Numerical modeling of embankment failure and its renewal

Modélisation numérique d'un remblai instable et sa stabilisation

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ABSTRACT: Based on preliminary investigation data sufficient safety had been expected for a 10 m high motorway embankment across a 100 m long shallow valley. During filling of the embankment a 60 m long fissure was observed, parallel to the road axis. Additional in-situ tests showed that the subsoil conditions were much worse than expected. Finite element and potential slip surface analyses of the failure mechanism were performed. A possible failure mechanism was found numerically but unfortunately it was not proven on site by inclinometers. Extensive in-situ, laboratory and numerical investigations were performed and measures for improving the embankment and subsoil stability were designed. Among five suggested possibilities of embankment renewal, anchored pile wall at the toe of the embankment was chosen. Elasto-plastic FEM analyses of pile wall showed that the implemented construction sequence resulted in a lower pile wall loading.

RESUME: En cours de réalisation d'un remblai routier de 10 m de hauteur, traversant sur 100 m de long une vallée peu profonde, une fissure de traction parallèle à l'axe et longue de 60 m est apparue, bien que les problèmes de stabilité n'aient pas été attendus d'après la reconnaissance préalable du tracé. L'examen ultérieur a montré que les conditions des sols de fondation étaient beaucoup moins favorables que prévu. La méthode des éléments finis ainsi que le calcul classique ont permis de découvrir un des mécanismes de rupture possibles, tandis que les mesures effectuées préalablement avec les inclinomètres ne nous ont malheureusement pas fourni ces résultats. Un examen approfondi, réalisé sur place et au laboratoire, ainsi que les calculs numériques ont permis de proposer plusieurs interventions possibles pour améliorer la stabilité d'ouvrage. Parmi les cinq propositions a été choisie celle d'un mur de soutènement ancré, qui a été construit au pied du talus. L'analyse en éléments finis a démontré que de cet ordre des travaux résulterait un chargement du mur de soutènement moins important.

1 INTRODUCTION

A relatively high embankment (7 m in the axis and 10 m at the edge of the motorway) on a new motorway section, constructed between 1994 and 1996, had undergone large deformations before its completion. A 60 m long fissure appeared on the embankment surface. In a culvert incorporated in the embankment a 10 to 15 cm large displacement was observed. The embankment is approximately 100 m long and is shown on Fig. 1 together with isohypses of the surface before the construction. The inclination of the natural slope was 8°.

During the design of this 11 km long motorway section in the north-eastern Slovenia not enough attention was paid to bad subsoil conditions. Only one bore-hole (marked as AC-42) was made in the embankment area. Based on the results of this boring it was foreseen that a 4.5 m thick layer of medium to stiff highly plastic clay is underlain by poorly cemented sandstone with unconfined compression strength q_u > 400 kPa. Taking into account these stratigraphical data and available mechanical properties of clayey layer, such embankment behaviour could not have been predicted.

2 ADDITIONAL IN-SITU TESTING

The earthworks on the embankment started in Autumn of 1994. The forthcoming winter forced the contractor to fill the embankment quickly. When the snow fell, the embankment had not yet been completed. Only the remaining one or two meters had to be filled in the Spring. After the thaw in the Spring of 1995 a fissure was observed, as indicated in Fig. 2. Half of the embankment was damaged. Additional five bore-holes (V-1 to V-5) were made on the slope in front of the embankment (Fig. 2). These borings showed that the clayey layer was much thicker than had been believed.

The laboratory test results showed that clays in the upper layer are predominantly stiff and highly plastic with undrained shear strength c_u > 60 kPa. A few samples showed that soft or medium clays with c_u between 30 and 40 kPa were also present in the

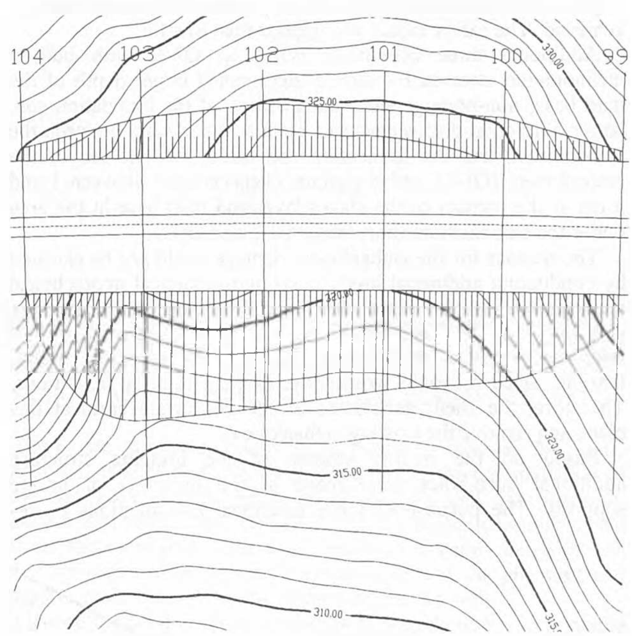

Figure 1. Designed embankment and the original surface.

subsoil. However, the depth and thicknesses of these softer clays were different in each bore-hole. It was impossible to construct a characteristic soil profile by taking into account some softer sub-layers and using them in numerical models. The level of underground water was measured in bore-holes V-1 to V-5. The water table was found almost at the surface.

Taking into account these new data and the most pessimistic undrained shear strength of the upper clay (c_u = 37 kPa), the limit state of the equilibrium was only found in profile 102, but not in profiles 101 and 100, where the fissure was observed.

The finite element analyses using Plaxis programme were also performed. Using the most pessimistic shear strength parameters

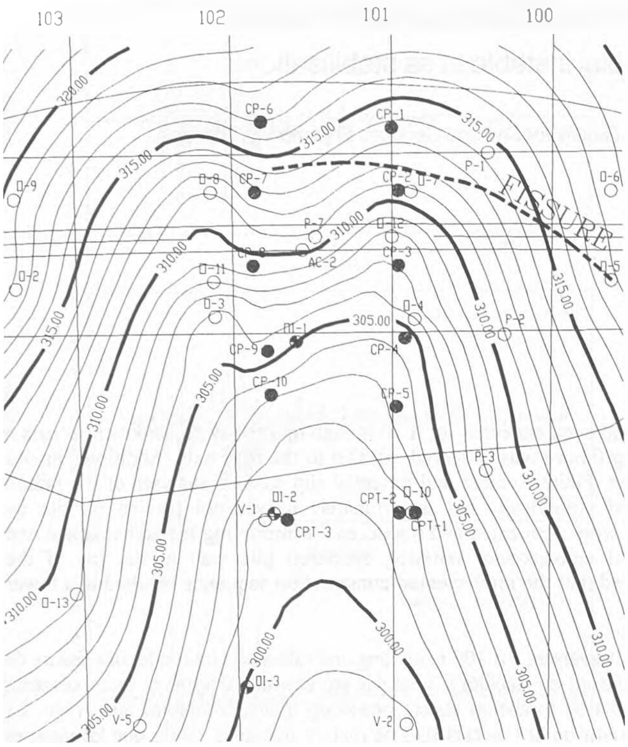

Figure 2. Locations of bore-holes and CPT tests, and the surface of the sandstone/marl base.

for the clayey layer in these analyses resulted in an unstable initial state, whereas higher values of undrained shear strength led to large horizontal deformations (1.3 m), but the limit state was not achieved. The safety factor was greater than SF=1.3.

Additional three bore-holes (OI-1 to OI-3) with built-in inclinometers showed the already determined larger depth of the marl base, non-homogeneous composition of the foundation soil, larger horizontal displacement of 5 cm at the contact between the embankment and the foundation soil under the left toe of the embankment (OI-1), and horizontal displacements between 1 and 2 cm at the contact of the clayey layer and marl base in the area under the embankment (bore-holes OI-2 and OI-3).

The reasons for the embankment damage could not be clarified by conducting additional in-situ tests and numerical geotechnical analyses. The results of the classical stability analyses in all three profiles (100, 101 and 102) showed that the construction of lateral embankment at the toe of the fissured embankment is not feasible, since it would worsen the existing stability conditions. Therefore, the client decided to bridge the terrain basin at this place and remove the existing embankment.

Based on the outline scheme of the bridging structure additional bore-holes were made at the locations of bridge supports. The purpose of these additional investigations (bore-holes O-1 to O-13) was to determine the depth of the marl base, appropriate for deep foundation. New bore-holes showed even more explicit basin of marl base (Fig. 2). In the central part of the embankment, between profiles 101 and 102 (Fig. 1 and 2), the established depth of the marl base at the right edge of the embankment was 11 m under the surface of the original terrain, 15 m in the embankment axis and 16 m at the external left edge of the embankment.

Also the new bore-hole profiles and the results of the laboratory tests on selected samples proved non-homogeneous composition of the subsoil. However, these data were not enough to determine the sub-layers of similar strength properties in the foundation soil. Nevertheless, the consideration of the new data on the depth of the marl base and on the depths of low values of undrained shear strength c_u allowed us to deny the results of the stability analyses, carried out by considering only the data from bore-holes V-1 do V-5, OI-1 to OI-3 and AC-42.

The decision to remove the embankment and to bridge the terrain basin was the reason why all previous studies on causes of damages at the embankment were abandoned. Only in the late Autumn of 1995 did the client realise that due to time pressure (the mentioned motorway section should have been opened in the Autumn of 1996), expensive renewal works, loss of transportation lines at the location of the damaged embankment and also because of legislative complications (the project of bridging structure, new building permit, ...) the construction of the bridging structure at the planned area was no longer possible. The client's wish was to keep the already constructed embankment partially or as a whole and to adequately improve the subsoil.

3 STABILITY ANALYSES BASED ON THE RESULTS OF THE CONE PENETRATION TEST (CPT)

In order to give an adequate solution for the renewal of the unstable embankment the position of the sliding surface and adequate values of shear resistance in the foundation soil had to be determined first.

The limit stress state could be determined in all three profiles by stability analyses ($F \cong 1.0$) with varying the "average" value of undrained shear strength $\bar{c}_u$ of the subsoil, if at least approximate distribution of the sliding surface had been known.

Beside the observed fissure there were no other data that would at least approximately define the form of the slip surface, its depth and length (measured displacements in inclinometers were not persuasive, monitoring time was too short, the lower edge of the slip surface could not be detected on site, and last but not least, the embankment was in a state of equilibrium throughout the investigation period).

The decision was to test systematically the composition of the subsoil and the depth of the sandstone/marl base with cone penetration test in the cross-sections 100, 101 and 102. The locations of CPT tests are shown in Fig. 2 (marked as CPT-1 to CPT-3, CP-1 to CP-10 and P-1 to P-3).

The tests confirmed the form of the basin of the sandstone/marl base in the transverse and longitudinal direction under the

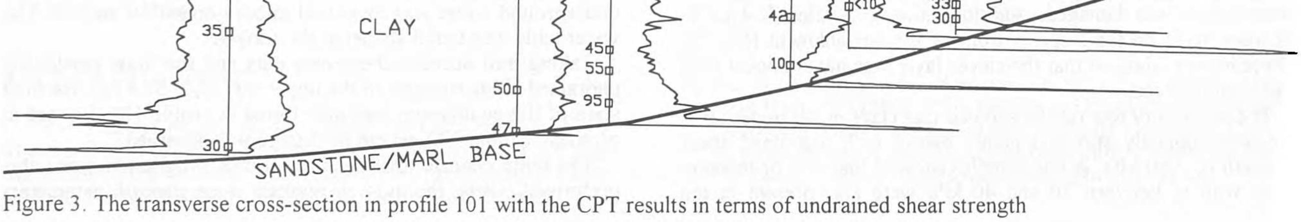

Figure 3. The transverse cross-section in profile 101 with the CPT results in terms of undrained shear strength

embankment that was previously determined by bore-holes. The measured distribution of undrained shear strength with depth by CPT showed the areas of the lowest values of undrained shear strength c_u. On average the lowest values of undrained shear strength were found to be 45 kPa with local minima, that were even smaller than 10 kPa. The CPT results are presented in profile 101 in Fig. 3.

The first step was to determine the lines of the same values of undrained shear strength c_u with adequate numerical procedure. Fig. 4a shows the areas of the same shear strengths in the subsoil in profile 101. The performed potential slip surface analyses did not give the desired result ($F \cong 1$).

The limit equilibrium state in all three profiles was found only when the polygonal slip surfaces through points with minimum c_u values (measured by CPT), were considered. There were several potential slip surfaces with safety factor close to SF=1. Fig. 4b shows the critical slip surface in the profile 101, which corresponds also to the observed displacements in inclinometers.

In each profile the average value of undrained shear strength ($\bar{c}_u$) exhibiting failure in potential slip surface analyses was also sought. In the profile 101 the state of limit equilibrium was demonstrated at $\bar{c}_u$ = 41 kPa (Fig. 4c), and in the profile 102 at a slightly higher value $\bar{c}_u$ = 44 kPa.

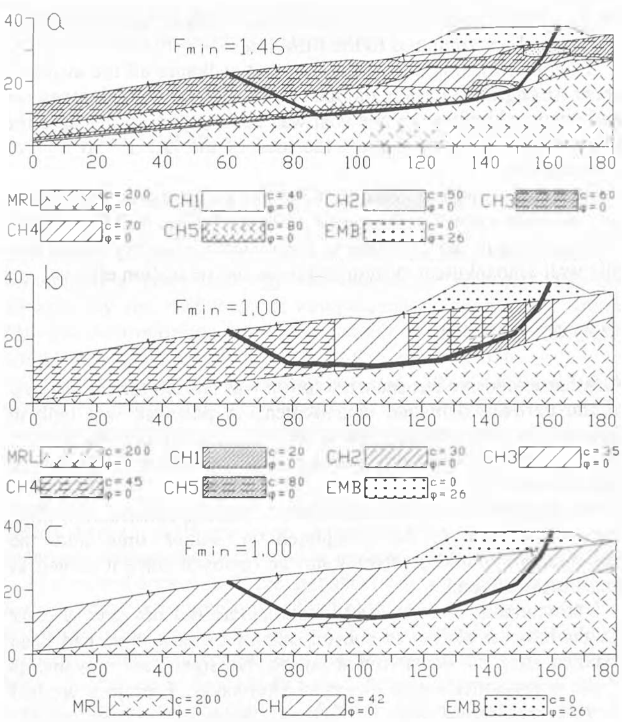

Figure 4. Numerical models for potential slip surface analyses

4 THE RENEWAL OF THE DAMAGED EMBANKMENT

Based on back-analyses of the current limit state at the road embankment it was concluded that deep polygonal slip surface which runs mainly along the contact of the clayey layer and sandstone/marl base is critical.

The elevation of the embankment to the designed height without additional measures would result in considerable worsening of the stability conditions. Based on the results of the stability analyses, and by taking into account the available technology of the contractor and a short unfavourable winter time (Autumn of '95 - Spring of '96) for the renewal of the current state, the client was offered four possible solutions:

1. Removal of the existing embankment, execution of gravel columns at the toe of the embankment and construction of a new, lighter embankment (several layers of 30 cm of light fly ash and 10 cm of gravel). At the bottom the columns would be bored 1 m deep in the sandstone/marl base and the lowest 2 m would be filled with concrete. The concrete would prevent the penetration of water into the marl base what could cause weathering of soft rock, and would also act as a dowel.
2. The removal of the existing embankment, installation of gravel columns under the embankment (with concrete part in the marl base) and the construction of a new embankment made of local material (marl).
3. Reinforced concrete piles (dowels) at the left edge of the embankment. The piles would be approximately 10 m long with the embedment length into the marl base of 5 m, and 5 m length in the clayey layer above the base. The rest of the excavation, between the concrete and the surface, would be filled with gravel.
4. The anchored pile wall along the edge of the embankment.

In order to prevent the seepage of ground water from the background into the subsoil beneath the embankment also a deep drain trench at the right edge of the embankment and along the cross-sections at the starting point (profile 99) and end point of the embankment (profile 104) was suggested. The deep drain trench would consist of closely spaced gravel columns. The gravel columns would reach at least 0.5 m deep into the marl base and at the bottom of the piles the longitudinal drain pipe would also be installed. In order to drain the water from the drain trench, transverse bored drains were designed from the lowest points of the longitudinal drain trench.

The client decided for the proposed deep drain trench, and the embankment should be supported by an anchored pile wall (Fig. 5). The pile wall was made from reinforced-concrete piles with a diameter of 150 cm and permanent soil anchors were used. Bored longitudinal drains at the bottom of gravel piles, and the transverse outlet pipes were installed by controlled horizontal drilling using the trenchless technology.

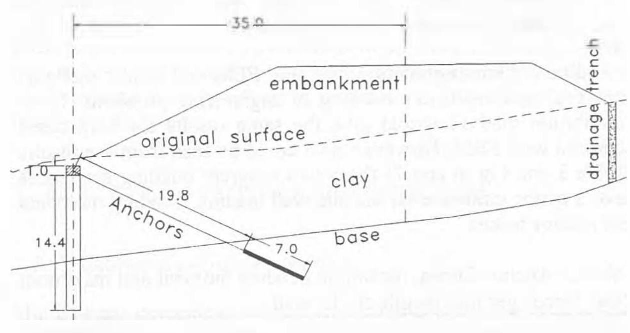

Figure 5. Typical cross-section of embankment after its renewal

5 GEOSTATICAL ANALYSIS OF ANCHORED PILE WALL

There are different numerical procedures known for static analyses of anchored pile walls. They enable the determination of the necessary embedment depth of the pile wall, the necessary anchor forces and the magnitude of the bending moments and shear forces in piles. These procedures are mainly based on the determination of active earth pressures behind the pile wall and passive resistance of the soil in front of the pile wall.

Since the displacement magnitude of the pile wall is in principle unknown in advance, the calculation of the adequate magnitude of the earth pressures on the retaining structure is also questionable. Usually, the earth pressures are calculated as a product of the effective overburden pressures and adequate earth pressure coefficients (K_A for active pressures and K_P for passive pressures). By taking into account the adequate safety factors, either by reducing the strength parameters or by multiplying active earth pressures and reducing passive earth pressures by a certain factor, the influence of unknown deformations on the mobilisation of shear strength is taken into account.

Such procedures are confirmed in the practice and therefore frequently used, but they are limited to simple cases where the expected stress changes in the surrounding soil are such that the described way of calculating earth pressures is justified. Normally the retaining structure is constructed first, then the excavation in front of it is carried out up to the designed depth, or an embankment is built above the retaining structure.

The static analysis of the pile wall at the toe of the damaged embankment was performed by limit equilibrium method and by the finite element method. Soil properties, reinforced concrete pile and anchor characteristics used in numerical calculations are given in the table 1 below.

Table 1. Design values of material properties

	SOIL LAYERS		
	embankment	clay	marl/sandstone
γ (kN/m^3)	21.5	20.5	24.0
φ (°)	26.0	-	-
c (kPa)	0.0	41.0	300.0
G (kPa)	15000.0	1300.0	7000.0
ν	0.3	0.4	0.3
PILE WALL			
ϕ (cm)	A (m^2)	I (m^4)	E (GPa)
150.0	1.767	0.249	30.0
ANCHOR			
F_{ult} (kN)	A (cm^2)	E (GPa)	α (°)
1000	6.158	210.0	25

In the sequel the results of the following analyses are presented:

L1 limit equilibrium method: active earth pressures behind the wall is calculated using mobilised shear strength parameters with safety factor SF=1.3. Passive earth pressure in front of the pile wall is calculated with safety factors SF=1.5 ($p^*_p=p_p/1.5$)

FEM1 finite element analysis simulating the installation of pile wall before the embankment is constructed

FEM2 finite element analysis of actual case: the embankment is nearly finished, then the pile wall is constructed and finally the embankment is completed.

Unlike the limit equilibrium method, FEM and similar methods enable a more realistic modelling of engineering problems. Limit equilibrium method would give the same results for both cases acquired with FEM. However, as it could be seen from the results (Table 2 and Fig. 6 and 7) these two different building sequences have a major influence on the pile wall loading, bending moments and anchor forces.

Table 2: Anchor forces, maximum bending moment and maximum shear forces per unit length of the wall

Analysis	F_{AH} (kN/m)	M_{max} (kNm/m)	Q_{max} (kN/m)
L1	573	4600	1260
FEM1	788	4160	939
FEM2	320	1560	390

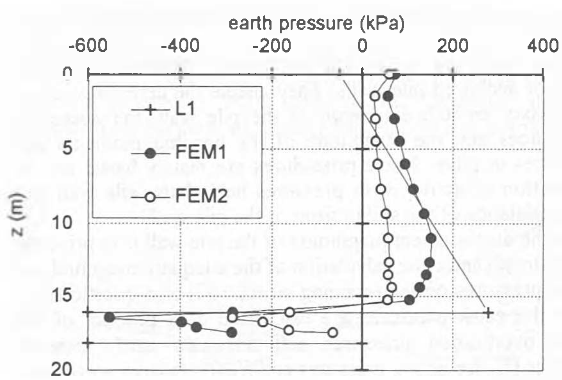

Figure 6. Calculated difference between passive earth pressures in front of the wall and active earth pressures behind the wall

The results of case L1 are similar to those of the FEM1 case. Thus the safety factors taken into account comply well with the case in which the pile wall is constructed before the embankment is filled. A rather big reduction of passive pressures is justified by high stiffness of the wall-anchor system which prevents deformations of the soil in front of the wall and thus the mobilisation of shear strength, which causes the increase in passive resistance.

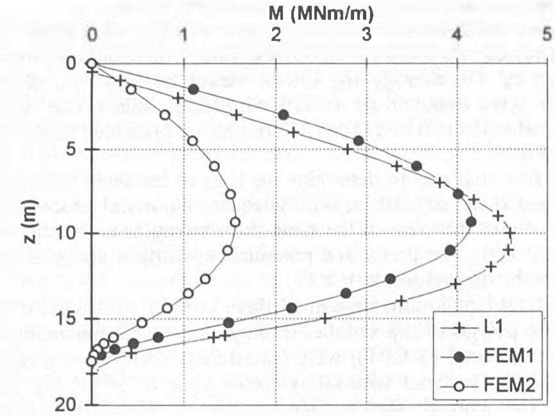

Figure 7. Calculated bending moments in the pile wall

The FEM2 case models the construction sequence as it occurred on site. One could observe that the earth pressures (Fig. 6), bending moments (Fig. 7) and shear forces are significantly reduced when compared to the FEM1 case.

This difference is due to the fact that at failure all the available shear strength had been mobilised and big horizontal pressures had occurred before the pile wall was constructed. In this case the pile wall had only to support the load of the last 1.5 to 2 m of embankment.

In actual design somewhat higher earth pressures were taken into account compared to those obtained by the FEM2 analysis. This was due to our intention to assure a proper safety of the soil-pile wall-embankment system and expected relaxation effects.

6 CONCLUSIONS

A rather expensive but secure solution was chosen for the renewal of the partially damaged embankment. A pile wall was built in front of the embankment and a deep drainage trench made from closely spaced gravel columns was installed behind the embankment.

The reasons for such decision were: limited construction time, the works had to be completed in winter time and the embankment should preferably not be removed since it served as transportation road.

Unfortunately the installed inclinometers did not give a clear insight into the sliding of subsoil, mainly because they had been installed after the embankment failure. No significant movements of the embankment were observed afterwards. Therefore we had to rely on boring and CPT data. In order to get reliable inclinometer readings some additional load should have been put on the embankment, but the risk to extend the embankment damage seemed to high.

Another hypothesis that was seriously studied but not presented in this paper is that failure had been caused by high artesian water pressure at the bottom of the clayey layer. The sandstone under the clay is permeable and the thaw in the Spring of 1995 could have caused a short-term build up in water pressures.

After all works had been finished it was observed that drainage system collected a lot of underground water, which would otherwise penetrate beneath the embankment and wet the contact between sandstone/marl base and clayey layer.

CPT results formed the main and reliable source of material data for all numerical analyses. Using FEM and potential slip surface analyses back-calculations of embankment failure were made. For the design of pile wall limit state analysis and FEM analysis were carried out. A considerable difference in pile loading was found in two different cases: (1) pile wall is constructed before the embankment is made and (2) pile wall is constructed when embankment is made up to approximately 80% of its final height. In the second case bending moments and shear forces in the piles as well as anchor forces are much smaller.