taking into account the mutual influence of the pile and surrounding soil (as per the “point-bearing pile” diagram) (fig. 2)

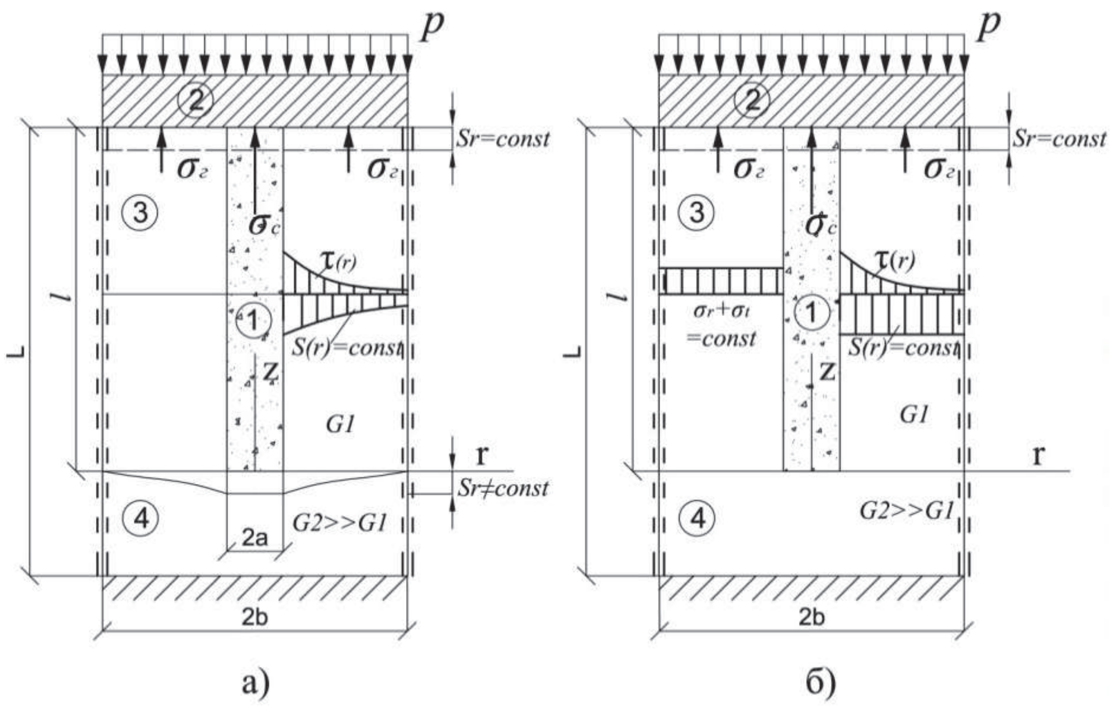


Fig. 2. The calculation diagram of interaction of bored cast in-situ crushed-stone pile of stone column with surrounding soil and raft as a part of the pile-raft foundation as per the "friction-bearing pile" scheme (a) and as per the "point-bearing pile" scheme (b).

Solution of this problem for the case of compression, when there are no radial displacements and stresses on the "pile-to-soil" contact, or when they are small and they can be neglected, should be considered as very tentative. It significantly simplifies solving of the set problem and allows to take into account the influence of different factors, including: non-linearity, creepage and consolidation of the surrounding soil.
Main initial equations for solving this problem are as follows:
1) equation of equilibrium:

$$p = \sigma_{z1}\omega + \sigma_{z2}(1-\omega) \tag{6}$$

where $\omega = a^2/b^2$; σ_{z1} and σ_{z2} are stresses in the pile shaft and in the surrounding soil;

2) equation of raft, soil column and surrounding soil settlement:

$$Sp=Sc=Sг \tag{7}$$

4.2. The account of elasto-plastic properties of the soil column
Under conditions of three-axial compression, which effects the soil column, non-linear longitudinal deformation can be determined on the basis of Hencky equation, i.e. we get:

$$\varepsilon_{z_1} = \chi(\sigma_{z_1} - \sigma_m) + \varepsilon_m, \tag{8}$$

where $\varepsilon_m = (\varepsilon_1 + \varepsilon_2 + \varepsilon_3)/3$ and $\sigma_m = (\sigma_1 + \sigma_2 + \sigma_3)/3$ is the volumetric deformation and medium stress, accordingly:

$$\chi = \gamma_i / 2\tau_i, \ \varepsilon_m = \sigma_m / K. \tag{9}$$

As the calculation formula for the purpose of description of the elasto-plastic deformation we can take the modified Timoshenko equation in the following form:

$$\gamma_i = \frac{\tau_i}{G^e} \frac{\tau_i^*}{\tau_i^* - \tau_i}, \tag{10}$$

where τ_i and τ_i^* are the intensities of the acting shearing stresses and its ultimate value accordingly, moreover:

$$\tau_i^* = \sigma_m tg\varphi_i + c_i, \tag{11}$$

φ_i and c_i are the parameters of the failure envelope in plane $\sigma_m - \tau_i$, moreover

$$\tau_i = \frac{\sigma_1 - \sigma_3}{\sqrt{3}}, \ \sigma_m = \frac{\sigma_1 + 2\sigma_3}{3}. \tag{12}$$

From (10) taking into account (11) we can determine:

$$\chi = \frac{1}{2G^e} \frac{\tau_i^*}{\tau_i^* - \tau_i}. \tag{13}$$

Value of σ_3 very tentatively can be determined through the surrounding soil passive pressure coefficient ($\sigma_3 = \sigma_{z_2} \xi_2$), assuming that:

$$\sigma_{z_2} = \frac{p\omega}{1-\omega} - \sigma_{z_1} \frac{\omega}{1-\omega} = \frac{p - \sigma_{z_1}}{1-\omega} \omega. \tag{14}$$

Then σ_3 we can determine as follows:

$$\sigma_3 = (p - \sigma_{z_1}) \frac{\omega}{1-\omega} \xi_2 \tag{15}$$

Putting this value σ_3 into (12), we get:

$$\tau_i = \frac{\sigma_{z_1}}{\sqrt{3}} - \left(p - \sigma_{z_1}\right)\frac{\omega}{1-\omega}\xi_2 \quad . \tag{16}$$

The deformation of surrounding soil we can determine based on the linear dependence $\varepsilon_{\Gamma} = m_{\Gamma}\sigma_{\Gamma}$, i.e. we get:

$$\varepsilon_{\Gamma} = \left(p - \sigma_{z_1}\right)\frac{\omega}{1-\omega}m_{\Gamma} \quad . \tag{17}$$

On the other side

$$\varepsilon_{z_1} = \frac{1}{G^e}\frac{\tau_i^*}{\tau_i^* - \tau_i}\left(\sigma_{z_1} - \sigma_m\right) + \frac{\sigma_m}{K} \quad . \tag{18}$$

Equating (17) with (18) we get the following:

$$\frac{1}{G^e}\frac{\tau_i^*}{\tau_i^* - \tau_i}\left(\sigma_{z_1} - \sigma_m\right) + \frac{\sigma_m}{K} = \left(p - \sigma_{z_1}\right)\frac{\omega}{1-\omega}m_{\Gamma} \quad . \tag{19}$$

Putting here the values τ_i, τ_i^* and σ_m here from (16), (11) and (12), we get the transcendental equation with respect to σ_{z_1}.
In case the material of soil pile has only friction (c = 0) and when under condition close to the ultimate the volumetric deformations ($\sigma_m / K \approx 0$) can be neglected from (18), the equation (19) is simplified and takes on the following form:

$$\frac{1}{G^e}\frac{\tau_i^*}{\tau_i^* - \tau_i}\left(\sigma_{z_1} - \sigma_m\right) + \sigma_{z_1}\frac{\omega}{1-\omega}m_{\Gamma} = p\frac{\omega}{1-\omega}m_{\Gamma} \quad . \tag{20}$$

Therefore, the set problem is completely solved. The dependence of unknown stress in pile σ_{z_1} is obtained from the load applied on the raft in the form of transcendental equations and in the simplified form.

Let us consider the example: l=20 m , a= 1 m, b=5 m , G^e = 20000 kN/m^2, υ_1=0,3, ξ_2=0,8, τ* = 135 kPa.

The sample calculation showed that this dependence is curvilinear and has the appearance as presented on figure 3.

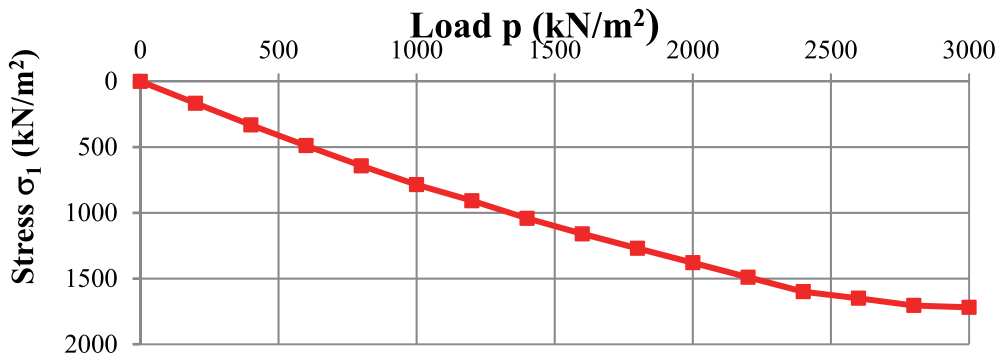


Fig. 3. Dependence of stresses σ_{z_1} from load p on the raft.

Putting the values σ_{z_1} into (20) we can determine the deformation of surrounding soil, and through it - the raft settlement as soon as $\varepsilon_{\mathrm{r}} = \varepsilon_{\mathrm{c}}$, $S_{\mathrm{r}} = S_{\mathrm{c}} = S_{\mathrm{p}}$ then we obtain:

$$S_{\mathrm{p}} = \varepsilon_{\mathrm{r}} l = \left(p - \sigma_{z_1}\right)\frac{\omega}{1-\omega} m_{\mathrm{r}} l \tag{21}$$

This dependence is curvilinear and has the appearance as presented on figure 4.

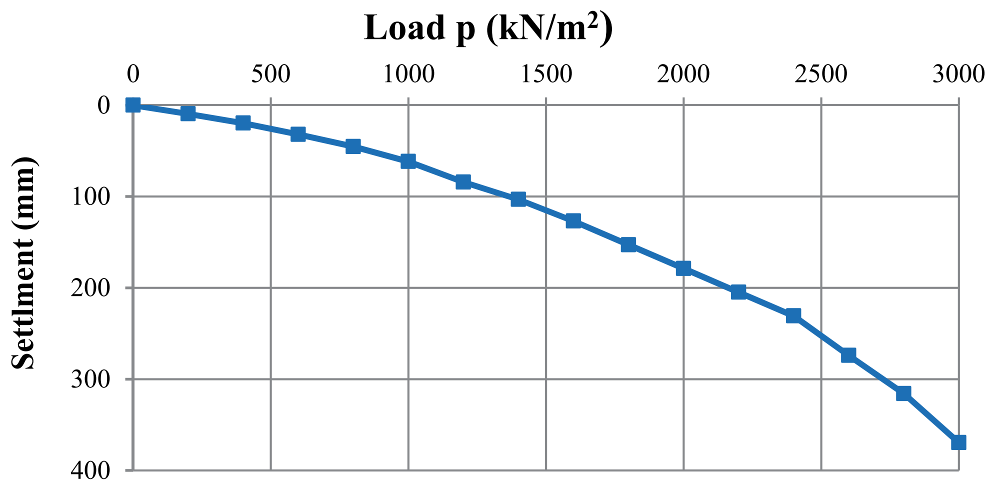
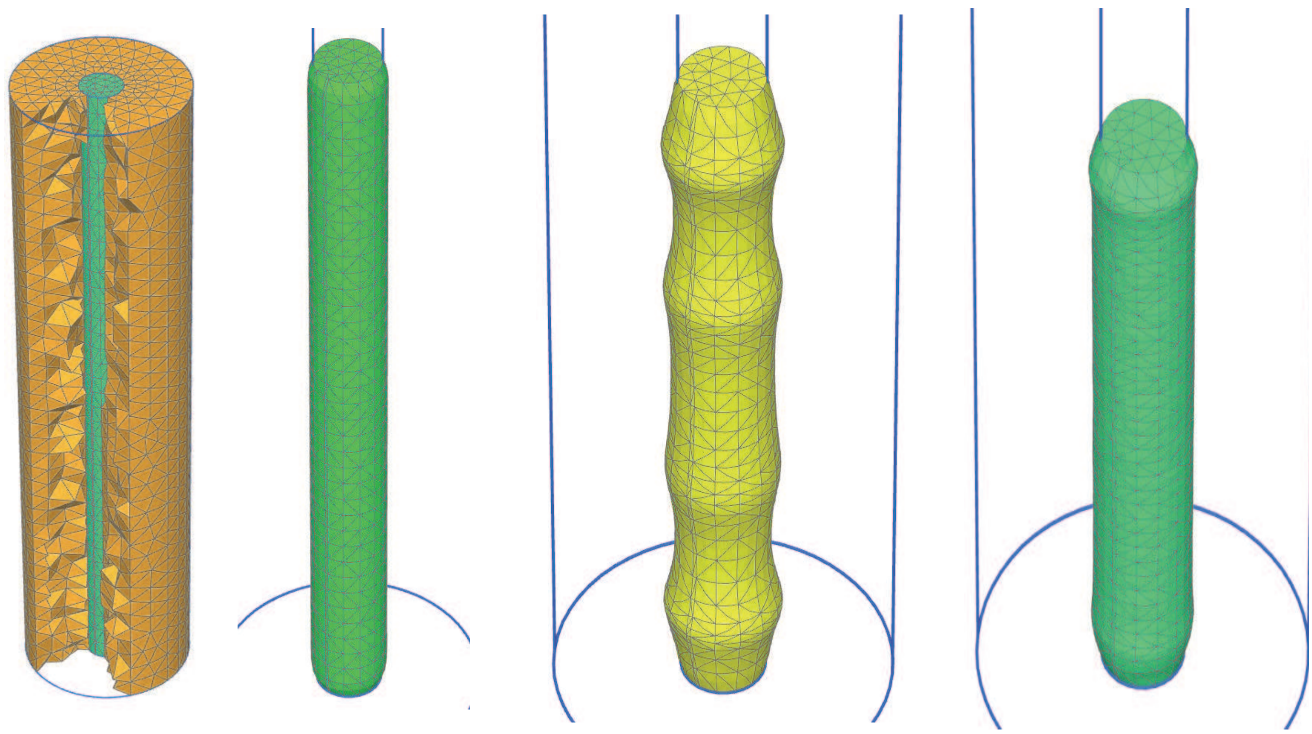


Fig. 4. Dependence of pile settlement - S` from load p.

For comparison purposes the set problem with the same parameters was solved by the numerical method of finite elements, that is much more simple. The elasto-plastic Mohr-Coulomb model was taken as the calculation model for soil column, and the linear model was taken for surrounding soil (fig. 5).

The calculation model by the finite elements method and the results of soil column calculations taking into account the interaction with surrounding soil in linear and non-linear arrangements are given on figure 5. Comparative analysis of the column strain-stress state, which column was calculated as per the Mohr-Coulomb models and hardening soil (HS) showed that they are significantly different when the ultimate condition is achieved. In the first case, the barrel-like swells of limited thickness (l≈2d) are formed having the certain repetitions throughout the pile length. Such form of soil failure has been determined for the first time. At the same time based on the results of the calculation as per HS model the barrel-like shape of the ultimate condition has not been detected.



(a) (b) (c) (d)

Fig. 5. The results of the numerical calculation of a soil column FEM taking into account the interaction with the surrounding soil in linear (a, b) and non-linear (c, d) arrangements.

Conclusions

1. During production process of the stone column of crushed-stone soil by deep (radial) compaction of the surrounding weak clayey soil, the partial penetration of weak soil to the porous space of the crushed-stone soil can take place. Due to the same, the column is getting strengthened and becomes coherent, besides, the weak soil when penetrating the pores of crushed-stone is getting compacted and strengthened.

2. The experiments show that the deformation properties of the column increase significantly and reach up to 100 MPa, and e of the surrounding weak soil grow from 5 MPa up to 20 MPa, at the same time the equivalent modulus of the cell (column + surrounding soil) reaches up to 30 to 40 MPa.

3. The deformation modulus of the compacted soil can be determined through its volumetric deformation on the basis of compression curve.

4. The column deformation modulus consisting from crushed stone, which pores have been filled with compacted clay, can be determined, when considering it as an aggregate on the basis of heterogeneous environments.

5. The column strength parameters can be determined only based on the results of field or laboratory experiment in the three-axial test machine for testing the large-size samples $(d \geq 30 см, \mathrm{h} \approx 2d)$.

Literature

Ter-Martirosyan Z.G. (1990)
Rheological parameters of soils and design of foundations. Moscow, Stroyizdat, 200 p.

Ter-Martirosyan Z.G. (2009)
Soil mechanics. Moscow, ABC Publishing House, 559 p.

Ter-Martirosyan Z.G. (1992)
Rheological parameters of soils and design of foundations, 1992, Oxford and JHB Publishing co.PVT.LTD. 190 p.

Vyalov S.S. (1976)
Rheological fundamentals of soil mechanics. Moscow. Vysshaya Shkola, 3610 p.