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Road embankments with reinforced base on peat

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ABSTRACT: Highly compressible peat ground always represents a challenge for a geotechnical engineer. Two road projects in SW Czech republic that faced peat ground for construction of embankments are described. In one project the thickness of peat reached almost 4 m, peat moisture was in the range of 600-650% and embankment height 1.4 m. In the second project the maximum peat thickness was 2 m, moisture content 420-440% and height of embankment was 5 m. The geotextile-reinforced base of embankment was used in both projects to increase stability of the fill, eliminate differential settlement and enable movement of the construction plants on site. In both cases, predicted settlement, calculated from laboratory tests results on undisturbed samples, was significantly lower than measured by the monitoring system in the field. A numerical model was used to explain the difference.

1 PROJECT DESCRIPTION AND GEOTECHNICAL DATA

Road project No.1 is the Strazny town by-pass on highway I/4 near the Czech-German border crossing that was recently finished and opened to public. The 1st class highway carries mainly truck traffic. The geological conditions of the new road alignment are highly unfavourable and change along the alignment from extremely hard rocks that require blasting for excavation to highly compressible peat subsoil. Approximately 1 km long section of highway is on embankments that reach 5 m height. Subsoil is composed of muskeg and peat of 2 m thickness. Peat deposit lies on clayey sand that passes to strong granite bedrock. Water table is 0.4 m below the surface of the terrain.

The soil investigation included boreholes, in-situ vane tests and laboratory tests on samples. Porosity of peat measured in laboratory was n=80% (e=4), dry density 80 to 85 kg/m^3, water content 420 to 440%.

The results of the in situ vane test, realised at various depth of the peat layer, gave the undrained strength between 10 and 50 kPa. In places with higher quantity of undecayed organic matter the vane test result reached 100 kPa or even more. The strength distribution was rather erratic. Oedometric modulus of deformation on undisturbed samples was in the range 0.3 to 0.4 MPa for stress range 25 to 50 kPa and 0.4 MPa for stress between 50 and 100 kPa. Coefficient of primary consolidation was measured between 1.0×10^{-7} and 1.0×10^{-8} m^2/sec with significant secondary consolidation after primary compression.

Effective shear strength, measured by direct shear box tests showed very small scatter of results, φ=31 to 33^0, cohesion c=4 to 9 kPa. Undrained strength as measured by vane test was c_u=10 to 40 kPa.

In the place of maximum peat thickness the road embankment reached 5 m. The settlement, based on the laboratory results, was estimated to reach 0.6 to 0.7 m.

Project No.2 at Dobrany consisted of a service road constructed in the flood plain of the river. The road was used during construction of the bridge and served for traffic of heavy trucks with concrete, pile driving machines etc. The designed embankment height was 1.4 m. The subsoil profile composed of 0.2 to 0.4 m of topsoil with grass turf followed by 3.4 to 3.6 m of fibrous peat. Medium dense alluvial sand formed the “bedrock” in the area. Water table was practically at the surface of the terrain.

Strength of the peat material was measured by cone penetration and vane tests in the field and by laboratory vane and unconsolidated undrained triaxial test on undisturbed samples in the laboratory. The specific dynamic resistance of the peat layer, determined by light cone penetration test (10 kg rammer), was at some places as low as 20 kPa and maximum values were around 300 kPa. Field vane test measurement of the peat resulted in average undrained strength of 23 kPa, laboratory vane test result was 11kPa and UU triaxial test gave c_u=10 kPa with φ_u=1^0 (cell pressure 10 to 50 kPa).

Compressibility of peat was measured by oedometer tests and resulted in E_{oed}=0.13 to 0.2 MPa for σ_v=20 to 40 kPa. Coefficient of consolidation, calculated from the time-settlement field observation curve, was in the range of 10^{-6} to 10^{-7} m^2/sec.

Effective shear strength was determined in CIUP triaxial tests at cell pressures 10, 30 and 50 kPa. The result was φ =27^0, c=6 kPa.

2 STABILITY AND SETTLEMENT OF EMBANKMENT

Due to relatively high effective shear strength of peat the long-term stability of embankment was sufficient. However, a real problem was short-term stability during construction and extreme and irregular settlement.

Short-term stability was analysed using LCPC nomographs for embankments on soft subsoil. In Dobrany the marginal stability was already reached with embankment height 1 m (reduced value of cohesion used in calculation was c_u=5 kPa). In the case of Strazny the max height was 2 m (for c_u=10 kPa) for unit weight of fill 2 t/m^3. Inclination of the side slopes was taken as 1V:3H.

Total settlement was estimated by simplified calculation as follows:

Dobrany

s_{max1}=h.γ.z/E=1.4x20x3.6/200=0.504 m

In the second step of calculation the height of embankment was increased by 0.5m (value of estimated settlement).

s_{max2}= 1.9x20x3.6/200=0.684 m

This was the maximum settlement value we expected to occur on site.

Strazny

s_{max1}=h.γ.z/E=5.0x20x2.0/400=0.50 m

As in the case of Dobrany we increased the embankment height for the value of estimated settlement and repeated the calculation.

s_{max2}= 5.5x20x2.0/400=0.55 m

This value was considered as maximum settlement of the embankment.

Calculation of settlement with oedometric modulus of deformation measured in laboratory on undisturbed samples (E=0.2 MPa in Dobrany, E=0.4 MPa in Strazny) was judged correct and conservative enough because under the wide embankment base one-dimensional compression should prevail.

3 CONSTRUCTION SEQUENCE

3.1 *Strazny site*

Due to very soft subsoil at the site no heavy construction plant could enter the place. After clearing the ground from trees and shrubs only (stumps were razed as close to the ground surface as possible) the geotextile reinforcement was rolled manually over the area perpendicularly to the centreline of the highway. Adjacent strips of geotextile overlapped 0.5 m without being fixed by sewing or other stiff connection. Protruding tree stumps were covered by separate pieces of geotextile as protection against damage of the main reinforcement strips that were rolled over them. Polyester geotextile of 200 kN/m tensile strength in longitudinal direction and 50 kN/m in transverse direction was used throughout the project. The measuring pipes for subsoil settlement observation by hydrostatic method were installed in two sections under the geotextile reinforcement.

Crushed aggregates were end-tipped at the edge of the peat ground and spread by dozers over the laid geotextile. Thickness of the first layer was 0.3 m. After spreading the aggregates (without compaction) the free ends of geotextile were folded back over the layer to form a mattress (Fig. 1) and the second layer of geotextile was rolled over it. Next layers of embankment were spread and compacted in usual manner. It took three years to finish the highway project. Placing of the pavement was delayed until the very end when the increment of settlement under the embankment decreased to few millimetres in one month.

Figure1. Strazny. Embankment base reinforced with geotextile

The embankment base was widened by 3 m in order to accommodate min 0.5 m of estimated settlement. The main body of the fill was constructed from pervious granular material, mostly sandy soil or aggregates produced by on-site crushers that worked out the rocks blasted in the nearby cuts.

3.2 *Dobrany site*

Also this site was inaccessible for current construction plant and a similar procedure as at the Strazny site was used. As no trees were on site the geotextile was laid down directly on turf. High-strength polypropylene geotextile (500 kN/m in longitudinal direction and 40 kN/m in transverse direction) in one layer was used in this case. Measuring pipe for monitoring of settlement was installed at one cross section prior to rolling out the geotextile. Crushed aggregate of 0.4 m thickness was spread over the geotextile by a dozer placing the material in the center a little ahead than at the embankment edge. By this way the mud wave produced by weight of aggregates on peat was pushed aside and did not endanger the stability of the fill. After placing the first layer of aggregates without compaction the free ends of geotextile were plied over the spread aggregates layer and formed a mattress (Fig.2).

Figure 2. Dobrany. Geotextile-reinforced embankment base

Subsequent layers were placed in the same way as the first one but compaction was possible by currently used rollers. As soon as the embankment reached its final height the pavement was constructed on the top and the construction traffic started immediately. The road was under heavy traffic for two years during construction of the bridge. Although the settlement of the road increased by 200 mm the pavement did not suffer any deformation.

4 MONITORING OF CONSOLIDATION

Monitoring of settlement of the embankment was done in periodical intervals corresponding to gradual increase of the fill height. Hydrostatic leveling method was used for this purpose in both projects.

4.1 *Strazny site*

Coefficient of consolidation c_v measured in the laboratory during oedometer tests was in the range between 10^{-7} and 10^{-8} m^2/sec. For lower value of c_v the major part of the peat subsoil compression should take 925 days or 2.5 years.

As can be seen from the periodic measurement of settlement in Figure 3, that shows the shape of compression at the contact between embankment and peat subsoil, and Figure 4, showing the consolidation curve at the centerline of the embankment, the prediction made from the laboratory tests is in good relation to reality. Now, at approximately 2.5 years from the beginning of the filling and two years since the completion of the embankment, the rate of settlement decreased to negligible values. However, the total predicted settlement is relatively far from reality (550 mm calculated vs. 900 mm measured till today). The shape of settlement curve shows very clearly the effect of geotextile reinforcement. The middle 10 m wide section under the road pavement is flat with no differences in settlement. Average extension of geotextile, calculated from the settlement profile, is 0.2% which is much less than expected (reinforcement force corresponding to max 5% extension was considered in calculations).

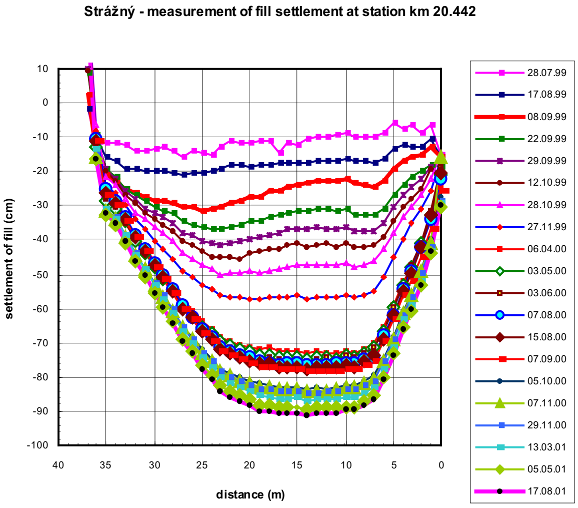

Figure 3. Strazny. Shape of compression curves under embankment

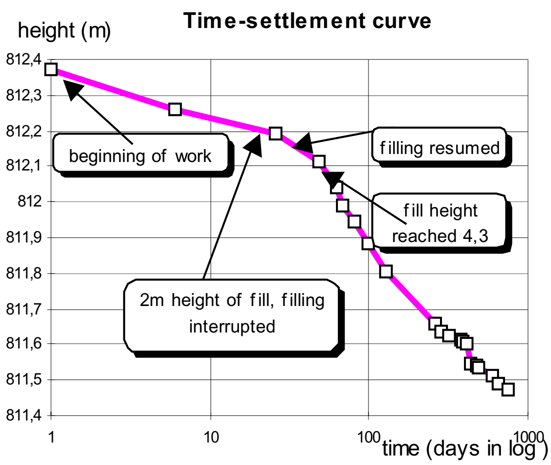

Figure 4. Strazny. Time-settlement curve below the centerline of the embankment

4.2 *Dobrany site*

As the coefficient of consolidation was not measured in the laboratory tests, it was estimated from the field measurement of settlement. Its value was of one order higher than in the case of Strazny site (10^{-6} to 10^{-7} m^2/sec).

Shape of the compression curve was not so flat in its middle part due to higher heterogeneity of the peat subsoil and smaller stresses because of the lower embankment height. Time-settlement curve (√days vs. settlement in cm) is in Figure 5 and the finished embankment in Figure 6. In spite of 200 mm settlement during the use of the service road (there was no time to wait until the consolidation is over) no cracks or irregularities appeared on the pavement. The geotextile reinforcement compensated all settlement differences and prevented formation of tension cracks. Also in this case the maximum measured settlement was more than 30% higher than prediction. Geotextile extension did not reach 1% (five times less than expected).

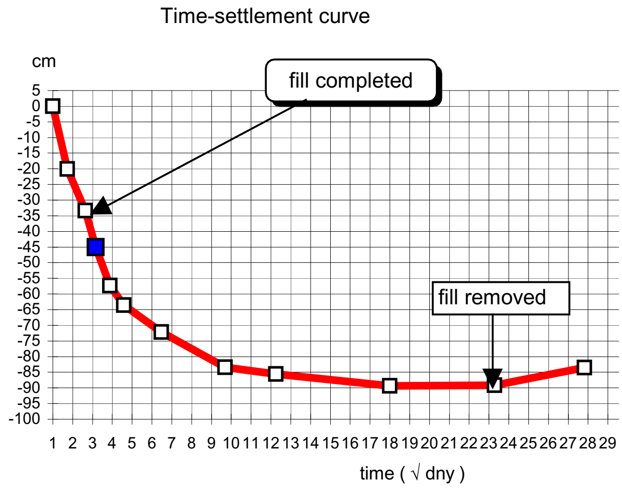

Figure 5. Dobrany. Time-settlement curve under the embankment

Figure 6. Dobrany. Finished road on peat

5 FE ANALYSIS OF SETTLEMENT

Because of the limited space the FE simulation will be presented for Dobrany site only. An elastic-perfectly plastic Mohr-Coulomb model was used for the description of the subsoil. The list of parameters is given in Table 1.

Table 1: Parameters of the subsoil used in FE calculations

Layer	E [MPa]	ν [-]	φ [^{0}]	c [MPa]	ψ [^{0}]	γ [t/m^3]
Crust	20	0.3	27.	0.025	0	1.5
Peat	0.2	0.1	27.	0.006	0	1.0
Sand	20	0.3	35.	0.001	10	1.7

There are several possible approaches how to simulate numerically the settlement induced by loading of subsoil by the embankment construction: one-dimensional (oedometric) compression, trapezoidal loading and comprehensive analysis including embankment fill and geotextile (Figure 7).

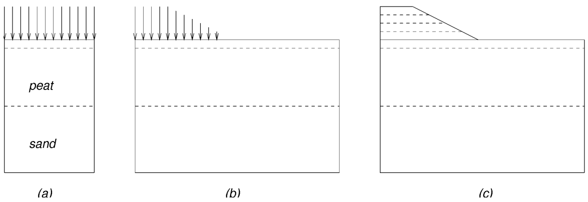

Figure 7. Numerical models for the calculation of embankment settlements.

One-dimensional compression is a standard way for the settlement calculation. The Updated Lagrange (large strain) formulation should be used for the calculation of strains in case of large deformations. The difference between the FE results in small and large strains, respectively, exceeds 20% in the present analysis. Applying small strains, the calculated value 0.65 m corresponds well to the estimated settlement.

A single value of the elasticity modulus E is but a crude approximation since the behaviour of soils is clearly pressure-dependent. Using a power law for E in the form

$$E = E_0 \, (\sigma/\sigma_0)^n$$

is more realistic. Assuming n=0.6 together with large strains, the calculated settlement reaches 1.02 m which is very close to the measured value in situ.

Oedometric conditions can be useful for the estimation of the maximum settlement but one does not get any results of the settlement profile over the embankment cross-section. Thus a further step may be a simulation of the trapezoidal loading (Figure 7b) of the soil surface which represents an embankment without reinforcement. The FE results shown in Figure 8 reveal that the settlement profile does not necessarily have its maximum at the embankment symmetry axis. This effect arises due to the horizontal spreading of the subsoil in less loaded areas where the stress-dependent soil stiffness is lower.

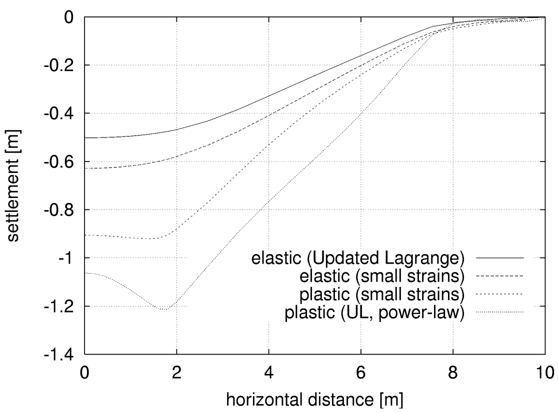

Figure 8. Settlement profile for the trapezoidal surface loading.

Numerical simulation of the construction of embankment including geotextiles (Figure 7c) is the most realistic but at the same time the most demanding approach. Especially the interface contact between soil and geotextile still poses a challenge for the numerical modeling.

The analysis of the Dobrany site was performed using the FE code TOCHNOG with trussbeam elements representing the geotextile and with normal and tangential contactsprings at the interface. Figure 9 shows the reinforcing effect of the geotextile which contributes to a more homogeneous settlement profile (cf. Figure 3). More details on the calculation procedure can be found in Herle & Herle (2001).

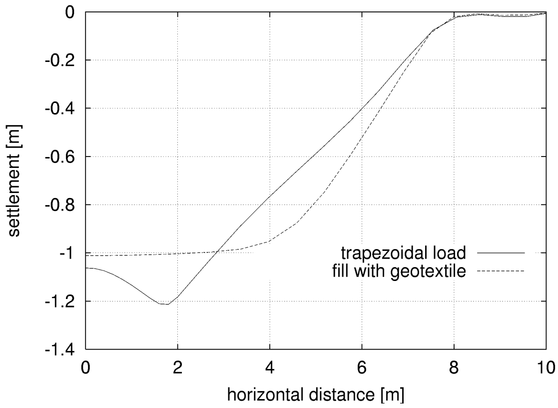

Figure 9. Settlement profile with and without geotextile.

6 CONCLUSION

Two case studies of the road constructions on peat bring the following experience:

- Use of geosynthetics reinforcement makes the construction of embankments on peat possible.
- Differential settlement and tension cracks are practically eliminated.
- Extension of geosynthetics is generally small (less than 1%) even at extreme settlements.
- Settlement prediction is difficult and use of oedometer test results with a single value of E may underestimate total settlement.
- More exact prediction of settlement and short-term stability could be reached by use of a comprehensive numerical models that includes also time effects (coupled consolidation and creep)

7 REFERENCES

Herle, I., Herle, V., 2001. Road construction on a soft organic subsoil. Proc. 15th Int. Conf. Soil Mech. Geot. Eng., Istanbul, A.A.Balkema Publishers, 2081-2084