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Monitored construction of a high embankment on soft soil, reinforced by stone columns

Etude et construction d'un remblai sur sol mou renforcé par colonnes ballastées

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ABSTRACT
In 2003 a 11.5 m high embankment was constructed on approximately the same site where, 17 years previously, the soft alluvium deposits of the Rižana river failed beneath the 7 m high embankment, which was being built at that time at this location. The subsoil beneath the new Srmin embankment was reinforced by approximately 12 m high stone columns in order to ensure sufficient safety during and after construction, as well as small relative settlement between the embankment and the neighbouring deeply-founded structures during operation of the motorway. The results of design calculations have been compared with the measurements performed during construction of the embankment.

RÉSUMÉ
Dans un site compressible dû aux alluvions de la rivière Rižana, un remblai routier s'était rompu en cours de construction à 7 m de hauteur il y a 17 ans. Cet accident a conduit, pour la construction récente d'un remblai de 11,5 m de hauteur dans le voisinage immediat à renforcer prélablement le sol de fondation par des colonnes ballastées. L'objectif a été de réduire le risque de rupture pendant et après la construction, ainsi que les tassements différentiels au contact avec un viaduc et d'autre ouvrages d'art fondés sur pieux. On compare les tassements calculés du remblai aux tassements mesurés lors de la construction.

1 INTRODUCTION

During the rapid construction of a motorway in the coastal region of SW Slovenia in 1986, a sudden subsoil failure occurred beneath the 7 m high Srmin embankment. Due to recent extension of this motorway, it was necessary to construct a 600 m long and up to 11.5 m high embankment on the same flat area with similar ground conditions. Past experience showed that the construction of such an embankment would only be possible after ground improvement had been carried out or lateral embankments provided. The construction of a viaduct, and use of lightweight fill, were also both considered, but rejected due to high costs. After preliminary analyses of possibilities of ground improvement had been carried out, stone columns were selected for the following reasons:

- stone columns improve stability, enable a high rate of construction due to accelerated consolidation, partly reduce primary and secondary settlements, and in this particular case lateral embankment could be omitted,
- lateral embankments could have solved the stability problem, but would have occupied a significant additional area of agricultural land, while not solving the problem of differential settlement between the embankment and its neighbouring structures, which are founded on stiff piles.

The geotechnical design of the embankment was based on field and laboratory data from three boreholes, and three CPTU sounding results. Additional boreholes were made during construction for the installation of inclinometer and piezometers casings. The CPTU tests were repeated in order to verify the increase of undrained shear strength. Settlements were monitored using settlement plates and three horizontal profile gauges.

A total of 6500 stone columns, each with a diameter of 60 cm and a length of 12 m, were installed in the subsoil, at 2.25 m centres in both directions.

2 GEOTECHNICAL INVESTIGATIONS

In 1999 three 15-16 m deep boreholes were constructed on the future site of the new embankment. The samples obtained showed fairly uniform ground conditions along its route, but it was noted that somewhat softer ground coincided with the location of the highest point of the embankment.

Three 12-13 m deep CPTU tests were performed, as well as dissipation tests, which provided the most reliable data for consolidation analysis.

The ground conditions are presented in Fig. 1, which shows the cross-section of the embankment at its highest point. The following soil layers were distinguished:

- At the top, a 5 to 5.5 m thick firm brown clay of high plasticity,
- a 4 to 5.5 m thick layer of soft gray to black organic silty clay,
- a 2 to 4 m thick layer of dense silty gravel,
- marl bedrock.

Table 1: Characteristic ground properties from laborat. and in-situ tests

Layer	CH firm	CH-OH soft	gravel
water content w_0 (%)	26 – 33	27 – 60	
liquid limit (LL)	61 – 73	34 – 97	
plasticity indeks (PI)	42 – 50	16 – 65	
unit weight γ (kN/m^3)	18.6 – 20.0	16.1 – 19.8	
triaxial UU test q_u (kPa)	81 – 187	38 – 83	
undr. strength c_u (kPa)	67	20	
effective shear strength	$c' = 7$ kPa; $\varphi'=27.7°$	$c' = 13$ kPa $\varphi'=25.8°$	$\varphi'=38°$
constr. modulus M (MPa)	5.9 – 6.6	1.7 – 4.1	30 – 44
permeability k (cm/s)	$3–8*10^{-9}$	$8–16*10^{-9}$	
secondary consolidation (α_e)	0.005 – 0.0075	0.006 – 0.03	

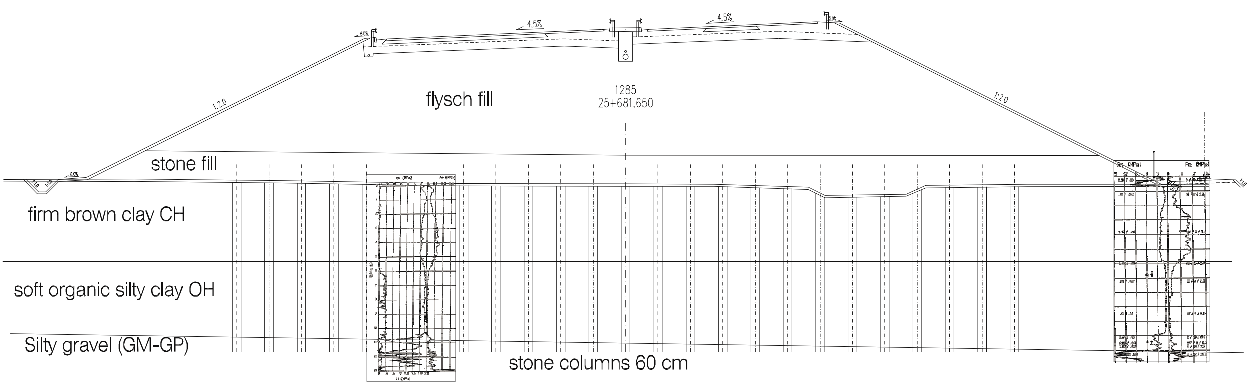

Figure 1. Characteristic cross section: soil layers, stone columns, embankment, CPT results

Groundwater was encountered in the gravel layer. Its artesian piezometric level is usually 0.5 to 1 m beneath the ground surface. The soft clayey layer was fully saturated, but the top layer of brown clay was not.

For the design the less favourable properties, which were obtained at the location of the highest point of the embankment, were used. The data obtained from the subsoil investigations and from back-calculation of the previously failed embankment were also considered.

In order to obtain an estimate of the increase in undrained shear strength of the subsoil, which could be expected at the site of the new embankment, use was made of measurements, which had been performed in the vicinity of a nearby, 7 m high previously constructed motorway embankment. Over a period of 13 years the undrained shear strength of the soft organic clay had increased from 20 to 60 kPa, and that of the upper firm clay from 60 to 85 kPa (no ground improvement was used for this motorway section).

3 THE DESIGN OF STONE COLUMNS

The main design criteria were the stability of the embankment, and the reduction to a minimum of relative settlements between the embankment and its neighbouring structures by improving the consolidation rate. The centre-to-centre distance between the stone columns was defined so that 95% of consolidation would occur during the construction period of 1.5 years. Thus the calculated centre-to-centre distance was, in the case of columns having a diameter of 60 cm, 2.25 m.

Settlements of the improved ground beneath the embankment were calculated using the Pulko method (Pulko, 2000, 2005). The estimated settlement for a 11.5 m high embankment was 72 cm.

The stone columns contributed to the stability of the subsoil beneath the embankment by significantly enhancing the rate of consolidation, and by their high shear strength. Stability analyses were performed using a Plaxis FEM computer programme assuming the Mohr-Coulomb material model for all soil layers. Initial analyses showed that very rapid construction of the embankment would not be possible for stability reasons. It was therefore decided to construct the embankment step-by-step. Rapid construction was allowed up to a height of 7 m. At this stage the subsoil was allowed to consolidate, and settlements were measured. Taking into account the already mentioned expected increase in undrained shear strength, the following safety factors were calculated (Table 2):

Table 2: Calculated safety factors from the design calculations

Construction/calculation stage	SF
6 m high embankment (c_{u1}=60kPa, c_{u2}=20kPa)	1.65
11.3 m high embankment (c_{u1}=60kPa, c_{u2}=40kPa)	1.20
11.3 m high embankment (c_{u1}=80kPa, c_{u2}=60kPa)	1.35

These results showed that the expected increase in the strength of the subsoil beneath the 7 m high new embankment would enable the safe construction of the embankment to its total design height.

Detailed analyses, including 3D FEM analyses, led to the decision to omit the installation of nearly 3000 stone columns which would otherwise have had to be constructed, on both sides of the embankment, under a 8 m wide strip, where the embankment height is less than 4 m (see Fig. 1).

Numerical analysis of safety, based on c_u values measured during construction, showed that 2D and 3D models gave comparable results (Table 3). In table 3 a comparison is given of the safety factors which were obtained for three different cases: case A, where the stone columns are built as shown in Fig. 1, case B, where they are built under the entire embankment, and case C, where they are built even outside the embankment. In the results referred to as 'undrained', undrained fast filling was assumed between a height of 9 m and a height of 10 m.

Table 3: Safety factors obtained with 2D and 3D Plaxis models for 10 m high embankment

	Case A	Case B	Case C
Plaxis 2D - undrained	1.33	1.37	1.39
Plaxis 3D - undrained	1.34	1.39	1.39
Plaxis 2D - drained	1.37	1.40	1.42
Plaxis 3D - drained	1.38	1.40	1.40

4 CONSTRUCTION

The embankment was constructed mainly from flysch material which was excavated in nearby cuts for the same motorway, but the first 2 m were made from crushed stone. The stone columns were installed using two construction methods: the vibro-displacement technique and the casing method with vibratory withdrawal of the steel casing.

The original plan was to construct the embankment up to a height of 7 m, to wait for 4 months to allow for consolidation to take place, and then to complete the embankment without pavement layers. Since most of the soft sub-layers consolidated faster then had been foreseen, the construction was faster than planned. Only 1 month after the height of 7 m was reached, construction continued to the 9 m level, and one month later the embankment was completed, except for its highest part next to the neighbouring viaduct. This part was completed one month later since geotechnical measurements had confirmed that the ground was softer in this area. Nevertheless, the embankment was completed sooner than expected, in May 2003. The pavement for the motorway was 1 m thick and was constructed 15 months after the earthfill embankment had been completed, and 3 months before the motorway was opened for traffic.

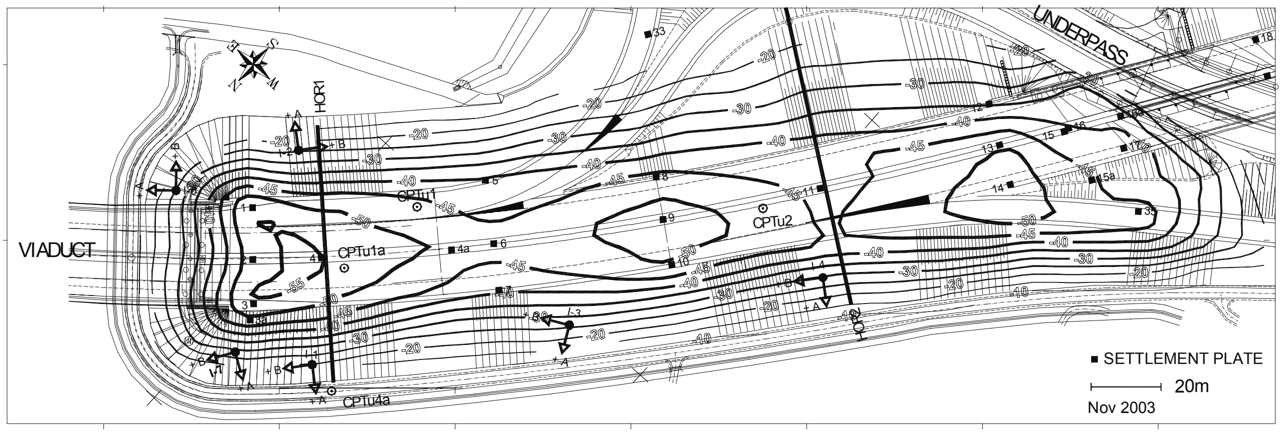

Figure 2. The Srmin embankment layout between the viaduct and the underpass together with measuring points and measured settlements in November 2003.

5 GEOTECHNICAL MONITORING

The established geotechnical monitoring system consisted of:

- Surveying of settlements on 41 settlement plates (SP),
- Settlement measurements at three cross-sections, using a hydrostatic profile gauge (HOR),
- Measurements of horizontal displacements using inclinometers (I),
- Repeated measurements of undrained shear strength using CPTU tests (CPTu),
- Excess pore pressure measurements using a BAT system.

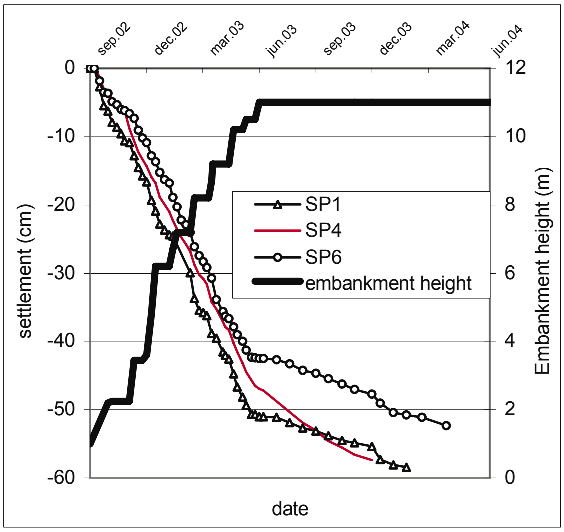

Figure 3. Time-settlement and time-loading curves

The layout of the embankment is shown in Fig. 2, together with the locations of individual measuring points and with the lines of equal settlement in November 2003, about 5 months after the earthfill embankment had been completed. The results of measurements of the selected settlement plates SP1, SP4 and SP6, located close to the viaduct, are presented in Fig. 3, which also shows the increase in height of the embankment over time. It should be noted that the settlement plates were not installed on the original ground surface but later, when the embankment was already 1 to 1.5 m high. This means that roughly 5 cm of settlement had occurred before the measurements started in September 2002. The observations of settlement plates were terminated after April 2004, when preparations for pavement construction started. The largest measured settlement was 58 cm. Taking into account the fact that the top 1 m thick pavement was missing, that consolidation had not yet been completed, and that 5 cm of settlement had occurred before the measurements were started, it can be seen that the settlement prediction (class A) was very good.

Settlements of a similar order were measured using a hydrostatic profile gauge HOR-1 (Fig. 4). The locations of the two measured profiles are given in Fig. 2. The first measurement was performed in October 2002, and the last one just before the motorway was opened for traffic i.e. when the embankment already had its pavement structure. Thus only the 5 to 6 cm of settlement, which occurred before the first measurement, have to be added to the maximum settlement at profiles HOR-1 and HOR-2.

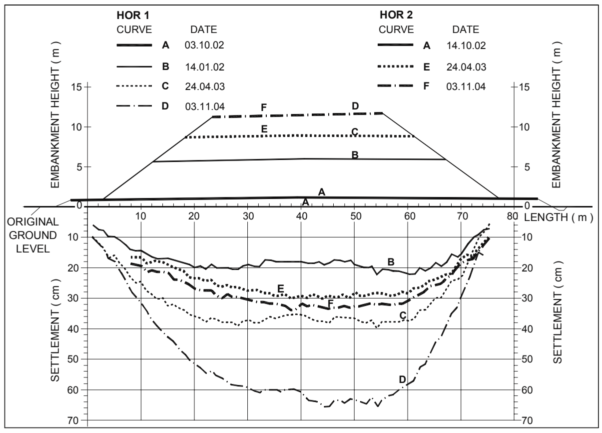

Figure 4. Measured settlement profiles HOR-1 and HOR-2

Measurements of settlements across the entire profile of an embankment show the differences in the ground and loading conditions along its base, and are very useful for assessing the amount of ground distortion that takes place beneath embankments. From Fig. 4 one can see that after fast filling the maximum settlements at HOR-2 are in the middle of embankment, which is not the case for profile HOR-1, where the maximum settlements were observed on the two opposite sides of the embankment. Larger distortional settlements therefore occurred at profile HOR-1, which was confirmed also by the results of vertical inclinometer measurements, which showed larger horizontal displacements next to profile HOR-1 (locations I-1, I-2, I-3 and I-7) than those measured at location I-4 (see Figs. 2 and 6).

Advancement of the consolidation process was monitored by the analysis of time-settlement behaviour only. When the construction was interrupted for a month in order to allow for consolidation of the subsoil, an Asaoka analysis (Magnan et al., 1983) was performed (Fig. 5). This analysis showed that most soil sub-layers consolidated radially in 55 to 70 days – much faster than expected. The same analysis performed at the end of construction with a much longer observation period revealed that some sub-layers needed the expected period of 18 months for consolidation.

The results of maximum horizontal displacements measured in the vertical inclinometers are shown vs. time in Fig. 6. Note that the first reading was not made at the same time for all the inclinometers, but the results are plotted from the same origin in order to be comparable. Only the results of the first installed inclinometer I-1 is shown twice: (1) from the first reading on (denoted by I-1), and (2) together with the other inclinometers (denoted by I-1*). A horizontal movement of 7 cm occurred in 50 days at I-1. The inclinometers were installed in the middle of either embankment slope, so an important part of the horizontal displacement occurred before monitoring begun. The maximum measured horizontal displacement was 22 cm, at I-1. The total displacement can be estimated to be more then 30 cm. It occurred mainly in the soft organic clay layer (Fig. 7). This item of data was useful for the construction of the piled foundation of the viaduct. In order to avoid the large horizontal loads which are caused by ground movement, the viaduct's abutment was constructed only after these movements had occurred.

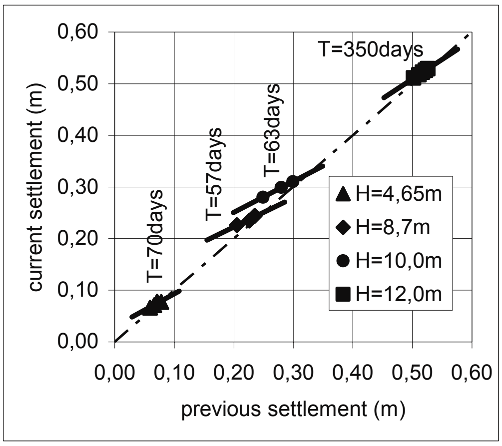

Figure 5. Analysis of measured settlements by the Asaoka method. T is back-calculated time for 99% consolidation at each marked load step.

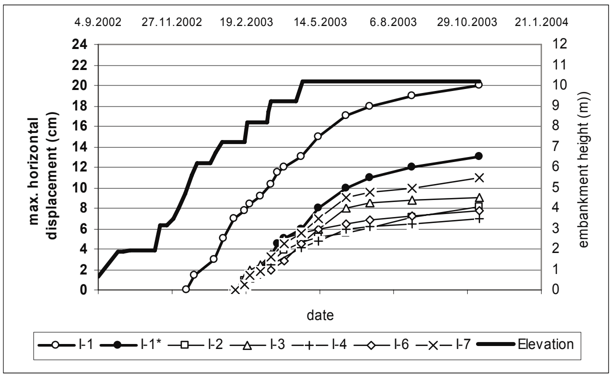

Figure 6. Maximum horizontal displacements obtained from inclinometer measurements vs. time

Apart from the displacement-monitoring systems, repeated CPT measurements of the undrained shear strength of the subsoil were, during construction, the most important means to check the stability of the next loading step. The results for sublayers with similar c_u values are summarised in Table 4. The first measurements (CPTU-1) were made before construction began, the second (CPTU-1*) after the embankment had reached the height of 7 m and was allowed to consolidate for 1 month. Both these tests were made at nearly the same location. The third test (CPTU-1a) was made after the embankment had been allowed to consolidate for 1 month at a level of 9 m. The location was changed (see Fig. 2) due to the fact that the right side of the embankment showed larger vertical and lateral displacements and the c_u values at this location were generally lower.

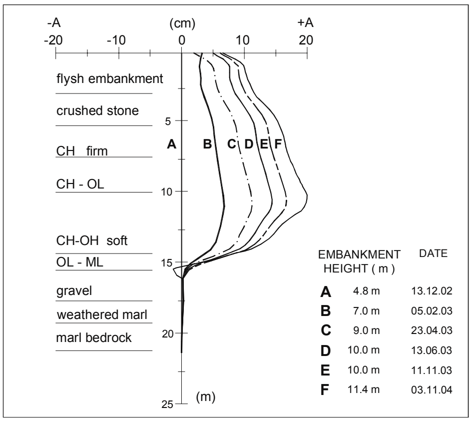

Figure 7. Horizontal displacement in the subsoil at location I-1

Table 4: Measured c_u values for characteristic sub-layers before and during construction

Depth (m)	*c_u (kPa)* *CPTU-1*	*c_u (kPa)* *CPTU-1**	*c_u (kPa)* *CPTU-1a*
0.0 – 2.0	47	71	50
2.0 – 5.5	88	105	85
5.5 – 7.0	19	27	26
7.0 – 9.0	24	61	35
9.0 – 11.0	24	61	60

6 CONCLUSIONS

The design and construction of the Srmin embankment on soft ground followed the principles of the observational method. The behaviour of the embankment and the subsoil beneath it, which had been reinforced by stone columns, was regularly monitored during construction, and the measured data were constantly analysed. In this way it was possible for the works on this demanding site to proceed fairly quickly and safely, in contrast with the previous, unsatisfactory experience with unreinforced subsoil.

The results of settlement and other measurements proved the validity of the design method used, and the appropriateness of the chosen characteristic values of the soil properties. At the same time the use of the approach with agreed step-by-step construction, whose progress was controlled by the results of analyses of the measured values, facilitated the decision to omit a significant number of stone columns, which would otherwise have been needed along both edges of the embankment.

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