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Geosynthetic reinforced retaining walls on ground improved soft soils

Murs de soutènement renforcés géosynthétiques au sol améliorés sols mous

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ABSTRACT: During the upgrade of electric transformation station near powerplant the original embankment was proposed to enlarge. Due to the land limits the only option in width direction was to propose very steep 7m height geosynthetic reinforced soil retaining walls. The proposal to use small concrete blocks as facing elements was accepted. As the extension of the embankment was located over existing already moved brook with surrounding area permanently wet with compressible clayey subsoil to about 12m depth and very sensitive equipment on top of the embankment, subsoil improvement was also required. In order to speed-up consolidation of the area, prefabricated vertical drains were used. In places with very sensitive equipment on top, stone columns were used as well in order to eliminate settlement. Details of the design, execution and monitoring will be presented in the paper.

RÉSUMÉ: Lors de la modernisation de la station de transformation électrique près de la centrale, il a été proposé d'agrandir le remblai d'origine. En raison de la terre limite la seule option dans le sens de la largeur était de proposer 7 m géosynthétique hauteur très raide renforcé les murs de soutènement du sol. La proposition d'utiliser de petits blocs de béton comme éléments de parement a été acceptée. Comme l'extension du remblai était situé au-dessus du ruisseau existant déjà déménagé avec ses environs humides en permanence avec le sous-sol argileux compressibles à environ 12 m de profondeur et de l'équipement très sensible sur le dessus du talus, l'amélioration du sous-sol a également été nécessaire. Pour la consolidation d'accélération de la zone des drains verticaux préfabriqués ont été utilisés. Dans les endroits avec des équipements très sensibles au sommet, des colonnes en pierre ont également été utilisées pour éliminer les tassements. Les détails de la conception, de l'exécution et du suivi seront présentés dans le document.

Keywords: Prefabricated vertical drains; Stone columns; Load transfer platform; Reinforced retaining wall; Ground improvement

1 INTRODUCTION

Existing embankment for electric transformation station constructed at the end of 1980-ties needed enlargement for transformation station rebuilt and capacity increase. The existing embankment required during its up to now life repairs as it showed signs of instability. The repairs were done by ballasting of the embankment toe via berm.

2 GEOTECHNICAL SUBSOIL PROPERTIES

At the beginning of the project first stage of Ground investigation was made for the planning inquiry project stage. This stage of investigation was made to identify the startigraphy of the subsoil with some index properties of identified strata and served for first geotechnical site model. Already this GI suggested that soubsoil is variably layered.

For the project stage of building permit, next phase of gound investigation was performed to obtain more information about the physical and mechanical properties of encountered soils and depth of underlying competent strata. This stage was performed for the purpose of getting all needed information for proper design of the embankment enlargement.

The existing embankment was identified as made from loose clayey to silty sands. The identified loose density at the embankment crest was attributed to the slope instability.

Top parts of the subsoil were characterised as clayey sand ranging to sandy clays of firm consistency. The underlaying subsoil was made from clay with layers of different consistency from soft to stiff. These clay layers were at their base interbedded with layers of organic soil (peat and coal). The underlying competent strata was identified at the enlarged embankment crest as wethered paragneiss in the depth of about 15 m below ground level. The weathering rate was decreasing with depth.

However in the trial pit outside of the new embankment footprint the competent strata was highly weathered sandstone in the depth of about 2m below ground level. This nicely shows the difficult ground conditions we have to tackle in this project with geological fault and steeply inclined underlying rock.

From performed laboratory tests on disturbed and undisturbed samples we have cautiously derived characteristic properties of encountered soils, which are presented in Table 1.

Table 1. Characteristic properties of encountered soils

Soil	Unit weight	Effective friction angle	Effective cohesion	Undrained cohesion	Edometric modulus	Consolidation coefficient
Existing embankment	18,5 kN/m^3	26°	5 kPa	-	6 MPa	-
Sandy clay	18,5 kN/m^3	23°	8 kPa	50 kPa	6 MPa	6×10^{-9} m^2/s
High plasticity clay / Organic soil	20,0 kN/m^3	17°	8 kPa	50 kPa	6 MPa	5×10^{-9} m^2/s
Clayey sand	18,5 kN/m^3	26°	10 kPa	-	29 MPa	10×10^{-9} m^2/s
Weathered paragneiss	20,0 kN/m^3	30°	15 kPa	-	50 MPa	-
New embankment	20,0 kN/m^3	26°	5 kPa	-	30 MPa	-

Excavated material from different part of this large project was proposed to be reused for new embankment. This material was mainly composed of sandy silts with moisture higher than optimal according to Proctor standard tests by 2 – 8%, therefore modification by about 3% of lime was proposed to allow for compactibility.

Groundwater level is very close to the ground level and corresponds to the water level in the brook which passes through the site.

3 EARTH STRUCTURE GEOTECHNICAL DESIGN

3.1 Earth structure geometry

Landownership of the investor influenced Earth structure geometry as well as spatial arrangement of the electric transformation station technology prior, during and after the update. The technology update is planned to be done without any outage of the transformation station operation. In the direction of embankment prolongation, the spatial limitations were not so tight and natural slopes with inclination of 1:3 could fit in. However, on sides where it was necessary to widen the embankment within very narrow strip, steep slopes with inclination of 66° are designed. In the highest point the embankment is 7 m high.

As the technology on top of the embankment is very sensitive to settlements and mainly to differential settlements, the requirement from the owner was to minimize the time needed for embankment settlement (max. 1 year) after construction.

3.2 Design of subsoil improvement

3.2.1 Subsoil consolidation

First of all, the design ground model was developed for embankment settlement. This model was used to calculate the settlements under the embankment without any measures and concluded that the total value of settlement is 114mm and the time needed for full consolidation is about 220 years, however 90% of consolidation will happen within 25 years. These results were not acceptable for the owner.

In order to shorten the consolidation time, we have suggested to use prefabricated vertical drains (PVD) as the first option. The optimization of the design using the equation of Kjelmann (1) (see e.g. Vaníček & Vaníček, 2008) up to the minimum sensible distance of drains was done, which resulted in drains down to 11 m in triangular grid with 0,8 m spacing. Even this arrangement did not shorten the consolidation time as the owner expected, when the 90% consolidation would be reached in 27 months since embankment construction.

$$t = \frac{D^2}{8c_h}\left[\ln\left(\frac{D}{d}\right) - \frac{3}{4}\right] \cdot \ln\frac{1}{1-U_h} \tag{1}$$

Where t (s) is consolidation time, D (m) is diameter of drained soil cylinder, d (m) is equivalent diameter of PVD, c_h (m^2/s) horizontal coefficient of consolidation of given soil layer, U_h is average degree of consolidation.

As a second option we have suggested to use stone columns to the depth of 15m and 0,6m in diameter with rectangular spacing of 3,5m to reduce the total settlement and also to shorten the consolidation time. The calculation has been performed using FEM code PLAXIS using axisymmetric calculation option. The results indicated total settlement of 30 mm with 5 mm during the construction and rest within 20 years.

The final proposal was to use both techniques together to achieve both goals of consolidation time shortening to the minimum and to reduce the total settlement. This combination satisfied the client to have residual settlements within rectifiable limits of 10 mm once the sensitive technology will be put in operation. On Figure 1 is presented the comparison of different subsoil improvement techniques we have worked with during the design process with respect to time and settlement value.

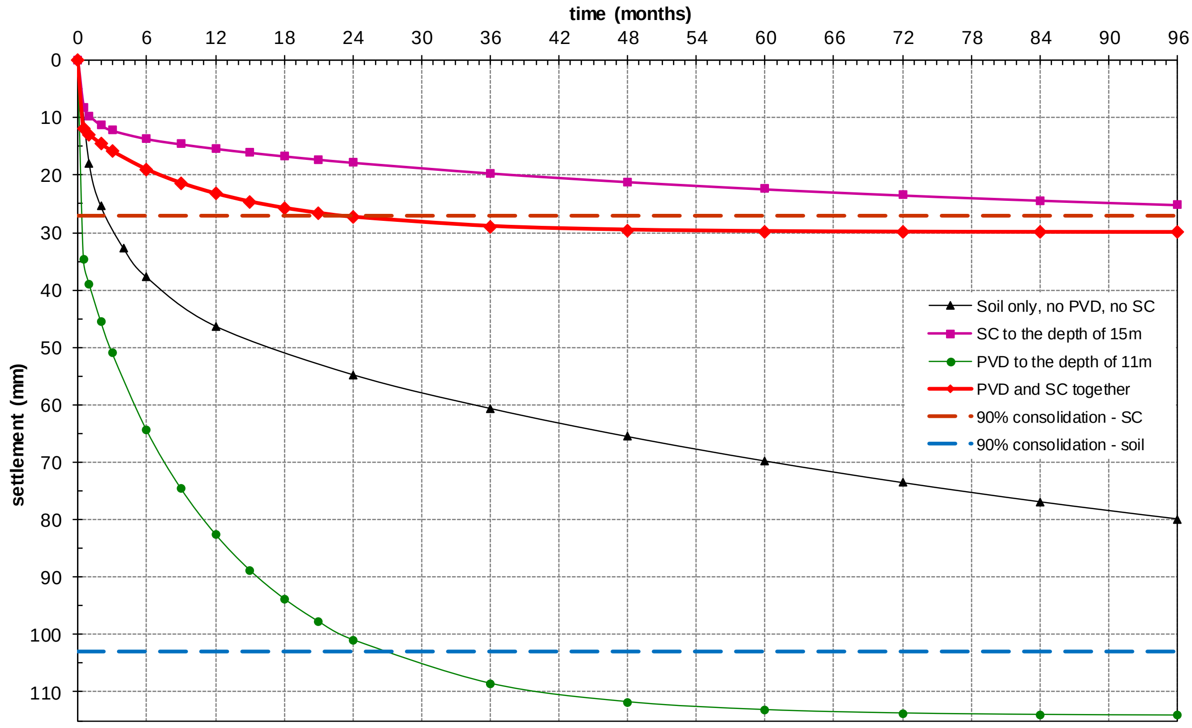

Figure 1: Time dependent settlement comparison for different techniques of subsoil improvement

3.2.2 Load transfer platform

For as much as possible uniform settlement of the whole embankment over stone columns it was necessary to design reinforcement of the load transfer platform above the columns. Alexiew (2005) presented a nice overview of the design methods applicable for load transfer platform. However, finally, the most conservative method according to British standard BS8006-1:2010 was selected. This method assumes that after subsoil settlement between individual stone columns there will be no contact between subsoil and reinforcing platform. Subsoil will not support the platform and reinforcing geogrids will be able to transfer all loading from fill to the columns by membrane theory.

Based on this calculation long-term design strength for reinforcing geogrids were determined. These were with respect to reduction factors for creep, installation damage and environmental effects for a design service life of 120 years transferred to the ultimate strength of geogrid. As these reduction factors are geogrid specific, it is not possible to determine the ultimate strength in geogrids generally. The outcome from design calculations is the requirement for tensile force of 400 kN/m at max. elongation of 3%. These requirements fulfils geogrid Fortrac® R 400/30-30 MPT placed in two layers in each direction, thus 4 layers altogether. In order to achieve the optimized performance of the whole load transfer platform the crushed gravel mixture 0/63 with grading category GA according to EN 13285 in 100 mm thick layers was placed between individual geogrids.

3.3 Design of reinforced retaining wall

In the highest section the retaining wall is 7m high. This section is outside of the area with ground improvement by stone columns and load transfer platform. As completely new embankment is built here the length of individual geogrid layers was not limited in any way.

However, in other sections, where the embankment was not so high, limits on geogrid layers were implied by the current partly cut embankment. The extend of cuts was on the other hand limited by the operational technology on top of the embankment. For the highest sections next to the current embankment the design was really very sharp.

To keep the facing of such wall nice and tidy the proposal to use small hollow concrete block as facing elements was accepted by the owner. The blocks are 170 mm high and define slope of facing at 66° due to staggering between successive layers of blocks. The embankment construction and geogrid placement were proposed in layers of 170 mm (block height) multiplier 1, 2 or 3. The design calculations for layout optimization were made for height increments of 510 mm, i.e. 3 blocks.

As described earlier the ground conditions were very complicated due to unexpected layering and therefore the calculations were made using rather conservative average properties of the whole subsoil above weathered bedrock.

The design calculations have been done according to the EN 1997-1, Eurocode 7: Geotechnical design - Part 1: General rules. Design approach 3 was selected for the actual calculations as it is almost the exclusive approach for slope stability analyses across whole Europe. For this design approach the characteristic soil properties are reduced by partial safety factors on material and surcharges are increased by partial safety factors on loadings. When applying this design approach, the determined forces in geogrids are directly design forces with positive influence on the analysed earth structure.

The structure design will be presented here for the highest section, i.e. 7m high reinforced retaining wall. For slope stability calculations in-house software SVARG was used, it is based on the rigorous method of Janbu (Vaníček & Vaníček 2000, Vaníček & Vaníček 2001). First of all, the analysis using total stresses and undrained soil properties was performed while minimising the ratio between driving and withstanding stresses, formerly known as factor of safety.

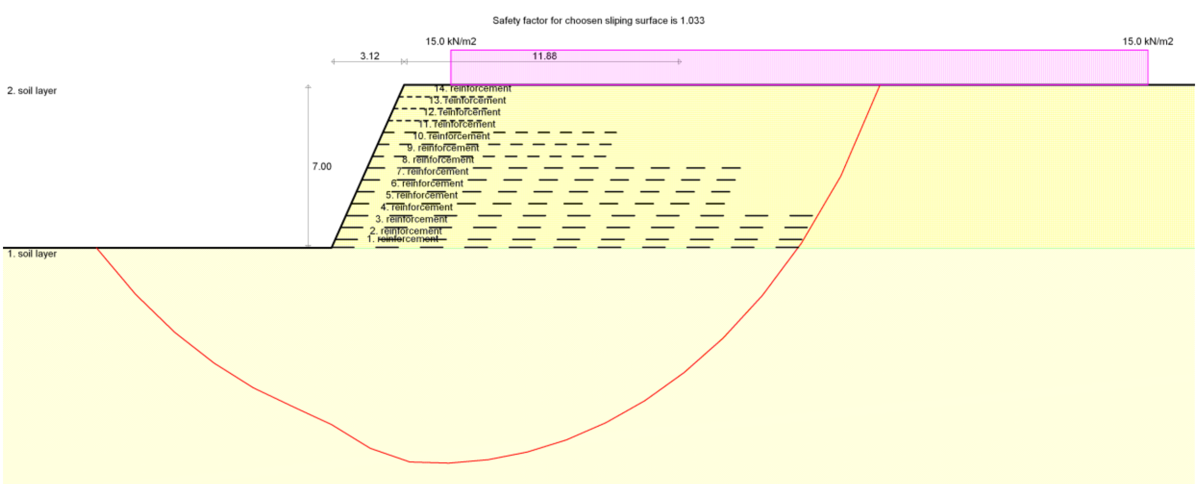

Figure 2: Structure overall stability check for total stress analysis (state just after construction)

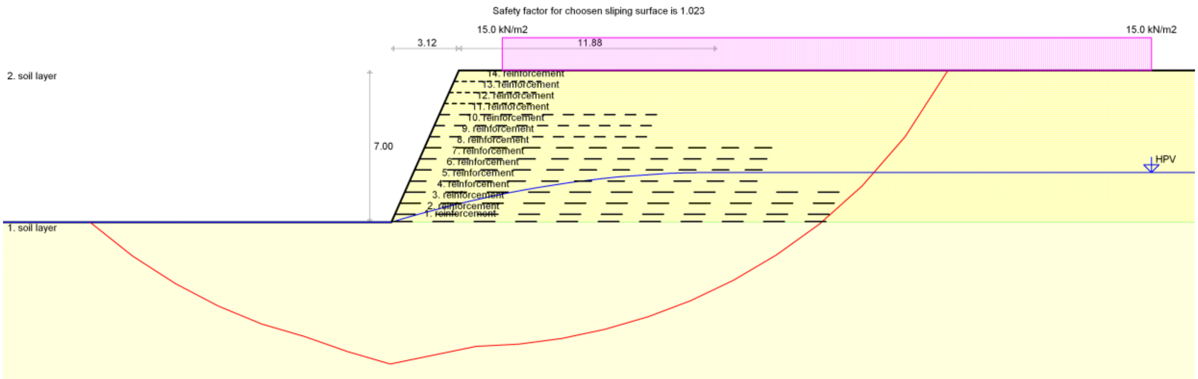

Figure 3: Structure overall stability check for effective stress analysis (state for end of service life)

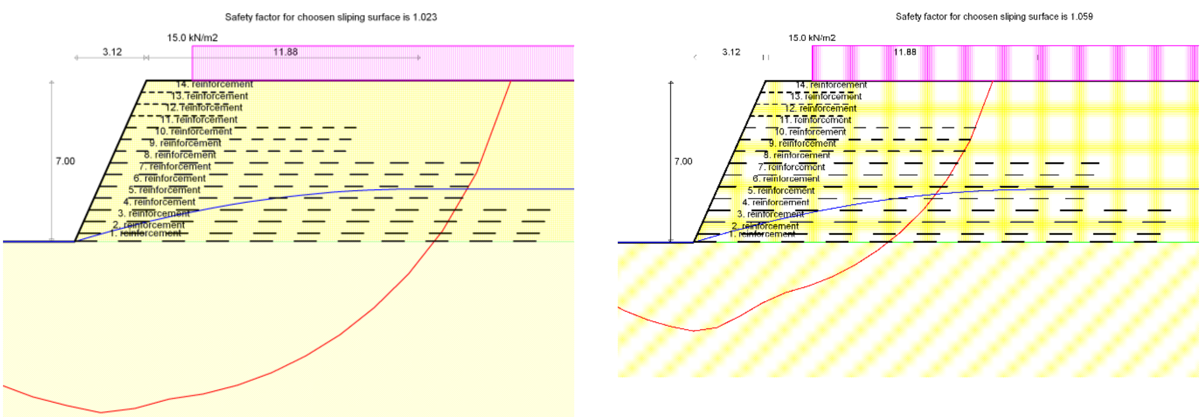

Figure 4: Structure compound stability checks for effective stress analysis (state for end of service life)

According to the design approach 3 this factor of safety shall be bigger than 1.00 for satisfying the safety requirements of the design. The total stress analysis is for checking if the structure is safe just after the construction, see Figure 2.

Than followed slope stability analysis for the end of 120 years' service life using effective stresses and drained soil properties. In this case we assumed that the sustained ground water level can be as high as one third of the embankment height inside the earth structure. The calculations have been made using optimisation of the potential slip surface. This optimisation is sometimes tricky as the process can lead only to local minimum and therefore several checks have to be made, mainly for overall stability (Figure 3) and for compound stabilities behind the blocks of reinforcing elements with their different length as is shown in Figures 4a and 4b.

And finally, the structure was checked as quasi-homogeneous gravity retaining wall, when the zone of reinforcement is assumed as stiff structure. This includes bearing capacity and sliding checks, which was done using in-house spreadsheet according to the design approach 3.

As the result of the stability calculations there was a specification for reinforcing geogrids, which assumed after optimisation 4 different grades of geogrids Fortrac® MPT – 35, 55, 80 and 110, lower at the top and higher at the bottom. Fortrac® MPT was required, as the embankment soil was modified by quick lime.

4 CONSTRUCTION PROGRESS

Construction sequence is documented on the photos that are presented at following Figures. The first step – in May 2017 – was to remove topsoil (organic) and unusable soils with high moisture content close to liquid limit up to the thickness of 0,6 m. This was followed by installation of monitoring system comprising piezometers in wells and hydrostatic horizontal inclinometers on the embankment base in June. In July the installation of stone columns, to improve subsoil settlements, took place (see Figure 5). In August and September, the installation of prefabricated vertical drains (Colbonddrain® CX 1000) have been performed (see Figure 6). At the end of September contractor installed the load transfer platform (see Figure 7). Finally, the construction of reinforced retaining wall started in October (see Figure 8). In about half-way up the construction of the wall was suspended for winter break from December to April 2018. In September the wall and embankment construction were finished and the technology construction of the transformation station could be started as it is shown in Figure 9. On wall facing blocks several geodetic points were placed for wall movements monitoring.

5 CONCLUSIONS

Presented case shows that even for very complicated subsoil conditions it is possible to construct relatively high and steep embankment on which a rather sensitive technology (from the serviceability limit state point of view) of the electric transformation station is situated. The solution was achieved by utilizing several techniques of special foundation engineering and soil improvement, as are stone columns, prefabricated vertical drains, reinforced load transfer platform and reinforced soil retaining wall. The monitoring results till November 2018 does not show any unpredictable results, hence confirming that the structure behaves as expected.

Figure 5: Subsoil improvement by stone columns

Figure 6: Subsoil improvement by prefabricated vertical drains

Figure 7: Load transfer platform construction

Figure 8: Reinforced soil retaining wall construction

Figure 9: Finished reinforced soil retaining wall with small concrete facing blocks and slope of 66°, view of maximum height section

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