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Lessons from Revisiting Three Cases of Slope Failure

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ABSTRACT

Three old failures of geotechnical projects are analyzed, where some original design uncertainties were lately discovered. The first case concerns the failure during construction of a slope at the portal area of a tunnel in Greece, due to unexpected excessive water pressures. The failure was restricted to the upper weathered zone of flysh formation. Due to difficulties in performing laboratory tests with such hard soil material necessitated the estimation of shear strength parameters on the basis of the rockmass classification systems. The next case deals with an extensive failure of an open pit mine slope in Greece. The ground consists of alternations of lignite layers with very stiff clay. Despite the detailed investigation on the peculiar nature of shear strength, the unavoidable scatter of the values in combination with the applied low safety factors and the development of excessive pore pressures, lead to this failure. The last case refers to the well known failure in Kimola Canal and addresses the probable failure mode and the proper analysis method. The analyses of the slope stability in formation of post glacial clays had been carried out with $\varphi=0$ and the use of undrained shear strength. The failure surface was developed upwards to a great distance and it could interpreted on the basis of the effective stresses analyses, taking into consideration the influence of the shear strength anisotropy.

INTRODUCTION

The short and long-term slope stability is mainly related to the soil nature. In highly overconsolidated stiff clays or very weak rocks, factors such as time-dependent softening, weathering and strain-dependent weakening influence the long-term stability. Back analyses of delayed failures contributed to the development of quantitative procedure to account for these factors, as e.g. by Skempton (1985), Taylor and Cripps (1987). On the other hand, slope failures in such soil formations are usual during or shortly after the construction often triggered by intense rainfalls. High pore pressures developed by rain infiltration and recharge of the water table, sometimes lead to slope destabilization.

The problems in slopes of N.C. or slightly O.C. saturated clays are different. Factors such as time dependent redistribution of pore pressures due to excavations, the soil inhomogeneity and shear strength anisotropy are also involved in the slope failure.

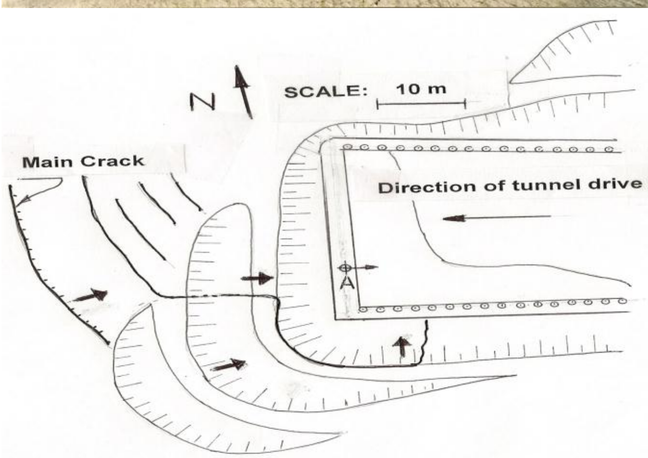
Today, several advanced laboratory or in situ methods are available to geotechnical engineers and analytical tools. However, due to scatter in the soil parameters and the physical complexity of the problems, experience and judgment are still of greatest importance. As remarked by Popescu (1993), it is now widely recognized that case studies are more likely to contribute in resolving slope stability problems than theoretical or laboratory work only. In this article three such slope failure cases are presented and analyzed.

CASE 1: SLOPE FAILURE AT THE PORTAL EXCAVATION OF THE TUNNEL S2, GREECE

Brief case-history and geotechnical conditions

The west portal of one of the Egnatia Highway tunnels S2 (Epirus) was constructed with open excavation and partial refill. During the initial phases of excavation 2-3 benches were formed, each of which was 10m high and had inclination 1:1. From the floor of the lower bench one series of retaining bored piles was constructed at the two sides, so that a vertical cut 10 m high can be formed during final excavation up to the designed level. On the contrary, the final 11m high slope at the face would be vertical and unsupported. In the area of the face, the maximum height of excavation would be about 25m. For the avoidance of rockfalls from the faces of the slopes, shotcrete and swellex or perfo anchors were placed, of 8 m length. For the excess pore pressures dissipation drainage holes, approximately horizontal with length 10 m, were constructed in 6-8 m space. The failure occurred at the left side (at the direction of the tunnel excavation) on January 1998 before the completion of the excavation of the lower vertical slope and after a period of intense rainfalls. The main scarp with 1-1,2 m depth was developed in 20-30 m spacing behind the slope crest, while minor scarps were observed in 5-10 m spacing. The overall appearance of the surface showed a slump type failure. The foot of the failure was almost at the level of the heads of the piles from the left side, while at the face was at the corresponding upper level of the final vertical slope, where a horizontal part existed with width 4 m approximately. The shotcrete suffered serious damage, while water flow was observed in many places including from some drain holes. In Fig.1a a general view of the portal area is given, a few months after the failure. In this area the tunnel passes through flysh, which was met in geotechnical investigation (alternating layers of siltstone and sandstone).


a



b

Fig.1a: General view of portal area looking towards the east (photograph taken from the point A).
Fig.1b: Plan of the portal area (sketch).

This formation covered by a mantle of colluvial deposits and intensely weathered flysh. The upper zone with 6 to 10m thickness pertains mixtures of different grain size distribution, like CL, ML, GM and also weathered flysh or fragments of sand/siltstone. The SPT tests in the mantle gave values N=17-62. In borehole samples classification tests but not shear strength tests were carried out, because the soil nature obviously didn’t allow the preparation of representative samples. On the other hand, systematic unconfined compression tests were carried out in core samples from the lower flysh, where a significant scattering of strength values was ascertained from their results. For example, in samples of siltstone the variation of unconfined compression tests in intact samples was σ_{ci}=0,6-26,5 MPa. Within the framework of geotechnical investigation, a classification of flysh and mantle formation was carried out according to Bieniawski’s (RMR) system. The groundwater level was met in boreholes at a level below the planned excavation. Lugeon in situ permeability tests in two boreholes found practically negligible permeability for the flysh.

<u>Investigation of the reasons of slope failure</u>

The slope stability analyses were based on the following presuppositions:

- The simplified Bishop method was applied. The influence of excess pore pressures wasn’t taken into account, because of the low level of underground water and the planned drainage measures.
- The measures of surface protection of the slopes (shotcrete and anchors) were ignored during the computations, in view of their negligible contribution to stability.
- The shear strength parameters of the two basic soil layers were based on RMR. For the upper zone of V category the corresponding parameters were estimated as φ′=15 and c′=100 kPa, while the safety factors were computed with tolerable values. However, the stability would be marginal with the most probable reduced parameters.
- The design was generally based on the prerequisite that the work is temporary and in any case during excavation its acceptances might be re-confirmed or modified.

Two committees of experts examined the causes of the slope failure:

(i) Hoek and Marinos (1998) came to the following conclusions:

- Slope was probably restricted to the upper mantle of colluvium and weathered flysh.
- It is possible that a failure surface or surfaces pre-existed as a result of previous landslide activity.
- Due to low permeability of the overall rockmass and the general topography of the surrounding area, high groundwater levels could be anticipated in the slope. This low permeability also means that additional recharge due to heavy rain will drain slowly, thereby contributing to excess water pressures in through the rockmass.
  Such high pressures, induced by the heavy rainfall in December 1997, triggered the movement that resulted in

the damage.

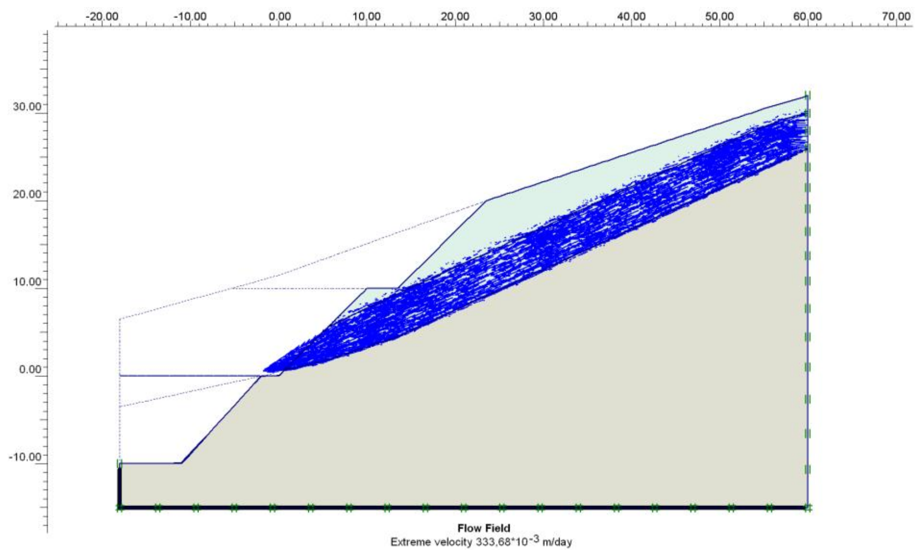
- The contribution of the shotcrete surface cover as well as of the Swellex and Perfo bolts to the overall stability of the slope was negligible.

(ii) Anagnostopoulos et al (1999) drew similar conclusions and furthermore:

- They deduced that the nature of weathered mantle didn’t seem to be included in the application field of the rockmass RMR classification system of Bieniawski; hence, the choice of shear strength parameters (φ΄=15 and c΄=100 kPa) might be questionable.
- They supported their views by the help of systematic back stability analyses for different combinations φ΄,c΄ and pore pressures parameter r_u, according to Bishop.
- They concluded that the main reason of the slope failure (high pore pressures) was the coincidence of unfavorable factors, like the contribution of the presence of shotcrete in development of excess pore pressures, the ineffectiveness of some drains, the nature of the upper layer, and the scatter of strength parameters.

New back analyses and comments

Using the F.E. program Plaxis V.8, the new back analyses were based on the findings of the previous paragraph. First of all, the probable because of the intense rainfalls flow net was investigated, which preceded the slope failure, according to coefficients of permeability of the two soil layers. In Fig. 2a the flow net is presented for the hypothetical case of free slope face. In Fig. 2b the flow net is shown for the real boundary conditions of anchored shotcrete slope.



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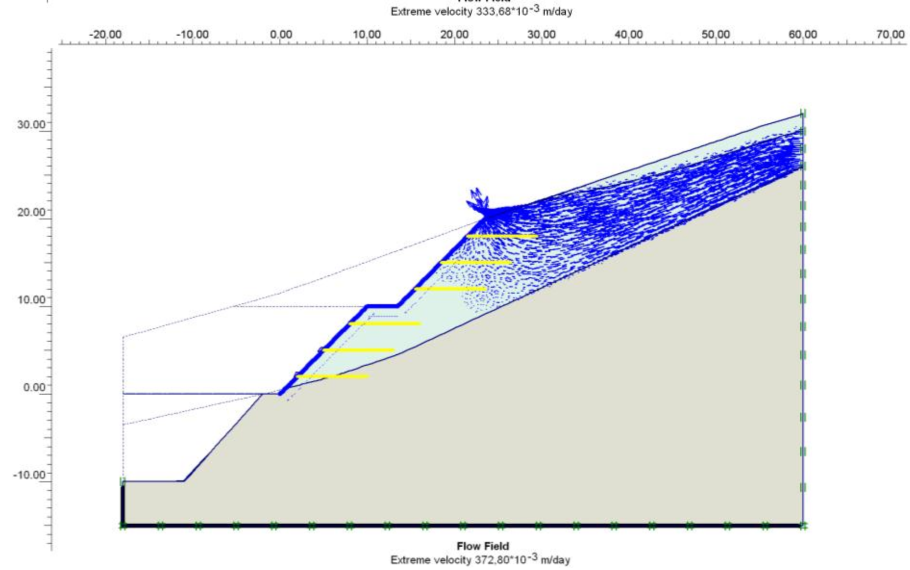

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Fig. 2a: Flow net in case of free slope face.
Fig. 2b: Flow net for anchored shotcrete on the slope face.

It is obvious that the rise of the water level in the upper zone during rainfall was due to the negligible permeability of the underlain flysh, but also due to the restriction of the dissipation of excess pore pressures by the shotcrete. Of course, if all the drainage holes were operating well, no differentiation of the flow would exist, as presented in Fig. 2a and 2b.

The choice of shear strength parameters of the upper zone, consisting of materials between hard soil and weak rock behavior, constitutes the main problem. The nature of the upper zone (colluvial deposits and weathered flysh) is such that Bieniawski's RMR system isn't applicable.

However, a classification rockmass system could be marginally used on the condition that during the final choice of φ´,c´ parameters, the nature of the soils and the influence of the in situ stresses would be taken into account. In Fig. 3 a diagram of correlation of equivalent shear strength parameters is given for different values of lateral stresses σ_3´, according to Hoek-Brown (1997) criterion. The parameters c´, tanφ´ were computed for two limited mean values of Geological Strength Index (Marinos and Hoek, 2000) GSI=12 and GSI=17 (for intensely weathered rockmass) with corresponding values of the parameters of the criteria m_i=7 or 8 and σ_{ci}=5 or 7MPa. In Fig. 3 two combinations of parameters are also shown for limited equilibrium of the slope from the back analyses. The corresponding points are included in the range of variation of the combinations c´ and tanφ´ and near the upper curve (GSI=17). In Fig. 4 the deformed finite element mesh is presented for a representative analysis. From the picture of the deformation of the anchored shotcrete its extensive damages could be justified. Finally, in Fig. 5 the plastic zones, and the shape of failure are presented, which seem compatible with the in situ observations.

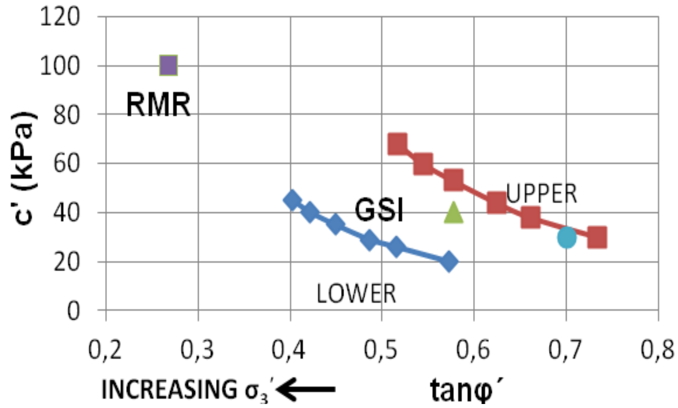

Fig. 3: Shear strength parameters according to Hoek-Brown criterion.

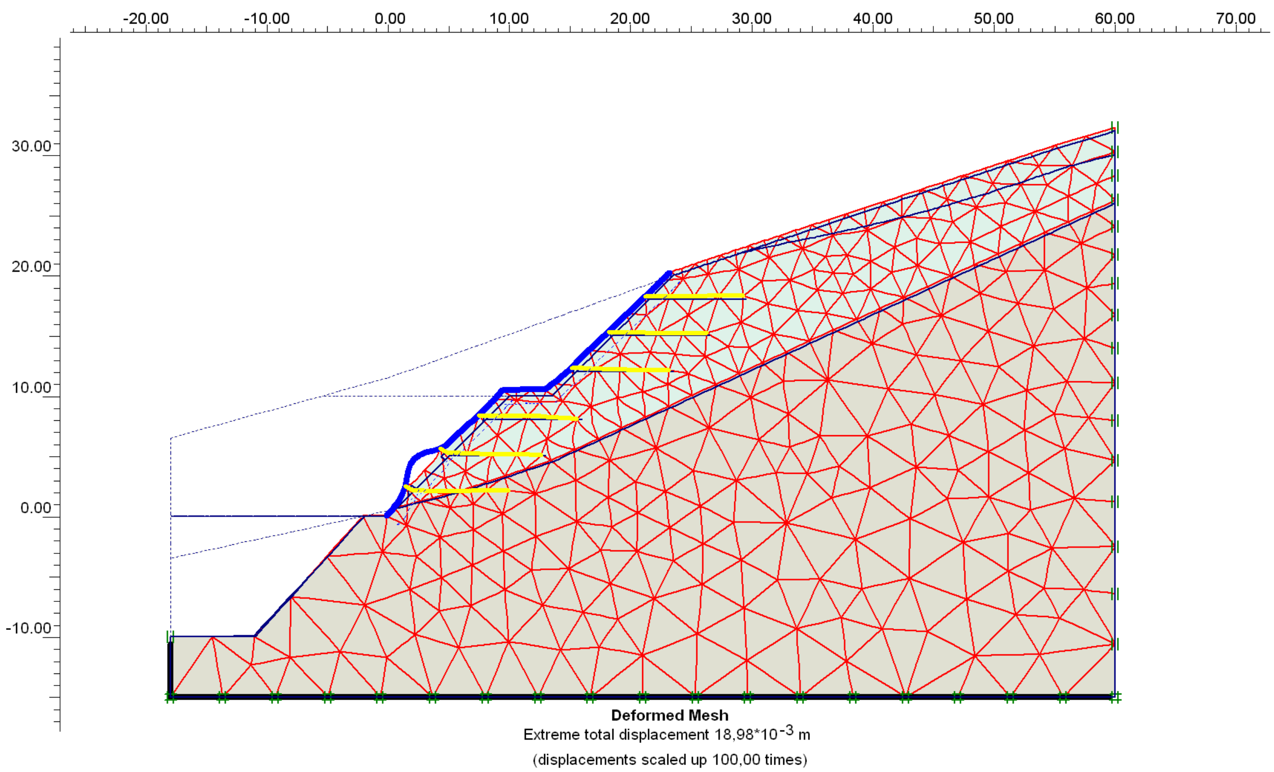

Fig.4: Deformed mesh and displacements of the anchored shotcrete.

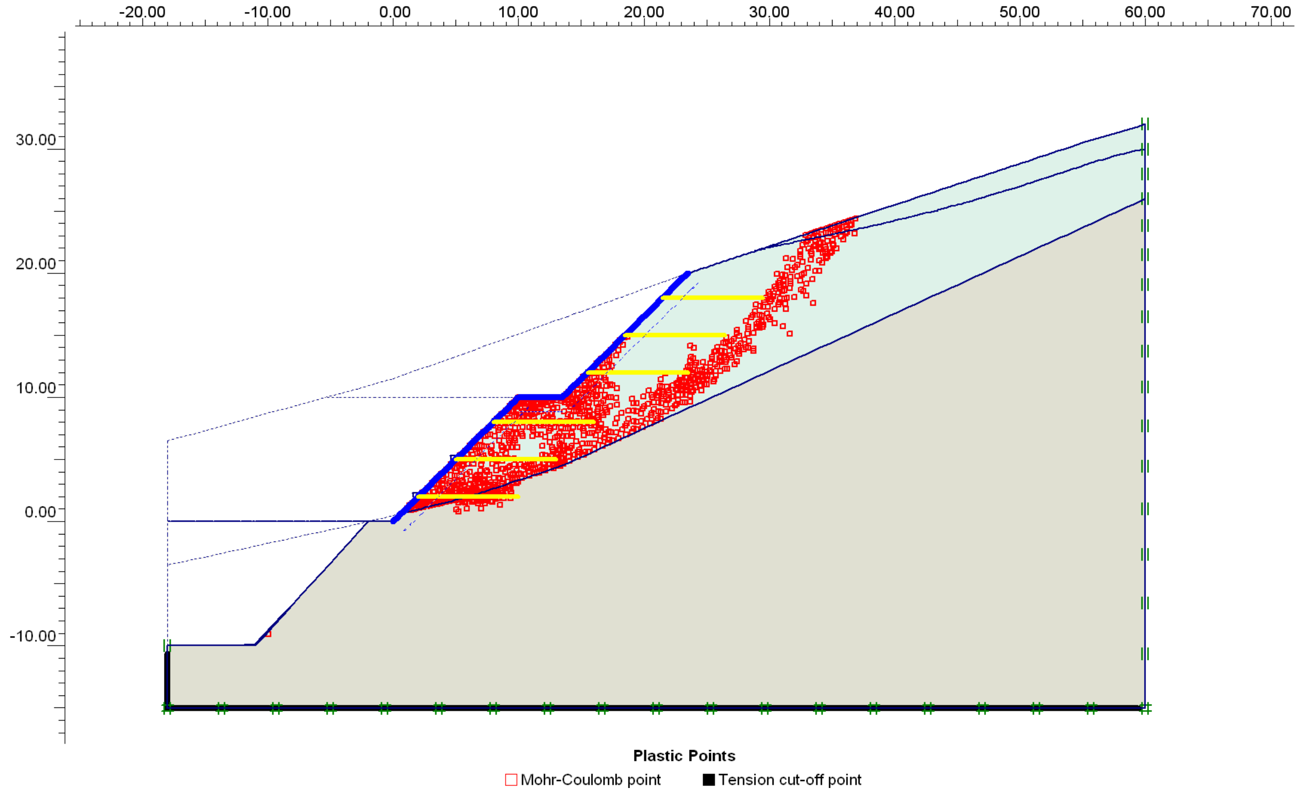

Fig.5: Mode of failure from a representative back analysis.

In conclusion, a suitable interpretation of the strength parameters, recognition of the inhibited water flow regime and numerical analysis offer a satisfactory explanation of the failure.

CASE 2: SLOPE FAILURE AT AN OPEN PIT MINE, GREECE

<u>Brief case history and geotechnical conditions</u>

Exploitation of surface lignite for power production creates different categories of slopes, the most important of which are the following:

(i) Short-term slopes, which are formed during the excavation with benches of exploitation.

(ii) Regional or permanent slopes at the boundaries of the mine.

Karas (1998) had mentioned the design and stability problems, reporting some cases of important failures. The specific slope failure was the biggest in the mine of Megalopolis, Peloponnese and occurred on January 1988 in a short-term slope during the operation of a large capacity excavator.

The slope had total height about 90 m, the failure occurred at a length of 400m, and the volume of the soil involved was about $2{\cdot}10^6$ m^3 (Fig.6).

Fig.6: General view of the slope failure.

The lignites of the Megalopolis area were investigated by Anagnostopoulos (1978), who found that their geotechnical behaviour is similar with that of highly overconsolidated clays. As Stamatopoulos and Kotzias (1981) had pointed out, the special characteristic of lignites compared with clays is the lack of plasticity. They have high void ratio, high water content and low bulk density, by reason of their open structure. These characteristics seem incompatible with their relatively high shear strength. The more important conclusions of the investigation on the causes of slope failure were the following:

- The main factor is the creation of temporary water level, because of the intense rainfalls, which had preceded of the slope failure. It is also important that the main group of joints followed the dip direction of the slope in the area of failure, so that these discontinuities opened up, because of the excavations, facilitating the inflow of the water.

- The specific place was an area of lenticular lignite layer, so that the proportion of the clay soil materials in comparison with the lignite was higher, while the shear strength of the clay was lower in the specific area.
- The significant scatter of the shear strength parameters of the clay as well as of the lignite was verified.

The back stability analyses with the use of simplified Bishop method for the typical design parameters and also unfavorable hydraulic conditions (which were unexpected for practically impermeable materials), justified the slope failure.

New analyses and comments

The new analyses were carried out after taking into account the variation of shear strength parameters and also the probable boundaries of piezometric line during the time of slope failure. The shear strength parameters of lignite usually range as φ′=24°-28° and c′=70-150 kPa. However, at the failure site, clay CL, CH, or OH prevails with lower shear strength as a rule. The range of shear strength parameters φ′,c′ variation is obviously important. From triaxal compression tests on samples of boreholes, which were made at the failure site the following values were taken: φ′=21°-28° and c′=25-80 kPa. In Fig.7 indicative results are shown from triaxal tests CIU on specimens, which were prepared from an undisturbed block sample.

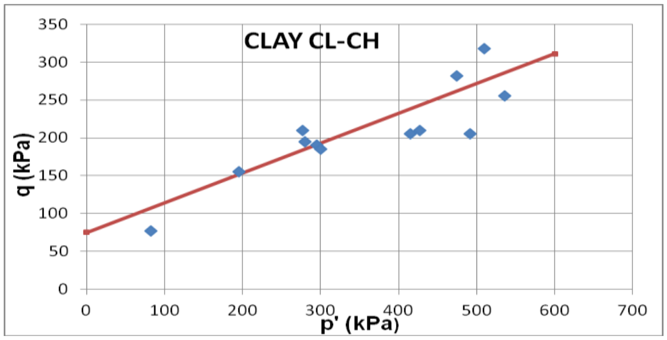

Fig.7: Representative diagram, p′-q from CIU tests.

The appreciable variation of results from only one sample is evident. The back analyses revealed important differences in the safety factor. Taking into account the mean design parameters mainly for the clay and the unfavorable possibility for the development of temporary groundwater level, the slope failure is successfully justified, as demonstrated in Fig. 8. Considering the low safety factors, which are applied at the open pit mines (MSF=1-1,3 in most cases), the importance of uncertainty in design parameters becomes obvious. Consequently, it seems that the experience of the staff and the capability of estimation of forerunner slope failure phenomena is a matter of particular interest.

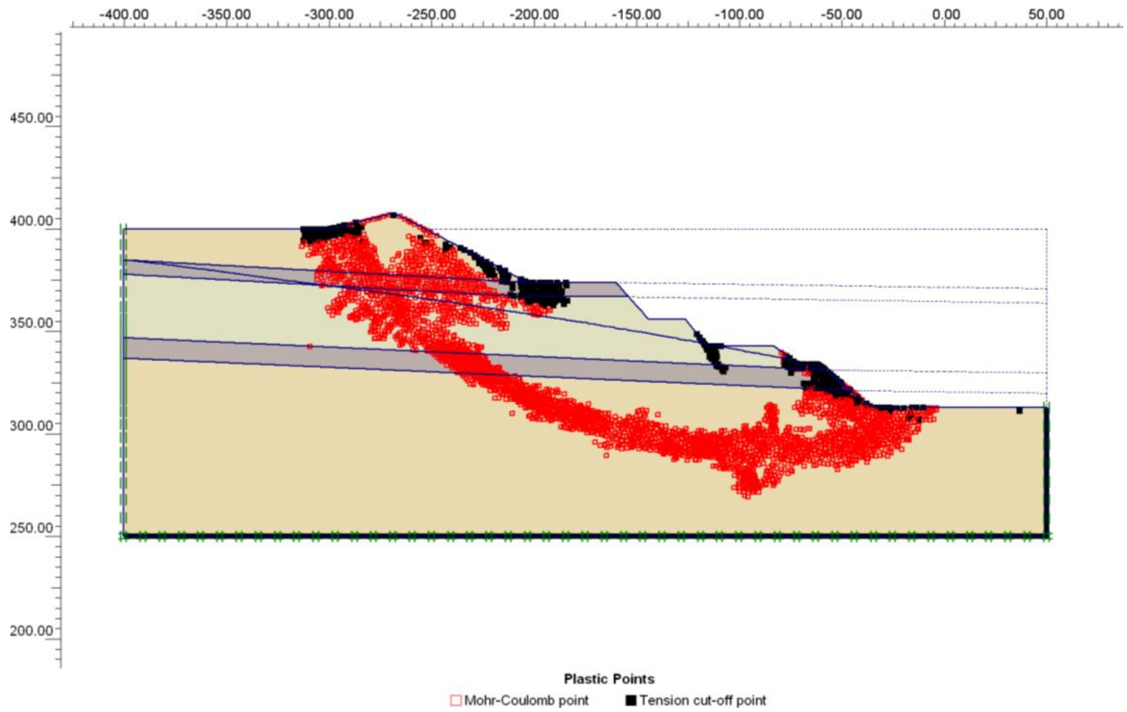

Fig.8: Plastic zones and failure mechanism.

CASE 3: THE SLOPE FAILURE IN KIMOLA CANAL, FINLAND, REVISITED

Brief case-history and geotechnical conditions

The Kimola Canal is located northeast of Helsinki, constructed to facilitate the floating of logs and timber. It consists of two segments at different elevations (13m apart) connected by a tunnel. For the realization of elevation requirements of the bottom and the trapezoid cross section of the canal, excavations in phases and steps were made with inclination 1:1,5 (vertical to horizontal). On November 3, 1965 a major slide occurred on the lower canal. Suddenly, at a location where excavation to grade was completed nine months earlier, 90000m^3 of clay slid into the canal. During the next months some failures of secondary importance occurred in levels with foot surfaces in the upper as well as the lower canal. The big slope failure in the lower canal became an object of reference, analyses and comments. Therefore, it is an important and familiar event among geotechnical engineers. The doctoral thesis by Kankare (1969) systematically presented the geotechnical data of the clay in the area of the canal, as well as relative back analyses of slope stability. In the seminal article of the late Prof. Leonard (1982) some basic data and conclusions of the above doctoral thesis are represented and important comments for the causes and interpretation of the big slope failure are set out. In Fig.9 a typical cross-section of the slope with theoretical critical circles as well as the active failure surface are shown.

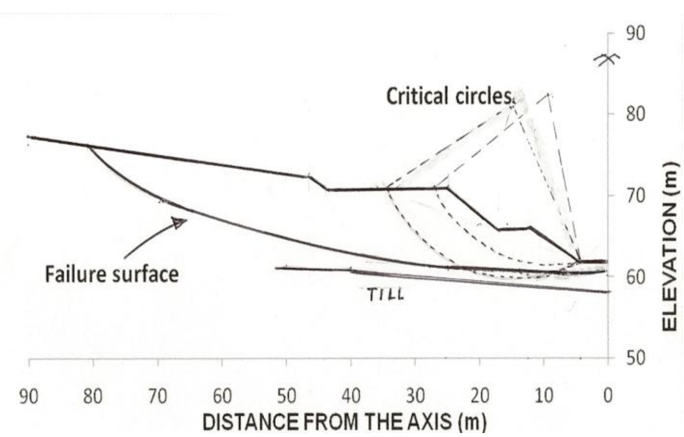

Fig.9:Cross-section of the failed slope (Sketch from a figure after Kankare, 1969).

Stability analyses and comments of researchers

(i) The canal is located in valley, covered by post glacial clay overlying interbedded sandy or silty tills, which layer of till-silty sand follow. The bedrock, which neither met the boreholes nor the slip surface, outcrops from the borders of the valley. The basic formation is clay of high plasticity, CH, (mean values of Atterberg limits LL=55%, PI=30%), where high sensitivity S_t=8-16 and slight overconsolidation was constated. In the wider area of the upper and lower canal boreholes were carried out, as well as drained or undrained triaxal tests on preconsolidated specimens from representative samples. In situ vane and cone penetration tests were also carried out. Finally, the pore pressures were recorded in several temporal intervals in piezometers, which had been installed in the upper and lower area of the canal before and after the slope failures.

(ii) The design of the work was conducted by stability analyses under undrained conditions and safety factor 1,5. The use of undrained shear strength instead of the effective parameters φ΄,c΄ began to be disputed after the appearance of the first small slope failures, which were indicative that the analyses were not representative. Within the scope of the doctoral thesis of Kankare (1969) back stability analyses were carried out in the two areas of important slope failures in terms of effective stresses by using of undrained shear strength. By the help of facts and information about the slope failure, Kankare drew the following conclusions:

- The analysis φ=0 (c=s_u), which was applied for the design, is unreliable for the cases of stability analysis in soil formations like these, namely as the slightly overconsolidated clay deposits of high plasticity.
- The analysis in effective stresses (φ΄,c΄) by the help of measured pore pressures can explain the slope failure, given that the resultant safety factors are approximately 1.
- However, the acceptance of circular failure surface is in contradiction to very extensive development of rupture surface upwards. So it was given the explanation that the failure was developed following a slightly inclined thin layer of sensitive clay (obviously with reduced shear strength).

(iii) In conjunction with the presentation of the data, information and the analyses of failure, Leonards (1982) set out his views as well as the views of other well known geotechnical engineers, such as Janbu and Bjerrum, the more important of them are the following:

- The slope failure, which broke out 9 months after the excavations is caused by the redistribution of the excess pore pressures, which was achieved after the stress equilibrium and analyzed in effective stresses. This view was supported with the calculation of the safety factors in different periods, where the excess pore pressures hadn't been measured. Taking into account this calculation the safety is limited in the period of the appearance of the slope failure.
- The shear strength parameters in terms of effective stresses φ΄,c΄ depend on the stress paths and the rate of strains in laboratory.
- The long term developed creep deformations could lead to limit values, where the structure of the clay collapses. This could be explained by reduced values φ΄,c΄ and the increase of deformability.
- The clay in the area of the canal is sedimentary and despite the fact that the failure surface wasn't developed

along the interface with the lower underlied zone, in a great part it was linear and parallel with the orientation of the bedding planes. It is doubtful that the calculated shear strength in lab tests in representative with the corresponding along the orientation of the bedding planes.

- The soil mass involved in the slope failure was much higher than this, indicated from the stability analyses in effective and total stresses. Furthermore, in other positions, where small slope failures in benches broke out, wasn't developed an extensive slope failure, like in the lower canal. Probably, the influence of other factors, which lead to the slope failure, had been neglected or ignored.

<u>New analyses and remarks</u>

- <u>Confirmatory back stability analysis</u>: The initial back analysis in this article was carried out with the geotechnical parameters of the doctoral thesis of Kankare (1969), as follows:

- The cross section 52+70 of lower canal was analyzed (Fig.9), where the geometrical data were taken graphically from the article of Leonards (1982).
- An analysis of long-term stability was carried out with effective parameters φ′=27,7° and c′=4,9 kPa, namely according to the values, which had been measured from triaxal compression tests (CID) and applied previously.
- The 2D program of finite elements Plaxis V8.6 was used.
- The pore pressures were evaluated in the basis of the piezometric measurements in this area. It is pointed out that in the doctoral thesis of Kankare (1969) this was realized by direct development of measurements in specific positions of cross-section and with interpolation at intermediate positions.
- In the analysis of the article, the flow net was calculated in the F.E. program of computer on the basis of the measurements. The safety factor, arised from the analysis, is MSF=0,992 (practically equal to 1), very close to the minimum values of the Kankare analysis, which according to the position of the circular failure surfaces varied from 0,97 to 1,01. In Fig.10 the plastic zones are observed at the limit equilibrium.

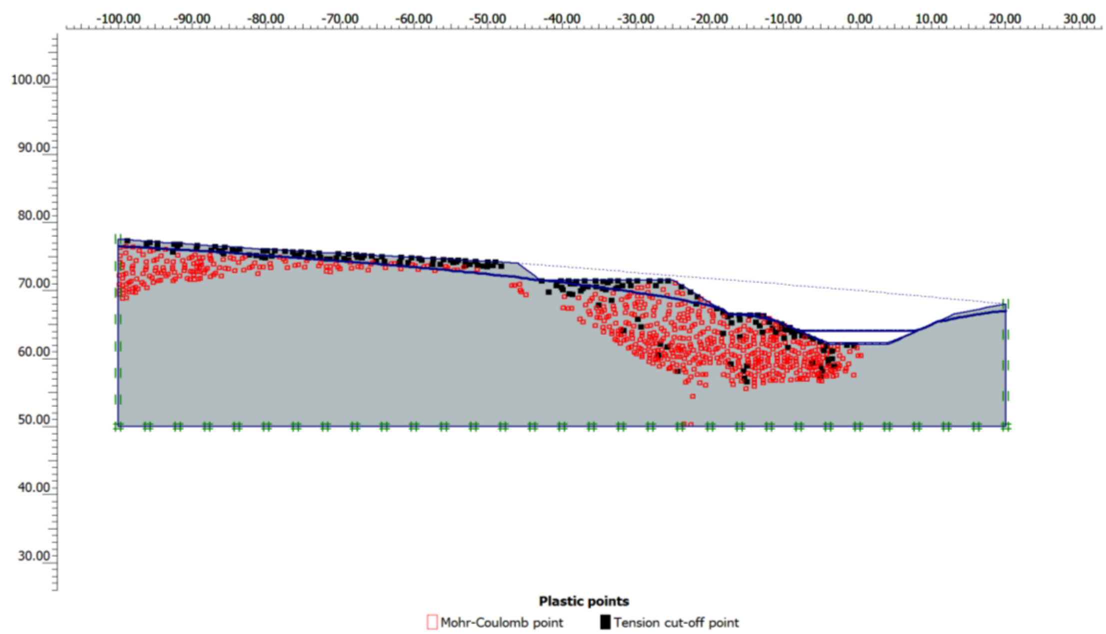

Fig.10: Plastic zones of the cross –section 52+70 during the failure. Analysis with effective stresses for homogenous clay.

From the first affirmative analysis and the available data, we draw the following conclusions:

(i) The analysis seems to be compatible with the time of the slope failure appearance, namely in a phase, where the initial excess water pressures, had been balanced. Obviously, the back stability analysis under undrained conditions (φ_u=0 analysis) makes no sense, which could be a way of approximation only from practical point of view.

(ii) During the limit equilibrium the tension cracks at the crest are shown off, like it happens in practice. The failure seems to have variable curvature and extends more than in Kankare analyses, which were carried out with acceptance of circular surfaces.

(iii) The safety factors, which arised from effective parameters, as it is exactly given from the laboratory tests, justify the slope failure. It is observed though that the analysis (without the acceptance of circular failure) isn't attributed to the extension of soil zone, which was collapsed. We are lead then to the conclusion that Leonards (1982) also drew, that probably the confirmation of slope failure by the analyses could be coincidental.

Following the article, we refer to the two factors, which weren't examined and couldn't be before hand examined, namely as the inhomogeneity and the shear strength anisotropy of the clay.

- The inhomogneity of the soil: We refer to the inhomogeneity in the upper zone, where the slope failure broke out and non to the presence of a lower zone till-silty sand doesn't seem to have any impact to the fact. The inhomogeneity is clear, because of the variation of shear strength versus the depth. Intuitively, we expect different failure surface in the soil with increasing strength versus depth, taking into account that the surface upwards zones have smaller strength than this at lower levels. Therefore, the development of shallow failure surface, which extends upwards, in comparison with the surface which corresponds to homogeneous soil, is possible. However, the rate of increase of shear strength in connection with the depth couldn't be directly and accurately developed, in order to be simulated the inhomogeneity of clay zone in terms of effective stresses. Yet, this procedure was attempted with constant angle of internal friction and linearly increasing cohesion c′ with compatible way with the geotechnical data. The arised safety factor, from the analysis, which had indicative character with the acceptance of inhomogeneous soil, was MSF=1,135. The plastic zones are presented in Fig.11 and it is confirmed that the failure mechanism extends upwards enough and approaches a lot the really developed mechanism.

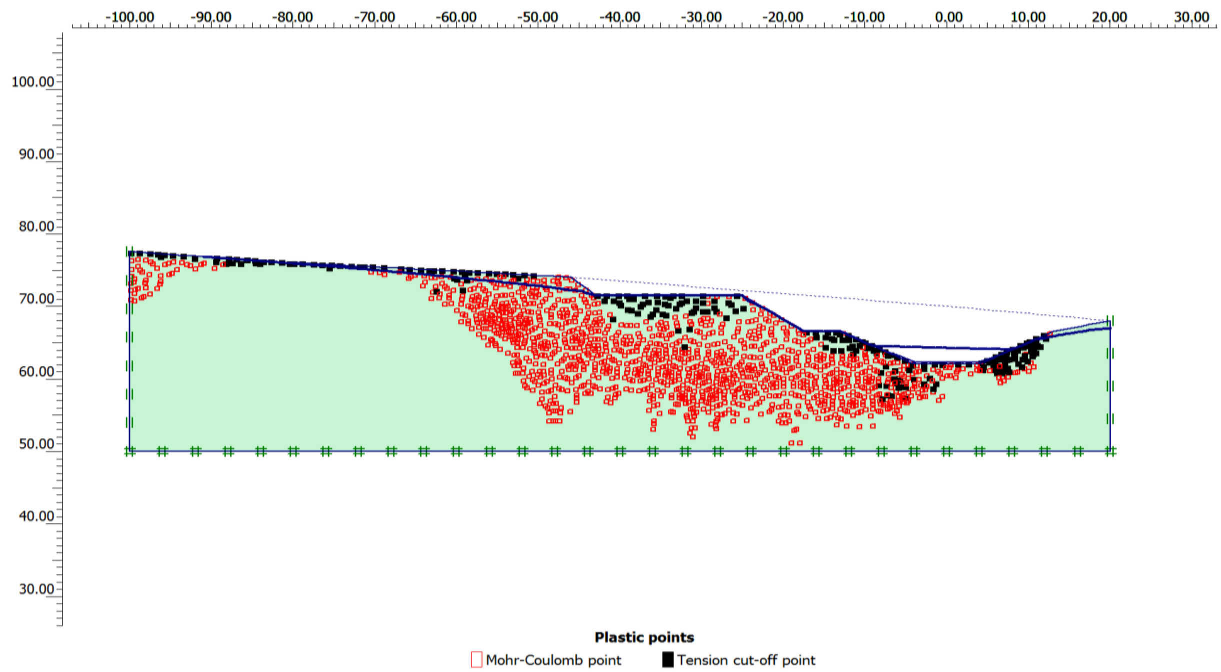

Fig.11: Plastic zones of the cross –section 52+70 during the failure. Analysis with effective for inhomogeneous clay.

- The shear strength anisotropy: After the ascertainment that the upper zone had orientation of stratification parallel to the lower till-or silty sand zone and to this orientation the failure surface was developed having plane its greater part, it seems that the degree of shear strength anisotropy, had an important effect in this fact. It is known that the stratified clays appear shear strength anisotropy, which obviously corresponds to anisotropy in terms of effective stresses, where it isn't always to be confirmed by practical way. Obviously, the minimum value of undrained shear strength, mins_u corresponds to the orientation of stratification, while the maxs_u is expected to be in perpendicular direction. Leonards (1982) in his articles wonders if the shear strength had been investigated at the direction of its minimum values, namely at the orientation of stratification. So the following comments are expressed:

(i) The stability analyses were based on the undrained shear strength, which was mainly investigated with vane tests. In failure, in this test the mean value of s_u is estimated on the lateral surface of the ideal cylinder with vertical axis, so the s_u at the vertical direction. Consequently, the calculated values seem to approach the max s_u, due to anisotropy and the slightly inclined stratification. Taking into consideration that the degree of anisotropy maxs_u/mins_u neither is known, nor can be estimated, the variation of s_u diagram with the depth is doubtfully representative of the shear strength as well as of the estimation of stress history (slightly overconsolidated or maybe normally consolidated clay).

(ii) The existence of direction of decreased shear strength (orientation of stratification in this case) leads to selected failure surface, which isn't circular, but follows on a great part the directions which approach the minimum strength. If it is taken into consideration the ascertainment of zone with decreased shear strength on the base of failure surface, the development of the observed failure slope mechanism will be completely justified. The indicative solution of Fig.12 was carried out with acceptance of inhomogeneous soil and shear strength parameters, exactly as in the analysis of Fig.11 (φ´ constant and c´ linearly increased with the depth), but with approximative simulation of the decreased shear strength at the direction of stratification. A plane was considered at the depth of the slope failure, interface elements were given along this plane and the strength reduction factor was presumes as R=0,5. This means that the values c´ and tanφ´ along the interface are decreased in comparison with these of underlied soil in 50%. It is clear that this solution gives with good approximation the real failure mechanism, which deviates a lot from a circular failure surface.

(iii) It is obvious that the conventional triaxal compression tests on samples, which were taken from vertical boreholes, give shear strength parameters, which are developed in the direction of failure surface. This deviated a lot from the minimum shear strength. We wonder then, for the practical meaning of taking into account factors, which influence the results during these tests (stress path and rate of strains), because the anisotropy, which is completely neglected, is maybe the more important. The more appropriate tests are direct shear strength tests, drained on preconsolidated specimens, in order to estimate the effective shear strength parameters in the surface of stratification or even though in direction very close to it.

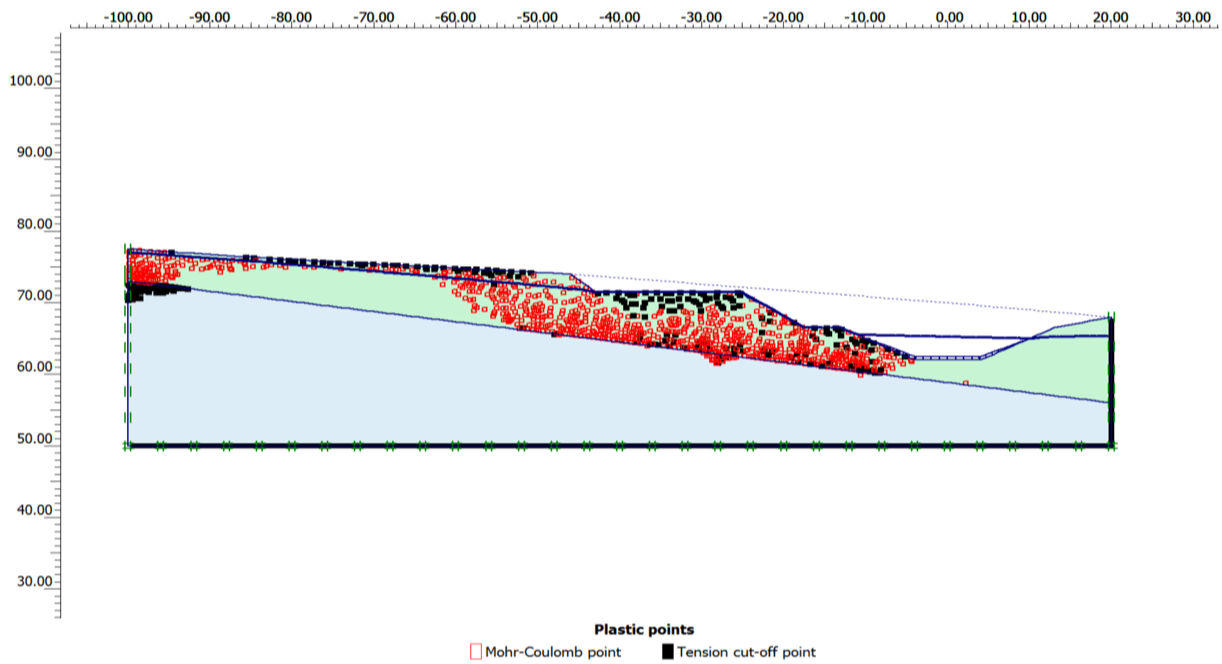

Fig.12: Plastic zones for inhomogeneous and anisotropic clay.

CONCLUSIONS

The main trigger of slope failures of the cases 1 and 2 was the unexpected excessive water pressures after a period of intense rainfalls.

The case 1 refers to inherent difficulties in choosing proper parameters φ´,c´ in soil with intermediate behaviour between hard soil and weak rock. Due to the peculiar soil nature, difficulty in preparation of representative samples for laboratory tests was presented, while questionable was also the capability of application of classification systems of rockmass. In case 2, firstly the peculiarity lies in the fact that a low safety factor must have been operated during the slope design. Another basic problem was the successful choice of shear strength parameters. The alternation of clay layers with great variation of parameters φ´,c´ and lignite layers, which is an unusual soil material, influenced the slope stability.

The great importance of the engineering judgment and the systematic observation and explanation of geotechnical findings is pointed out in slope stability.

The case 3 deals with the unsuccessful choice of analysis method φ=0, s_u, during the slope design. Analyses in terms of effective stresses (φ´,c´) taking advantage of measured pore pressures, explained the slope failure but not its shape. Factors like inhomogeneity and anisotropy of the clay could justify the extension upwards of the failure surface.

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