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Stability analysis and rehabilitation measures of landslide Rebernice

Analyse de stabilité et mesures de réhabilitation du glissement de terrain dans la zone de Rebernice

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ABSTRACT
At the alignment of the highway Razdrto-Italian border, at the section Razdrto-Vipava, in November 2001 a slope failure occurred in the Rebernice area during the excavation of a deep cut. With surveying and geologic geotechnical investigations it was found out that, a widely extended landslide occurred with the total volume of the slid mass about 600,000 m^3. The initial data, the data obtained during additional geotechnical investigations, monitoring data and soil and rock properties obtained from laboratory tests formed the basis of a geological-geotechnical model for stability analysis. The verified geological geotechnical model was used for the design of the stabilization measures. An anchored pile wall with an additional anchored grid beam structure was proposed for the stabilization of the landslide.

RÉSUMÉ
Lors de la construction de l'autoroute entre entre Razdrto et la frontière italienne, un glissement de terrain s'est produit pendant l'excavation d'une grande tranchée en novembre 2001. Les études et investigations géologiques–géotechniques ont montré qu'il s'agissait d'un grand glissement impliquant un volume de terrain de 600.000 m^3. Les paramètres du modèle géologique-géotechnique pour l'analyse de stabilité proviennent des données initiales, des investigations complémentaires, de l'instrumentation et des propriétés du terrain obtenues par des essais de laboratoire. Le modèle a été calé sur le glissement, puis utilisé pour dimensionner la confortation du glissement. Un rideau de pieux ancré à deux niveaux et un rideau similaire seulement ancré en tête mais avec une partie de talus multiancré ont été proposés pour stabiliser le glissement.

1 INTRODUCTION

At the alignment of the motorway Razdrto-Italian border, at the section Razdrto-Vipava, in November 2001 a slope failure occurred in the Rebernice area during the excavation of a deep cut. At that period the greater part of the 26 m deep excavation had already been carried out. The bottom of the motorway cut was approx. 6 m above the final design level when the failure cracks were observed on the right slope far beyond the upper cut edge just after heavy rainfall in November 2001. The designed slope inclination was 1:2 (H:L).

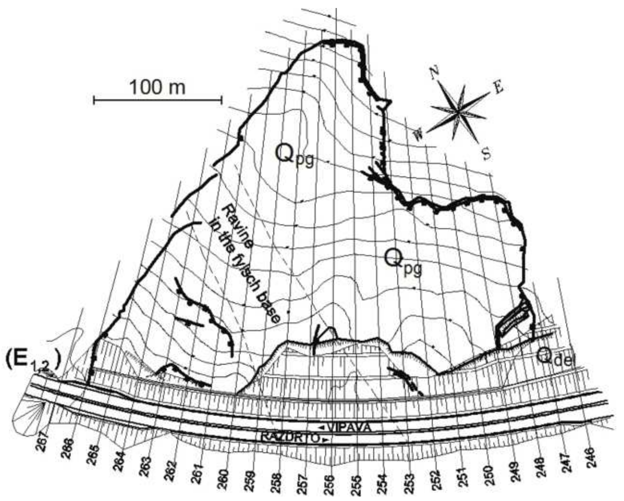

Figure 1. Situation of the landslide

The landslide, clearly marked by wide and deep failure cracks, reached a width of 360 m, while its length varied from 180 m between the cross-sections P247 and P253 and up to 310 m between the cross sections P255 and P258 (Fig. 1). The volume of the sliding mass was estimated to about 600,000 m^3 and it slid at a rate of 1 cm/day or more. It is the largest landslide caused by the earthworks in the framework of the motorway construction in Slovenia.

2 GEOTECHNICAL INVESTIGATIONS

The preliminary and project investigations could not have predicted the occurrence of the slide. Thus, after the landslide was triggered, a detailed additional investigation followed to form a complete geological-geotechnical model of the landslide and to find the final solution for the motorway cut. Geodetic measurements were set up immediately to record the rate and the direction of the landslide movements. In spring 2002 additional 8 trial pits were excavated and 18 boreholes were drilled. 16 boreholes were equipped with inclinometer casings, 10 of them were perforated to enable the measurements of groundwater level. In two boreholes a pair of BAT pore pressure cells was installed at two different levels. Samples were taken from characteristic soil layers and rock types out of bore cores and were investigated in the laboratory.

Observations, monitoring and interpretation of the gathered data disclosed a deep-seated transitional slide. The inclination measurements in the central part of the landslide showed the depth of the failure plane between 11 and 13 m. The slip surface followed the 2 to 3 m thick heterogeneous layer consisting of plastic clay and silt with varying contents of sand and gravel between the underlying flysch and the overburden of highly-permeable limestone debris. This intermediate layer originates from both materials, limestone and flysch.

3 GEOLOGICAL CONDITIONS

The location is geologically dominated by the overthrust of Cretaceous limestone formation over the Eocene flysch $E_{1,2}$, which comprises alternating, variously thick beds of marly claystone, siltstone, sandstone and marly calcarenite. Due to tectonic effects and water seepage, a rock base is weathered up to several meters in depth. Stratification is still noticed in the weathered rock formation [$E_{1,2}$].

Weathered flysch [$E_{1,2}$] is overlain by the layer of flysch debris Q_{del}. The grains are highly weathered and the clayey matrix is stiff to very stiff with low permeability.

The layer of flysch debris Q_{del} is overlain by the deposit of up to 12 m thick limestone debris (scree) Q_{pg}, which is locally cemented in breccia. Gravel is coarsely grained, with the average diameter of particles of 10 cm and with sandy to silty matrix. The stage of karstification is diverse and the permeability is high. Seepage waters rarely occur in this layer, while they frequently appear along the contact with the lower layer.

Between limestone debris Q_{pg} and flysch debris Q_{del} there lies a 2 to 3 m thick intermediate layer Q_{del}-Q_{pg}. It is formed partly as a mixture of gravel with previously present flysch material and partly by the clayey particles as a product of the chemical weathering of lime and rainfall induced transport from the upper layer. The layer is heterogeneous with more than 50% of sand and gravel particles in plastic clayey and silty matrix. The clay of high plasticity as soil matrix is the weakest material from the geotechnical point of view. The interpreted ground conditions are presented on the cross section in Figure 2.

Due to the large limestone aquifer within the Nanos Karst plateau and because of impervious flysch bedrock an inflow of water in relatively permeable ground formations is present. Ground water was recorded in different levels. In all boreholes drilled on the landslide, intensive water inflows were recorded on the base of the limestone debris (scree) Q_{pg} and in the intermediate layer Q_{del}-Q_{pg}. Since the underlying layer Q_{del} is relatively impervious, water seeps predominantly along the intermediate layer. Periodical pore pressure measurements in this layer showed up to 20 kPa of excess pore pressure as a consequence of intensive precipitation and steep background. Another important finding was that modest precipitations do not influence essentially the water table in the boreholes.

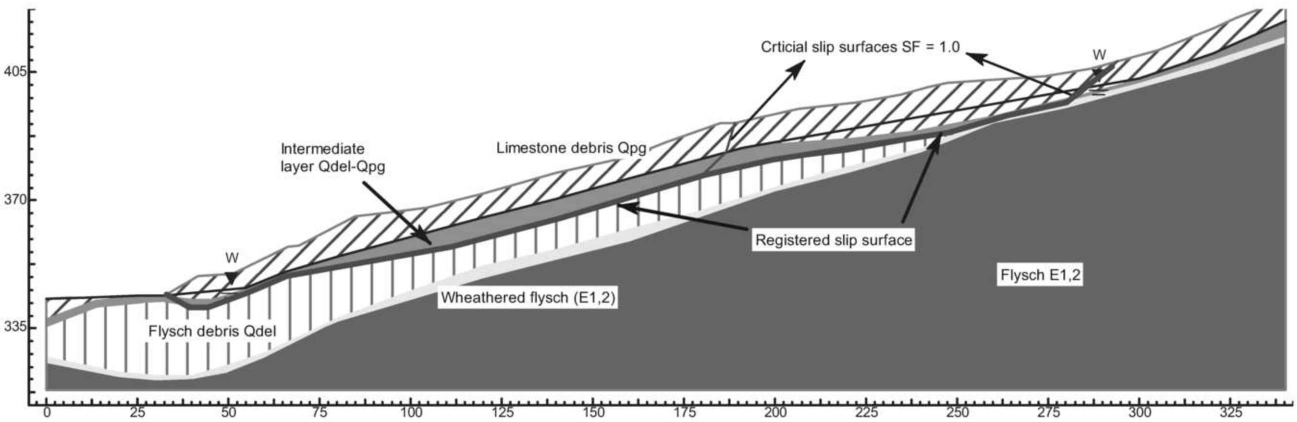

Figure 2. Cross section P257 of the landslide with intermediate soil layer formed of clayey gravel and flysch debris

4 MONITORING AND TEMPORARY MEASURES

Soon after the landslide had occurred, a geodetic survey was established. 24 pairs of geodetic points were put on the tension cracks for the measurement of crack width increase and 25 geodetic points for 3D measurements of displacements were put on and around the landslide area. According to the measurements from the end of December 2001 to the middle of January 2002 the eastern part of the landslide moved by 11 to 20 cm and western part by 16 to 30 cm. Movements of the sliding mass developed at an approximate rate of 1 cm/day. In the spring 2002 the total displacements of the landslide reached about 2.0 m.

Figure 3. Toe of the landslide after backfilling

In order to temporarily stop the movements of the landslide it was decided to partly backfill the excavated volume using stone material with high shear strength (Figure 3). The minimum amount of the backfill material necessary to reduce the movements was determined by the stability calculation.

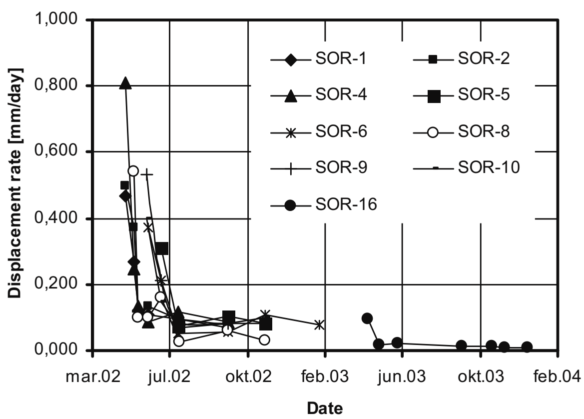

Figure 4. Landslide displacement rate vs. time (inclinometer readings)

During the backfilling of the cut in April and the first half of May 2002 the displacement rate of the sliding mass was greatly reduced to about 0.2 mm/day and this was also the period when additional investigations were performed and inclinometers were installed. The inclinometer measurements indicated a well defined slip surface and a uniform translational movement of the sliding mass. Most of the inclinometer measurements had to be abandoned when those situated within the landslide area were sheared off at the end of 2002 or beginning of 2003. Since then geodetic measurements have provided data on the ongoing slope movement. Since July 2002, when the temporary remedial measures were completed, a linear decrease of the rate of sliding can be observed from both inclinometer (Fig. 4) and geodetic readings. The inclinometer and the geodetic measurements performed in 2003 and 2004 indicated further movements of the sliding mass with the reduced sliding rate about 0.01 mm/day, clearly showing the beneficial effect of the backfill.

5 SOIL PROPERTIES

The samples taken from boreholes were subjected to laboratory investigations. Due to the nature of the problem the laboratory tests were mainly focused on the properties of the intermediate layer Q_{del}-Q_{pg}. It was soon recognized that due to the heterogeneity of the material in this layer, the soil properties varied considerably.

The grain size distribution (Fig. 5) shows that the soil in the intermediate layer had more than 40% of gravel particles and up to 30% fines (< 0.06 mm). The soil was found to be almost completely saturated with water with the natural water content between 15 and 25%. This fines were classified as clays of intermediate or high plasticity (40% < LL < 59%, 26 < PI < 42). The amount of fines in this particular layer is high enough to have dominating influence on the mixed soil. These findings were also confirmed by the direct shear and triaxial tests (CU-tests) performed on clayey soil samples. The angle of internal friction of the soil was found to be between 20° and 30° with residual values around or below 18° for fines alone.

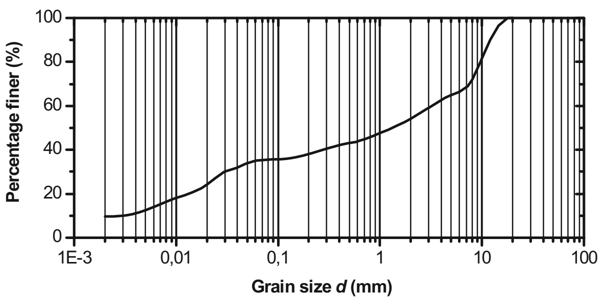

Figure 5. Grain size distribution of the soil from intermediate layer

The geotechnical properties of soils and rocks are presented in Table 1.

Table 1. Geotechnical properties of soils/rocks

Rock/Soil	Unit weight (kN/m^3)	Modulus E_{50}^{ref} (MPa)	Cohesion c' (kPa)	Shear angle φ' (°)
Eocene flysch $E_{1,2}$	25.0	100	50.0	40.0
Weath. flysch [$E_{1,2}$]	23.0	80	25.0	35.0
Flysch debris Q_{del}	22.0	60	0.0	30.0
Inter. layer Q_{del}-Q_{pg}	21.0	15	0.0	18.0/20.0
Limestone. debris Q_{pg}	22.0	45	0.0	38.0

6 STABILITY ANALYSIS

The stability of the motorway cut was initially considered during the design period of the motorway. The negative effect of intermediate layer on the stability of the cut was overlooked due to large amount of gravel particles (> 40%) and medium density of the soil. The intermediate layer was initially considered as geotechnically non-problematic and only additional field investigations and laboratory tests on soil samples taken from the sliding zone showed the opposite.

First the limit equilibrium analyses were performed in order to verify the proposed geological-geotechnical model (Fig. 3). The friction angle of the intermediate layer was of special interest, therefore sensitivity analysis were made for cross sections P257 and P261. The friction angle of intermediate layer Q_{del}-Q_{pg} was varied in the range $\varphi = 18° \pm 6°$ to get its effect on the calculated safety factor. The effect of friction angle φ of the intermediate layer on the factor of safety for the cross section P257 is shown in Figure 6.

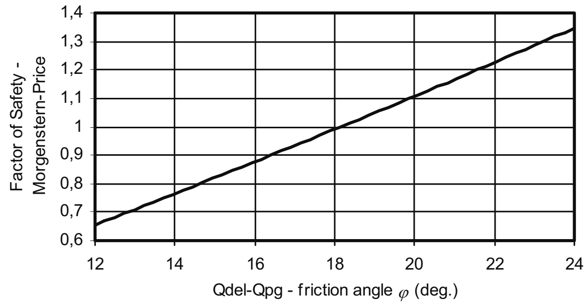

Figure 6: Sensitivity analysis results (cross section P257)

Once the geotechnical model was confirmed by back analyses, the finite element code PLAXIS was used to analyze the stability of the landslide in five representative cross sections. For the mathematical modelling of soils the "Hardening Soil Model" was used. Finite element analyses were conducted for three different stages; the initial state before excavation, the state at landslide triggering (Fig. 7) and the state after backfilling (Table 2). For assessing the stability PLAXIS uses a so-called "*phi-c"* reduction procedure. In this procedure the strength parameters $\tan\varphi$ and c are reduced simultaneously up to point where the system fails numerically (Fig. 8). The final value of safety factor *SF* is defined as

$$SF = \frac{\tan\varphi_{input}}{\tan\varphi_{reduced}} = \frac{c_{input}}{c_{reduced}} \qquad (1)$$

This approach is similar to conventional methods and usually yields the failure mechanism with the minimum safety factor.

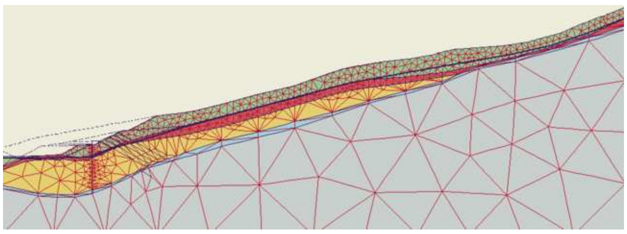

Figure 7. Finite element mesh – cross section P257

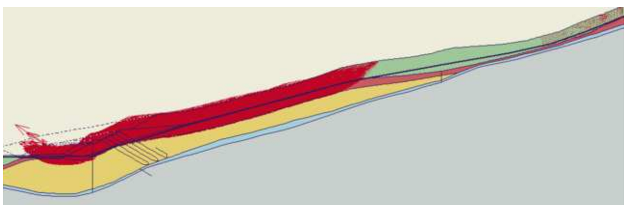

Figure 8. Displacements vectors due to "*phic-c*" reduction (SF = 1.02)

The finite element analyses clearly indicate a stable situation before excavation (safety factor SF = 1.10 to 1.52) and an unstable situation after excavation (SF ≈ 1.0 - limit equilibrium). Safety factors for the stage after backfilling in the range of SF =

1.13 to 1.41 are rather low, so some small movements are still observed in the critical areas of the landslide.

Table 2: Calculated safety factors at different stages and cross-sections

Profile	P251	P257	P261	P263	P265
Initial state	1.36	1.26	1.52	1.10	1.12
Back analysis	1.02	1.02	< 1.0	< 1.0	1.10
After backfilling	1.17	1.13	1.41	1.25	1.13
Final rehabilitation	1.35	1.28	1.42	> 1.35	1.28

With the pore pressure distribution registered on site and using the shear strength parameters obtained in laboratory and in-situ, the performed numerical analyses fully confirmed the sliding mechanism observed on site.

7 FINAL REHABILTATON MASURES

Several possibilities were considered for the final rehabilitation of the landslide. The tunnel constructed with "cut and cover" method, reinforced concrete dowels of large diameter and anchored pile walls were initially considered for the final rehabilitation of the landslide. Finally, an anchored pile wall with additional anchored grid-beam structure and two deep trench drains were accepted by the state road authorities for the final stabilisation of the landslide.

For the design of the supporting structures finite element analysis was employed. All the necessary construction steps were considered in the analysis. The stability of the final state was checked by "*phi-c*" reduction procedure to achieve the required safety factor SF = 1.25 (Table 2). The design of structural members was done according to Eurocode standards.

7.1 *Pile wall*

The pile wall in a length of 347 m between the cross-profiles P248 and P266 will be located at a distance of 29.5 m from the road axis. The slope between the road and pile wall will adopt an inclination of 1:2. The construction of bored piles ϕ118 cm in lengths from 10 to 26 m at spacing of 1.50 and 1.75 m, respectively, is foreseen. The spacing of 1.75 m is proposed between the profiles P248 and P252, and at the end between the profiles P265 in P266. At the top, a cap beam will link the piles. Between the profiles P248 and P258+10m and between the profiles P264+4.75m and P266, the structure will be anchored in one level (Fig. 9), and between the profiles P258+10m and P264+4.75m in two levels (Fig. 10).

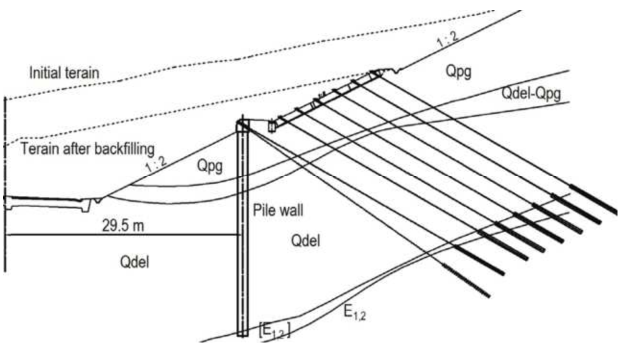

Figure 9. Cross section P257

The distance between permanent geotechnical anchors (5x0.6", f_{py}/f_{pu} 1570/1770 MPa) in the upper cap beam of 1.25 m is foreseen. The adopted average anchors inclination in the upper cap beam is 30°. The total length up to 38 m, bond length 7 m and lock-off loads between 500 and 625 kN are proposed for the anchors. Between the profiles P258+10m and P264+4.75 m (Fig. 10), the second beam will be placed, to be anchored with identical anchors as the upper beam. Up to three anchors will be placed into the lower beam at a spacing of every 1.50 m. These anchors will have various inclinations and lengths up to 28 m. The anchor lock-off load 500 kN is proposed.

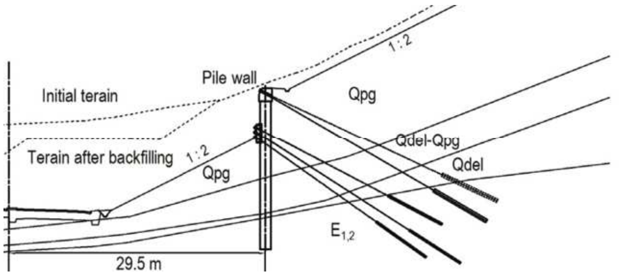

Figure 10. Cross section P261

7.2 *The grid beam structure*

Between the profiles P248 and P258+10m, at the slope inclined 1:2 (H:L) beyond the pile wall, the construction of the anchored reinforced concrete grid beam structure is foreseen (Fig. 9). The grid beam structure comprises "vertical" beams (100x80 cm), running perpendicular to the road axis at the spacing of e = 4.7 m. The "vertical" beams are bond by two "horizontal" beams (80x80 cm), going parallel to the road axis. At the bottom of the structure a bond foundation (80x100 cm), linking "vertical" beams is proposed.

The "vertical" beams will be anchored with permanent geotechnical anchors. Between the profiles P248 and P253, the "vertical" beams will be anchored in four levels with permanent geotechnical anchors (4x0.6″, f_{py}/f_{pu} 1570/1770 MPa). The anchors will have an inclination of 30°, the total length 51 m and the bond length equal to 7.0 m. The anchor lock-off load 400 kN is proposed. Between the profiles P253 and P258+10m the »vertical« beams will be anchored in six levels with permanent geotechnical anchors in a similar way. The anchor lock-off loads 400 kN for the upper two anchors, 450 kN for the middle two anchors and 500 kN for the lower anchors are proposed.

8 CONCLUSIONS

Based on extensive geological, geotechnical in hydro-geological investigations and analyses of the landslide it was concluded that the intermediate layer formed partly as a mixture of gravel with previously present flysch debris and partly by the rainfall induced transport of clayey particles (product of the chemical weathering of lime) from the top of gravel layer to its bottom, increased pore water pressure during heavy rainfall and road construction activities were main reasons for the landslide occurrence. Comprehensive analyses were made using limit equilibrium and finite element method for the verification of the geological model. The performed numerical analyses fully confirmed the sliding mechanism observed on site and served for the final design of rehabilitation works including the construction of anchored pile wall and grid beam structure. So far the performance of the proposed structures, which are still under construction, is according to the design expectations.

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