A SLOPE STABILITY CASE IN ARCADIA: Case Narrative

Highway on the move (EN) – Αυτοκινητόδρομος εν κινήσει (EL)

Note: In the description that follows, actual findings from geotechnical/geological investigations and reports are embedded in a case narrative developed for education purposes; to this end, the narrative involves fictitious characters of project team members and some hypothesized project tasks.

Where are we?

From the Mediterranean region we zoom onto Greece (see accompanying PowerPoint presentation). We are in the prefecture of Arcadia (or Arcady), at the central part of Peloponissos (or Peloponnese) peninsula, where a fertile plateau is surrounded by mountains covered with lush vegetation. In European Renaissance arts, Arcadia was celebrated as an idyllic place of simple, pastoral life.

What is the problem? – Instability of highway earthworks during construction

Things are a little less idyllic in the mid 1990s, time of the construction of a highway going over these mountains, connecting Tripolis, the capital of the prefecture of Arcadia, to Kalamata (as in Kalamata olives…), the capital of the neighboring prefecture of Messinia to the southwest. Problems with embankment instabilities appear soon after construction of earthworks. At about the same time, the management of the project is transferred from the regional level to the ministry of public works in Athens. Due to the change of the original design, which called for a two-lane road, to a four-lane highway, calculations are rechecked for a problematic section of the highway, constructed at an area of colluvial deposits underlain by flysch.

Part I: Stability calculations for representative cross section of earthworks assuming overall stable conditions

As a young engineer in a consulting company working for the ministry, you are asked to do these calculations for a cross section constructed partly on embankment and partly cutting through the colluvial material, as shown in **Figure 1**. Geological mapping covers a zone extending from 150 to 250 m on either side of the road. Geotechnical investigation is focused on problematic areas and in areas where large cuttings or embankments are designed. The geotechnical cross section of Figure 1 is a typical result of such an investigation. The major units are limestone colluvium, about 20- to 35-m thick, and flysch, separated by a zone of clayey weathered flysch.

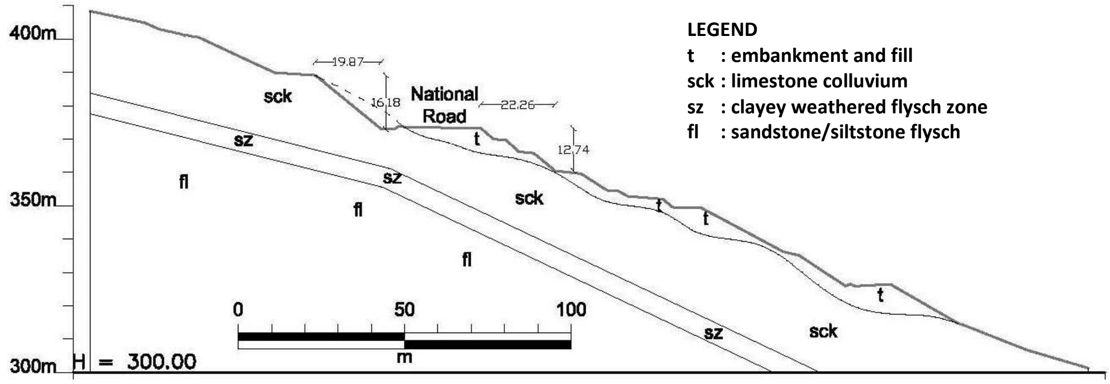

Figure 1: Cross section of the embankment area.

The water table within the in situ material is expected to be well below the embankment area, close to the weathered flysch. Some perched water within the cut slope is dealt with drainage pipes. Unit weights and shear strength parameters for the materials involved are included in **Table 1**. The values for the embankment material are considered reliable. However, the values for the colluvium are approximations resulting from the experience gained at the region during the investigation and construction phases.

Table 1: Material properties for earthwork stability analyses.

Formation	c' (kN/m^2)	ϕ' (°)	γ_{total} (kN/m^3)
Fill material	15	28	20.5
Limestone colluvium	20	30	20.0

As you have never before dealt explicitly with geotechnical analysis of earthworks in any of your geotechnical courses, you consult a manual for geotechnical engineering. In relationship to the typical trapezoidal cross section for an embankment [e.g. Burland et al. (2012): Fig. 70.5], the manual states that you are supposed to check for settlement of the underlying material, as well as for slope stability. You discuss the analysis with your supervisor, who advises you to focus on stability issues (and analyze separately the cut slope, the internal stability of the embankment slope, as well as the overall stability of the embankment-parent foundation material), since settlement is mostly going to be immediate.

Part I - Slope stability analysis results

Calculations show that the cut slope has a factor of safety (FoS) of 1.472, the embankment slope has a factor of safety of 1.992, while the combined embankment/foundation material cross section has a factor of safety of 1.973. Respective values of FoS for dynamic loading are as follows: 1.197, 1.419 and 1.497 (note that the Greek regulations for seismic design stipulate higher seismic acceleration parameters for embankments compared to cut slopes). The critical failure surface for the case of the cut slope is depicted in **Figure 2.**

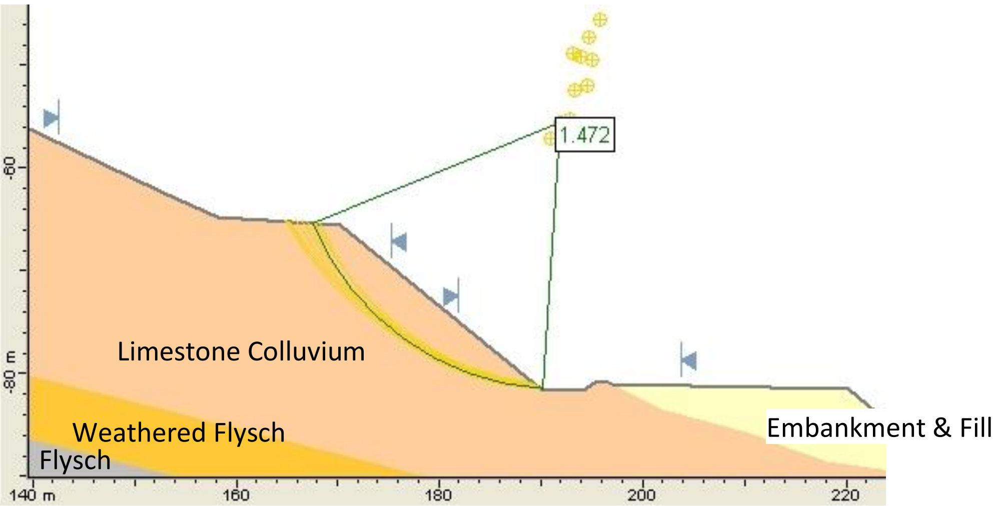

Figure 2: Critical failure surface of the cut slope, long-term conditions (Bishop method).

Since the calculated factors of safety are adequate, your supervisor decides to take your group for a site visit, where you have a chance to see the slope in real life (**Figure 3**). There, the group notices deposited material in addition to the area of the cross section you have checked. A geologist colleague points out for you some of the units you encountered in your calculations (see Figure 3). He also shows you a depression and below it a milder slope in the natural relief of the colluvium underneath the limestone, which could indicate a possible movement in the geological history of the slope.

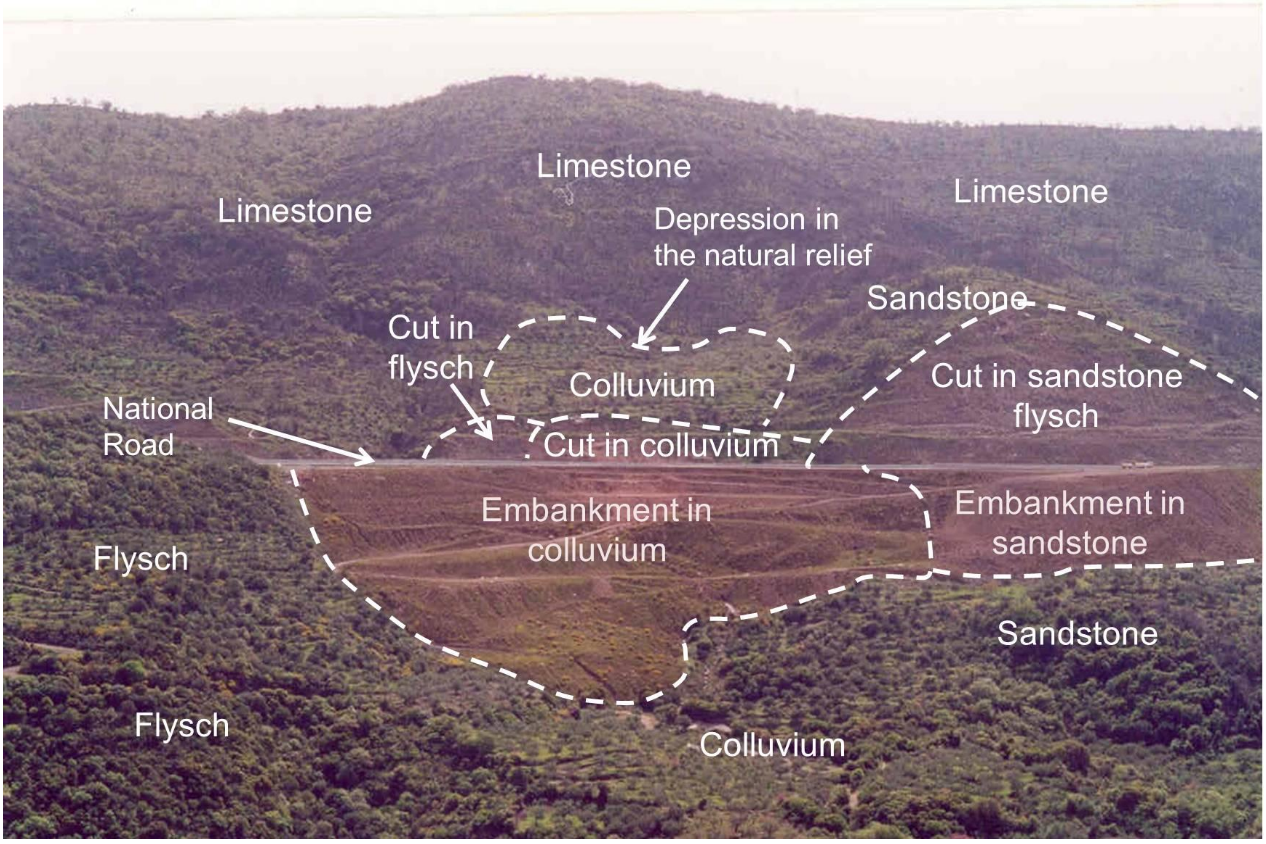

Figure 3: View of construction area (adapted from Dounias et al., 2006).

<u>Highway opens to traffic – problems continue</u>

Construction of pavements was completed in 2000 and the highway opened to traffic. Soon afterwards, cracks, perpendicular to the highway axis, and settlements appeared in the pavement, necessitating paving over with asphalt.

As cracks continued to get larger, albeit at a slow rate (Dounias et al., 2006), the ministry commissioned an in-depth site investigation, which included borehole sampling and logging, in situ and laboratory tests, and recordings of inclinometers, surface monuments and piezometers. The investigation was completed in 2001 and established the existence of a sliding surface 680m long and 200m wide at the highway axis, reaching a maximum width of 370m downslope of the highway (see **Figure 4**). As shown in **Figures 5 and 6**, the main part of the slip surface was located (on the basis of inclinometer readings) within the zone of the weathered flysch (sz), a clayey material of medium to low plasticity, at a variable depth of about 25 to 35m.

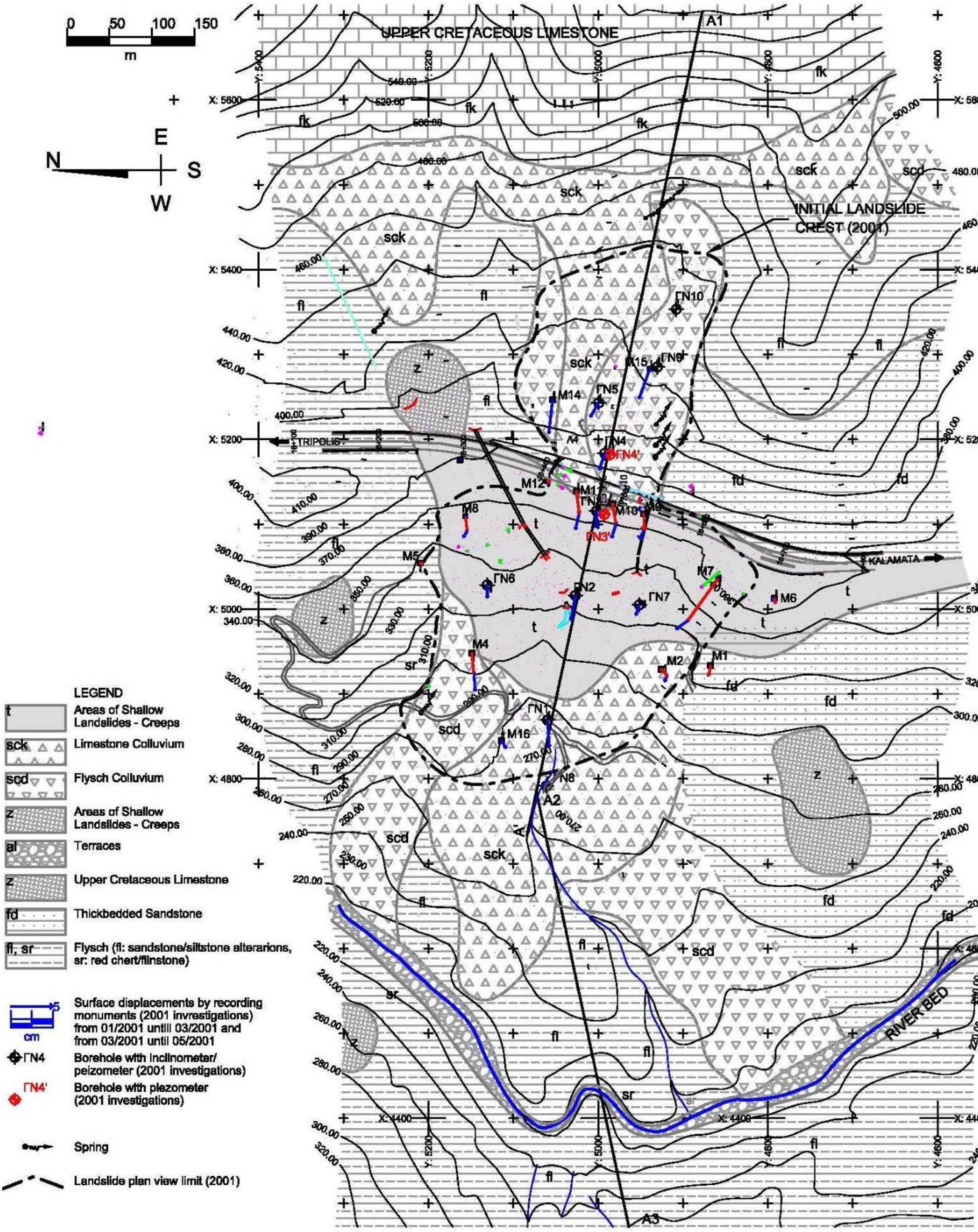

Figure 4: General plan view with the limits of the slip surface in 2001 and the horizontal surface displacements (adapted from Dounias et al., 2006).

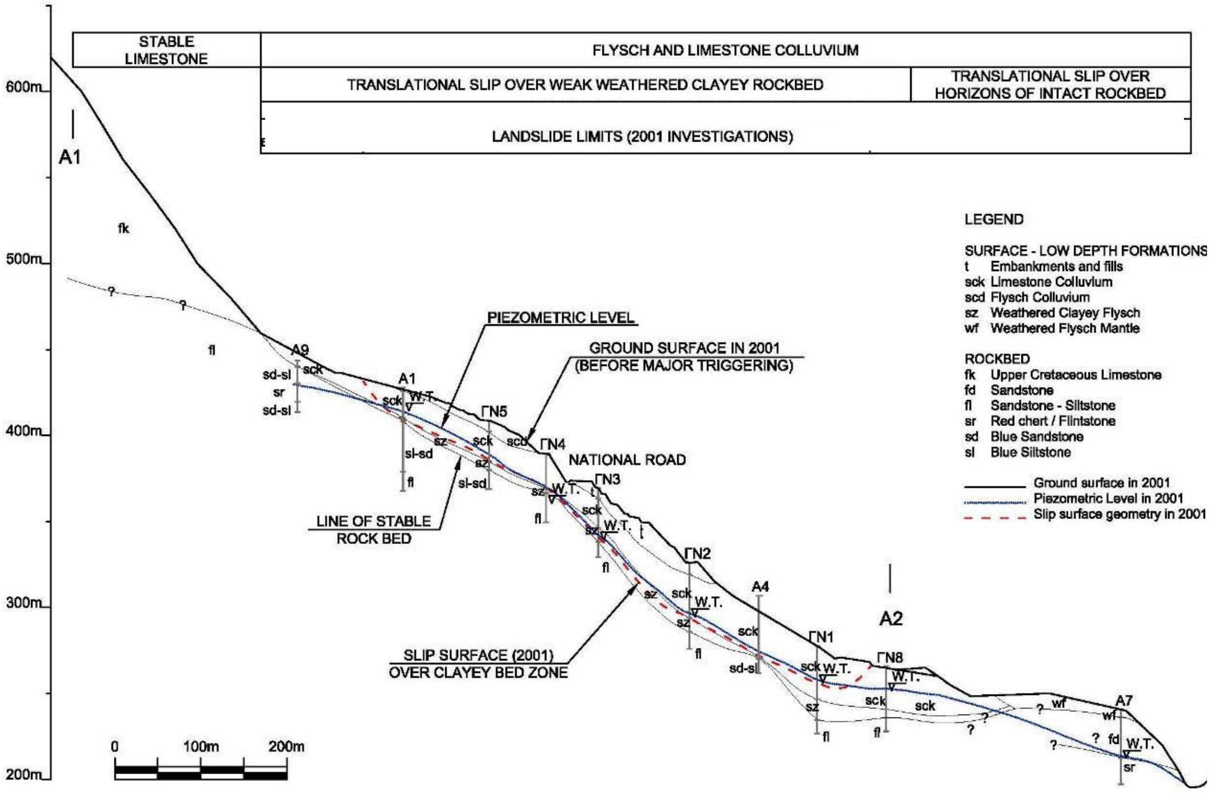

Figure 5: Cross section of the slip surface in 2001 along axis shown in Figure 4 (adapted from Dounias et al., 2006).

Measurements obtained over a period of six months (November 2000-May 2001) gave an average displacement rate of 20cm/year, indicating an active but slow landslide [according to TRB (1996) slides moving at a rate of 1.6mm-1.6m/y are characterized as very slow], which necessitated the evaluation of alternative repair measures. Given the observed movement, the material was likely at a residual state within the slip surface.

Part IIa: Back analysis of cross section of the 2001 slip surface

Back analyses are preformed in order to evaluate the shear strength parameters along the slip surface. Due to the considerable displacement over the aforementioned 6-month period, back analysis for a FoS=1 is expected to give a value of average mobilized shear resistance that corresponds to the average residual strength of the material along the slip surface. Two alternative sliding mechanisms are considered: one single slip surface or two semi-independent slip surfaces, uphill and downhill of the highway, involving areas A1 and A2, respectively, as shown in **Figure 6**. The two-section slip surface is the kinematically plausible sliding mechanism suggested by the geometry of the surface of the flysch bedrock (see Figure 6). As this back analysis is more involved than the one corresponding to the cross section of Figure 1, you are not expected to perform it on your own. A senior geotechnical engineer discusses with you the analysis for A2 and you are asked to do the same for A1. In both cases, the geometry of the slip surface depicted in Figure 6 indicates a translational type of slide instead of a rotational (i.e. circular) one.

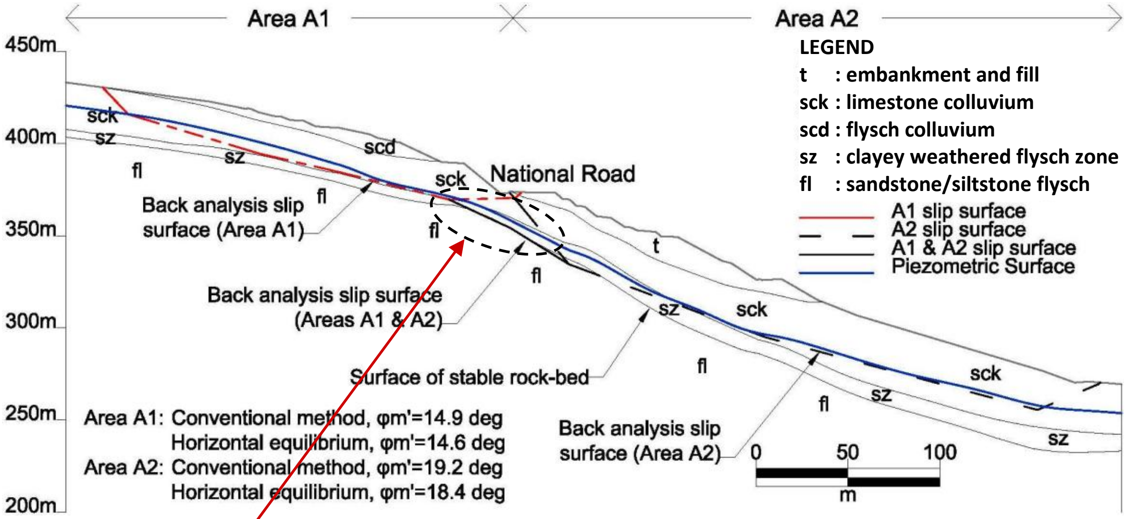

Figure 6: Cross section showing the two-part slip mechanism along the axis of the slip surface. The dashed-line oval shape highlights the hump in the curvature of the intact/weathered flysch that imposes a kinematic constraint on the failure mechanism.

Note that in the initial calculations for the embankment area, peak shear strength parameters were used. In contrast, for the back analysis, the factor of safety (FoS) is set to 1 and the respective value of the mobilized angle of friction ϕ_m' is calculated, assuming zero cohesion, c′. For the non-circular failure surface considered in this case, which resembles an infinite slope, the method of slices was combined with two alternative methods for calculating FoS. Method A is known as the conventional method, whereby FoS is expressed as the ratio of the sum of the resisting shear forces on the base of each slice over the sum of the driving forces of each slice's weight resolved parallel to its base (e.g. Equation 12.19 in Knappett and Craig, 2012). For an infinite slope, method A corresponds to calculating the FoS through equilibrium of forces in the direction parallel to the slope. In method B, FoS is calculated through force equilibrium for the entire slope in the horizontal direction (e.g. Equation 5 in Fredlund et al., 1981).

Part IIa – Results from back analysis

The results from the back analysis of area A2 for FoS=1 give a mobilized angle of friction ϕ_m' equal to 19.2° and 18.4°, with methods A and B, respectively.

You now have to perform the same analyses for area A1 and to back calculate the mobilized strength. You should get values close to ϕ_m' = 14.9° and 14.6°, with methods A and B, respectively. These values will be used to evaluate the feasibility of repair measures, which include excavation (Fig. 7), a grid of stabilizing piles, and anchored retaining walls (Dounias and Belokas, 2010).

Residual strength measurements on soil samples, obtained with the reversal direct shear technique, gave a comparable range for the residual angle of friction ϕ_r' = 16° to 20°. Moreover, samples of this material gave Atterberg Limits of about PL=15% and LL=35%. According to Lupini et al. (1981) and Stark et al. (2005), the values determined for the weathered flysch correspond to the low end of possible values for residual strength, for the measured Atterberg Limits.

Part IIb –Feasibility analysis of excavation as a repair alternative

Your final task for the project is to help the senior geotechnical engineer of the team with the analysis for the repair option with excavation, for sliding area A1 (**Figure 7**). Excavation as a repair alternative, in general, aims to relieve the slope from some weight, mainly at the upper part of the sliding area, thereby increasing the overall stability (i.e. FoS) of the slope. In this case, however, the geometry resembles that of an infinite slope, for which FoS does not have a strong dependence on the thickness of the sliding mass. Nevertheless, since the average surface slope inclination and the inclination between berms in Figure 7 are milder than the inclination of the initial A1 area in Figure 6, the new geometry could be stable.

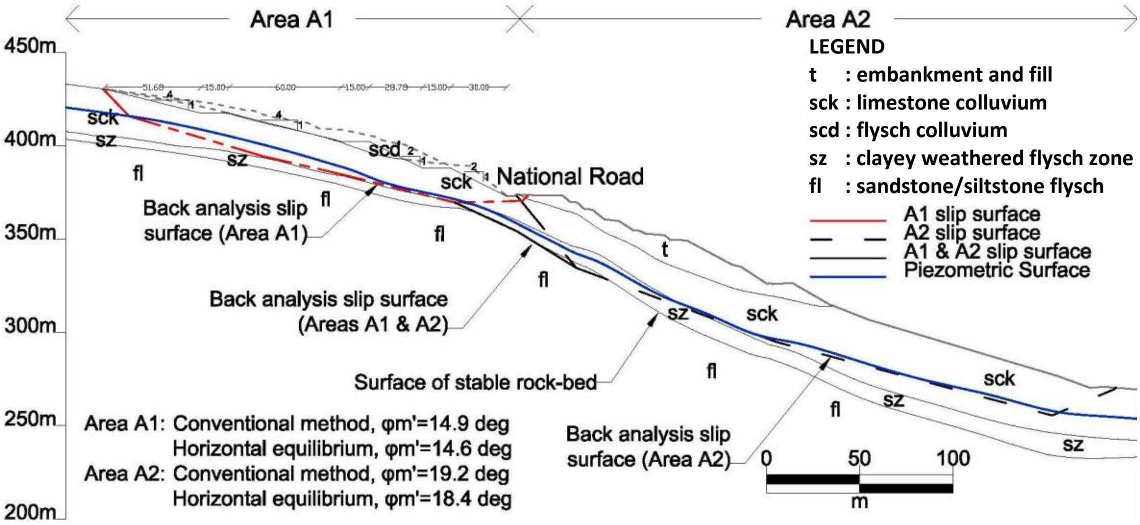

Figure 7: Cross section showing the excavation in area A1 evaluated as a possible remedial measure.

Your supervisor advises you to focus on the calculation of overall stability for sliding along the existing slip surface. You will assume that the relevant mobilized angle of shearing resistance along this slip surface is equal to the previously calculated ϕ_m' through back analysis. First, you will perform a stability analysis for the piezometric level considered in the back analysis. Then, a series of analyses will follow for various values of pore pressure ratio $r_u=u/\sigma_v$, which represents a mean piezometric level above the slip surface (the piezometric level for the back analysis corresponds to a value slightly higher than of $r_u=0$). The new A1 area (i.e. after excavation) has a mean surface slope of about 12°, which results in a theoretical value of r_u of about 0.47 when approximating the slope as infinite and assuming that flow is parallel to the ground surface (Belokas and Anagnostopoulos, 2011). Therefore, the repair alternative can be evaluated for plausible ground water conditions, described by an r_u value varying from 0 to 0.3.

Part IIb – Results from repair alternative analysis

The calculated FoS assuming the piezometric level used in the back analyses is 0.961 and 0.973, for the conventional and the horizontal equilibrium methods, respectively. In other words, the slope is even more unstable after excavation! This unanticipated finding is likely a result of the reduced height of the sliding mass for the same piezometric level, i.e. of the higher percentage of saturated soil within the sliding mass. Hence, analyses were performed for only a small range of r_u values, in order to investigate the effect of further draining of the slope. The results are given in **Table 2** and show that the slope is marginally stable even when fully drained (r_u=0) and, hence, excavation is not a viable repair alternative.

Table 2: Calculated factor of safety for area A1 assuming an extensive excavation, using the activated slip surface, residual shear strength and small r_u values.

r_u	0.00	0.05	0.10
FoS (Conventional Method)	1.082	1.023	0.965
FoS (Horizontal Equilibrium)	1.080	1.022	0.964

What happened at the end?

During the heavy-rain winter of 2003, the pavement suffered considerable settlement in January, which soon developed into a large pothole (**Figure 8**). Cracks were enlarged, and increased flow rates were recorded in the drainage system of the slope. In early February 2003, with rainfall continuing, a rapid movement of earth material took place, cutting through a 200m section of the highway. Movements continued over the next several days. When the sliding mass reached a resting position, the pavement had moved 100m horizontally and dropped 40m vertically (Fig. S.9 in the supporting material). The extent of the 2003 failure on the cross section is shown in Figs. S.4c and S.5b of the supporting material. The limits of the landslide extended further downslope to the riverbed (shown in Fig. 4), reaching approximately 1km length.

Figure 8: Large pothole at the problematic section of the highway (from Dounias et al., 2006).

Due to the large volume of the sliding mass, the repair alternatives were more costly and more uncertain than bypassing the unstable area altogether. Two such solutions were considered, a tunnel behind the unstable mass, going through the flysch stable bedrock and below the slip surface, and a bridge, with a span of 300 m to ensure the foundation of bridge piers on stable material. At the end, the bridge was selected as the most economical solution.

Lessons learned (in hindsight)

- Changes in design, construction provisions and overseeing authorities mid-way in a project create heightened communication needs to address potential communication gaps.

- Some observations before the final rapid soil movement provided “hints” of the developing problem: the milder slope indicates a transition to a less competent material, while cracks perpendicular to the road axis point to a slide, either first-time or reactivated. However, it is a very tough decision for an engineer to halt construction or request additional costly investigations on the basis of such hints alone.

- Careful observations of the natural relief can provide clues of past earth movements, which may recur. Often these observations are meaningful at a scale larger than the area immediately affected by the geotechnical project at hand. Clearly, this is knowledge gained in hindsight, which underscores the usefulness of case studies in helping notice things in another project.

- Average rates of displacement measured for just a few years cannot be used to predict future displacements, particularly if they are not linked to rainfall records. In this case, a prolonged very wet season most probably provided the trigger for the large movement.

- Although the displacement rate measured during the 2000-2001 investigation was low, it could not be dismissed since an acceleration of the movement is more probable in a modified environment compared to a natural one. It could be argued that if immediate deep drainage measures were applied, they might have delayed the evolution of the slide and reduced the possibility for the major triggering until permanent repair measures were in place.

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