

DESIGN OF SUBSOIL IMPROVEMENT BELOW HALL FLOORS

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ABSTRACT.

The construction of an industrial park is now being prepared near the town of Nitra. The investor fixed very strict conditions for the bearing capacity and, above all, the settlement of halls and their floors. The geological conditions at the construction site are difficult: there are soft clay soils with high compressibility and low bearing capacity. A detailed analysis of soil improvement was made. Stone columns were prepared to be fitted into an approximately 5 m thick layer of soft clay. The paper shows the main steps used in the design of the stone columns

KEYWORDS: Engineering geological investigation, soil improvement, gravel columns, decreasing the settlement, consolidation.

1. INTRODUCTION

The construction of an industrial park was prepared near the town of Nitra in 2015. Difficult geological conditions, created by a layer of soft to firm Quaternary clays approximately 5 m in thickness resulted in unfavourable soil characteristics: low bearing capacity and high settlement. For this reason, it was decided to improve the soil properties. Following the evaluation of a preliminary engineering geological investigation, an initial design of stone columns was made during detailed investigation works. Before the final design, the efficiency of the stone columns was verified by field tests. Three test fields for three various improvement technologies were prepared. Penetration tests were applied before and 10 days after the stone columns completion.

In test field No. 1, stone columns with a diameter of 500 to 600 mm were prepared using the technology of deep vibrating compaction, and filled with gravel of grain size 4/15 mm, 8/32 mm and 4/32 mm. The stone columns were arranged to form a square and triangular grid.

In test field No. 2, dynamic compaction combined with stone columns preparation using a non-traditional technology was used for soil improvement. First, a borehole with a diameter of 600 mm was made and filled with gravel of grain size 0/63 mm or 0/90 mm containing low contents of fine particles. Soil particles smaller than 0.063 mm formed less than 5 %. The boreholes were not protected by casing pipes; they had to be filled by gravel as soon as possible to avoid the collapse of boreholes. The next step was compaction of the stone column in two phases by 50 hits. The stone columns were spaced within a square and triangular grid with varied distances from each other.

In field test No. 3, dynamic consolidation with a weight of 20 t falling in free fall from a height of 10 m was performed. Using this technology created

craters approximately 2 m in diameter. This squishy space was filled by stones with grain size 0-400 mm. The content of fine soil was very low. The space of compacted columns formed a square and triangular grid with a spacing of more than 5 m.

After the evaluation of data from field experiments [1], dynamic consolidation was rejected, because the results had proved this technology improper for application in local geological conditions. Also, the settlement of columns produced by dynamic compaction was higher than by the other technologies. As the depth reached by dynamic compaction was not sufficient for the project requirements, this technology was also rejected.

Field experiments helped to prepare recommendations which became input data for soil improvement. More halls with different floor loading will be constructed in the industrial park in the near future. The paper focuses on the design of soil improvement for uniform floor load intensity of 60 kPa.

2. DESIGN OF STONE COLUMNS

Various connections of columns to other structural layers were analysed before the design of the stone columns. After removing the topsoil, the separation geotextile was put on the ground level. Then, a 500 mm thick gravel layer was laid as a base below the embankment. The 500 mm layer was made up of two layers of crushed stone with grain size 0/125 mm compacted to min. $I_D = 0.85$. From this working plane, the stone columns were assembled. After they had been completed, the ground surface was levelled and compacted, as needed, to min. $I_D = 0.85$ again. The embankment was built on this base layer. Its first three layers levelled the non-horizontal terrain. The main part of the embankment was designed from these layers: 200 mm of the bottom layer of crushed stone 0/125 mm, 300 mm of pit-run gravel with small volumes of fine soil (grain size max. 100 mm), then

again 200 mm of crushed stone 0/125 mm and 300 mm of pit-run gravel with small volumes of fine soil (grain size max. 100 mm). The last 800 mm of the embankment were prepared from a 400 mm layer made up of crushed stone 0/63 mm and two 200 mm layers of crushed stone of grain size 4/32 mm. The main reason for using different types of material was the possibility of their delivery to the site. The length of the construction site was 1380 m. A schematic sectional view of improved subsoil is in Fig. 1, while Fig. 2 presents an overall view of making the stone columns.

One part of the construction site will contain halls whose floors will be loaded by 60 kPa. The investor's requirement for the settlement of the floors was 10 mm. The thickness of the embankment situated on improved subsoil was 1.5 m. Based on a parametric study, a raster of stone columns (SC) 1.8 · 1.8 m in a square grid was selected. The average length of the stone columns was 5.5 m. This length resulted from the requirement that the bottom of the columns would extend into the quarternary gravel layer. It was necessary to verify the bearing capacity, settlement and consolidation of the stone columns for the prepared project. The bearing capacity validation manifested great reserve and it will be not be dealt with this paper. A traditional analytical calculation method was applied for the calculation of settlement and consolidation, partly complemented by Priebe's theory for vertical gravel stone columns.

2.1. TRADITIONAL CALCULATION OF SETTLEMENT AND CONSOLIDATION

After the withdrawal of a humus layer, a 0.5 m thick gravel layer would be prepared on the ground. The stone columns would be built from this level. Calculations were made for a simulated model area of 20 · 20 m, as there must a reserve left for transport corridors and, if necessary, a free space for technology in the halls.

When a stone column with a diameter of 600 mm is made, the cross-sectional area equal to one stone column is

$$A_1 = \pi r^2 = \pi 0.3^2 = 0.283 \text{ m}^2 \quad (1)$$

Using the distribution of the stone columns in a square grid with a distance between the columns of 1.9 · 1.9 m, 27.67 columns will be placed in an area of 10 · 10 m (5.26 columns fit in a line up to 10 m in length, the distance between the rows is 1.9 m and the 10 m section holds 5.26 columns). Therefore, the total area of the columns will be

$$A_c = A_1 \cdot 27.67 = 0.283 \cdot 27.67 = 7.83 \text{ m}^2 \quad (2)$$

The original soil will occupy the area

$$A_s = 100 - 7.83 = 92.17 \text{ m}^2 \quad (3)$$

The following bedrock model was compiled for the area where a hall with a floor loaded by 100 kPa will be placed:

- 3.3 m layer of clay of high plasticity with mainly soft consistency; where $E_{def} = 3 \text{ MPa}$;

$$E_{oed} = E_{def}/\beta = 3/0.37 = 8.1 \text{ MPa} \quad (4)$$

- 1.2 m thick layer of clayey sand with mainly firm consistency; where $E_{def} = 5.95 \text{ MPa}$;

$$E_{oed} = E_{def}/\beta = 5.95/0.62 = 9.6 \text{ MPa} \quad (5)$$

- 1.0 m thick layer of silty gravel with medium relative density; where $E_{def} = 35 \text{ MPa}$;

$$E_{oed} = E_{def}/\beta = 35/0.74 = 47.3 \text{ MPa} \quad (6)$$

- Tertiary clay with high plasticity and firm consistency was found below silty gravel; where $E_{def} = 4 \text{ MPa}$;

$$E_{oed} = E_{def}/\beta = 4/0.37 = 10.8 \text{ MPa} \quad (7)$$

The average oedometric modulus value in a 3.3 m thick clay layer was determined using the equation:

$$E_{oed} = \frac{E_1 \cdot A_1 + E_2 \cdot A_2}{A_1 + A_2} = \frac{166 \cdot 7.83 + 8.1 \cdot 92.17}{100} = 20.46 \text{ MPa} \quad (8)$$

Similarly, for the average value of clayey sand the oedometric modulus determined was $E_{oed} = 21.85 \text{ MPa}$, for silty gravel $E_{oed} = 56.59 \text{ MPa}$ and for Tertiary clay $E_{oed} = 22.95 \text{ MPa}$.

The calculation of settlement was made (as described above) for an area of 20 · 20 m. The embankment layer thickness of 1.5 m acts on the surface of the original ground by a stress with an intensity of 1.5 · 20 = 30 kPa. It was anticipated that a soft clay layer 3.3 m thick with clayey sand and Tertiary clay would be compressed after improvement by 5.5 long stone columns in a raster of 1.9 · 1.9 m (fixing the stone columns 1.0 m into the gravel layer was considered). The calculation is processed in Tab. 1.

In the same way, the settlement due to a uniform surcharge of the floor by 60 kPa was calculated. The expected settlement under this load is $s_2 = 9.012 \text{ mm}$. The total estimated settlement (embankment + surcharge of the floor) will reach $s_{tot} = 4.886 + 9.012 = 13.898 \text{ mm}$. But the magnitude of this settlement needs to be corrected taking into account consolidation, because part of the settlement will occur during the construction time.

The calculation of consolidation in a drainage system consists of two parts: consolidation into vertical elements (stone columns) and consolidation into the horizontal permeable subbase. Then, using Terzaghi's theory the average degree of consolidation can be expressed as

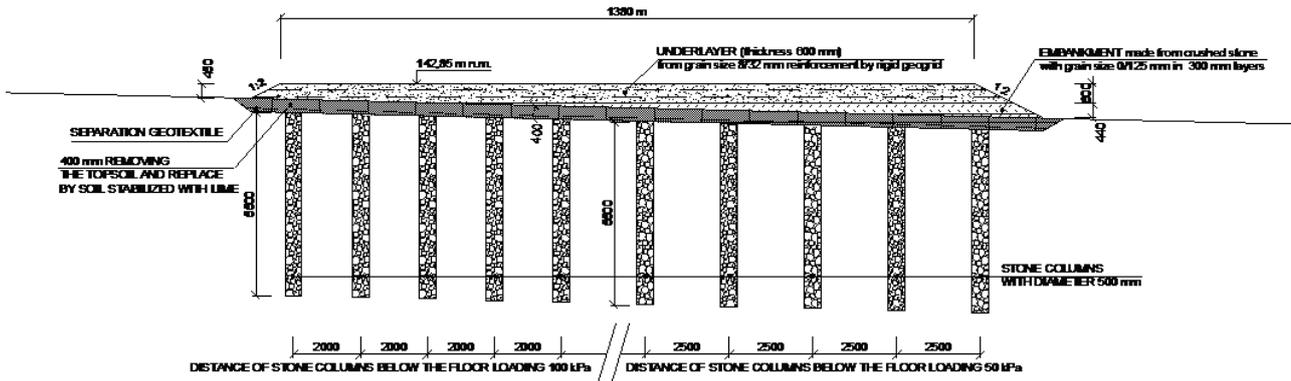


FIGURE 1. Schematic sectional view of improved subsoil.



FIGURE 2. Overall view on making stone columns.

Point no.	h (m)	z (m)	z/B	I_2	σ_2 (kPa)	σ_{or} (kPa)	$m_2 \cdot \sigma_{or}$ (kPa)	E_{oed} (MPa)	s (m)
1	1.6	0.80	0.040	0.98	29.4	16	1.6	20.46	0.002174
2	1.7	2.45	0.122	0.85	25.5	49	4.9	20.46	0.001712
3	1.2	3.90	0.195	0.71	21.3	78	7.8	21.85	0.000741
4	1.0	5.00	0.250	0.62	18.6	100	10.0	56.59	0.000152
5	2.8	6.90	0.345	0.52	15.6	138	13.8	47.30	0.000107
6	2.0	9.30	0.465	0.43	12.9	186	18.6	10.80	-
7	2.0	11.30	0.565	0.38	11.4	226	22.6	10.80	-
$\sum s_1 =$									0.004886

TABLE 1. Settlement of the embankment ($\sigma = 30$ kPa) after improvement by SC in a raster of $1.9 \cdot 1.9$ m. Explanatory notes: h - thickness of the soil layer; z - distance between the footing bottom and the point at which vertical stress is calculated; B - width of the foundation; I_2 - coefficient of stress spreading influence; σ_2 - vertical stress from the foundation at a depth z ; σ_{or} - original vertical stress at depth z ; m_2 - coefficient of structural strength; E_{oed} - oedometric modulus of soil; s - settlement of the soil layer with a thickness h .

$$U = 1 - (1 - U_h) \cdot (1 - U_v) \quad (9)$$

where:

U_h - degree of consolidation for horizontal dewatering,

U_v - degree of consolidation for vertical dewatering.

Firstly, the average degree of subgrade consolidation was determined:

$$E_{oed} = \frac{\sum E_i \cdot h_i}{\sum h_h} = \frac{8.1 \cdot 3.3 + 9.6 \cdot 1.2}{3.3 \cdot 1.2} = 8.5 \text{ MPa} \quad (10)$$

In the calculation of consolidation in the vertical direction, the time factor T_v was determined:

$$T_v = \frac{t_v k_v E_{oed}}{\gamma_w h^2} = \frac{1.0368 \cdot 10^7 \cdot 1 \cdot 10^{-9} \cdot 8.5}{0.01 \cdot 4.5^2} = 0.435 \quad (11)$$

where:

t_v - duration of vertical consolidation - 4 months
($4 \cdot 30 \cdot 86400 = 1.0368 \cdot 10^7$ s);

k_v - filtration coefficient in the vertical direction
($1 \cdot 10^{-9}$ m/s);

E_{oed} - oedometric modulus of soil (8.5 MPa);

γ_w - specific gravity of water (0,01 MNm⁻³);

h - thickness of the drainage layer of soil (4.5 m).

From Terzaghi's consolidation graph, we obtained the degree of consolidation $U_v = 83$ %.

These boundary conditions have been taken into account for the calculation of consolidation in the horizontal direction: $d_e = 1.13 \cdot d = 1.13 \cdot 1.9 = 2.147$ (where d is the axial distance of drains).

The time factor T_h for determining the degree of consolidation U_h is:

$$T_h = \frac{t_h k_h E_{oed}}{\gamma_w d_e^2} = \frac{1.0368 \cdot 10^7 \cdot 1 \cdot 10^{-9} \cdot 8.5}{0.01 \cdot 2.147^2} = 1.91 \quad (12)$$

where:

t_h - duration of horizontal consolidation - 4 months
($4 \cdot 30 \cdot 86400 = 1.0368 \cdot 10^7$ s);

k_h - filtration coefficient in the horizontal direction
($1 \cdot 10^{-9}$ m/s);

For $n = \frac{d_e}{d_1} = \frac{2.147}{0.60} = 3.58 \Rightarrow$ in this case, auxiliary graphs yielded $U_h = 99$ %. The resulting average degree of consolidation is then

$$U = 1 - (1 - U_h) \cdot (1 - U_v) = 1 - (1 - 0.83) \cdot (1 - 0.99) = 0.9983 \approx 100 \text{ \%} \quad (13)$$

Thus, it can be concluded that as long as the floor is not loaded earlier than 4 months after the completion

of embankment bodies, the consolidation below the embankment will be finished.

The measurable settlement of the hall structure, therefore, should only be the component induced by the operating load in the hall. The value of this part of settlement is 9.012 mm.

2.2. CALCULATION OF SETTLEMENT USING PARTIALLY PRIEBE'S THEORY

The solution comes from the definition of the equivalent diameter of the cell d_e . Auxiliary magnitudes:

equivalent diameter:

$$d_e = 1.13 \cdot 1.9 = 2.147 \text{ m}$$

cross sectional area of the stone column:

$$A_c = \pi r^2 = \pi \cdot 0.32^2 = 0.283 \text{ m}^2$$

area of a cell with a diameter d_e :

$$A_s = \pi r^2 = \pi \cdot 1.0735^2 = 3.62 \text{ m}^2$$

ratio of the column area to the cell area:

$$a_c = A_c/A_s = 0.283/3.62 = 0.078$$

stress in the foundation base of the floor:

$$\sigma = 60 \text{ kPa}$$

The basic design parameter is the so-called concentration ratio of stresses

$$n = \frac{E_c}{E_s} = \frac{180}{3} = 60 \quad (14)$$

where:

E_c is Young's modulus of the stone column,

E_s is Young's modulus of soil around the stone column.

The settlement of subsoil with reinforcement by stone columns is expressed by the parameter β . This parameter represents the ratio of the reinforced soil to original soil settlement. The parameter β can be approximately determined using the equation

$$\beta = \frac{1}{1 + (n - 1) \cdot a_c} = \frac{1}{1 + (60 - 1) \cdot 0.078} = 0.1785 \quad (15)$$

The calculation of settlement caused by a 1.5 m high embankment ($\sigma = 30$ kPa) without improvement and by the floor followed. The expected settlement of the embankment was $s_3 = 11.854$ mm and the floor settlement due to loading by 60 kPa was $s_4 = 20.150$ mm. Then, the total settlement without improvement is $s_{tot} = 11.854 + 20.150 = 32.004$ mm.

Using Priebe's theory the assumed settlement of the stone columns is reduced to the value

$$s = s_n \cdot \beta = 32.004 \cdot 0.1785 = 5.713 \text{ mm} \quad (16)$$

Comparing both theoretical methods we obtained the difference: 9.012 mm against 5.713 mm. This

difference can be explained in this way: in classical theory medium stiffness of subsoil was assumed, while in Priebe's theory the load is carried by the stone columns to a greater extent. According to both methods, the investor's requirement was fulfilled: the settlement was less than 10 mm.

3. CONCLUSION

The extremely exposed construction of an industrial park near the town of Nitra required improving an approximately 5 m thick layer of soft and firm Quaternary clays. In the first step, the suitable technology of soil improvement was selected. To this end, a field experiment in a 1:1 scale was carried out. Deep vibrating compaction was evaluated as the most suitable technology. In the second step, the design of stone columns spacing was made. The paper shows the procedure of the design of soil improvement below the floors exposed to uniform loading by 60 kPa. After the accomplishment of subsoil improvements of the whole area of the industrial park, the construction of halls started.

4. ACKNOWLEDGEMENTS

The paper presents part of the results from a research project supported by the Slovak VEGA Agency No. 1/0882/16 "The Influence of Boundary Conditions on Limit States of Geotechnical Structures".

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