A Large Scale Landslide in a Coal Mine in Marly Formations: Evaluation, Analysis and Rehabilitation

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ABSTRACT: *This paper describes a large scale landslide in a coal mine in Servia area in Kozani, Greece, which occurred in February 2011. The landslide, of ~250m width and ~350m length, took place in marly lacustrine sediments. The main factors for the manifestation of the landslide were a temporary interruption of works, the existence of a specific weak surface, the clayey nature and the sensitivity of marl to environmental agents, the action of water and the large slope height. Due to the magnitude and the position of the failure, a geotechnical solution was proposed that involved rearrangement of ground masses. The proposed solution led to successful rehabilitation and unimpeded continuation of the exploitation process.*

KEYWORDS: Landslide, marl, progressive failure, coal mine, rehabilitation

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

The landslide discussed in the present paper took place in a coal mine near Prosilion village, in the wider region of Servia, Prefecture of Kozani in northern Greece. The coal mine is in operation since 1995, with an intermediate interruption of works between 2000 and 2005. In September of 2008, cracks were observed at the ground surface at the NW side of the coal mine. For the evaluation of the phenomenon, a systematic surveying monitoring was initiated, which indicated a slow but constant kinematic activity. In order to control displacements and avoid failure in the area of sliding movements, retaining action was taken with the commencement of construction of a lateral counterweight from excavation products (noncommercial deposits). However, at the end of 2008, works were stopped due to a relevant adjudication. During winter of 2010-2011 an increase of displacements was observed, with an increasing rate that led to a large scale landslide on February 27th, 2011. The authors were requested by the exploitation contractor to provide expert consulting services for the necessary landslide rehabilitation works.

The width of the landslide was more than 250 m and its length, from its scarp up to the lowest point of deposition of its products, exceeded 350 m. The maximum depth of the failure surface reached 35 m, resulting in a total landslide volume of more than 5×10^5 m^3. The sliding mass impacted a local road, the pavement of which was displaced around 25 m horizontally and approximately 12 m vertically. The landslide took place practically entirely in marly lacustrine sediments. Specifically the wider area is dominated by marls, with a considerable degree of geotechnical deterioration due to weathering at their upper parts, while the thickness of the surface scree formations is quite limited, especially in comparison to the scale of the landslide feature. Weathering recedes with depth, where fresh marl is encountered, with characteristics of rocky material. The interface between the upper weak and the underlying competent marl is an easily recognizable weak surface that is generally following the bedding of the formation.

Submitted: 05 June 2015; Published: 16 December 2016
Reference: Prountzopoulos, G., Fortsakis, P., Marinos, V. and Marinos, P. (2016). *A Large Scale Landslide in a Coal Mine in Marly Formations: Evaluation, Analysis and Rehabilitation*. International Journal of Geoengineering Case histories, Vol.4, Issue 1, p.29-45. doi: 10.4417/IJGCH-04-01-03

Figure 1 includes a demarcation of the position of the landslide, together with an aerial photo of the coal mine, only few days before the landslide. The landslide area, the local road affected by the landslide and the initially placed deposits for lateral retaining action to the mine slope are indicated on the satellite photo.

Figure 1. Position of landslide (up – red pin insite the red rectangle) and aerial photograph of the coal mine 2 weeks before the landslide (down). The area that was later occupied by the landslide is indicated with the transparent shape.

FAILURE MECHANISM AND DESCRIPTION OF LANDSLIDE

Figure 2 shows a plan view of the landslide area, with demarkation of the main features of the incident, as mentioned or referred to in the present paper. After examination of historical, geometrical and geotechnical data provided by the exploitation contractor, in-situ inspections and evaluation of the available information, it was concluded that the factors that played an important role in the initiation and triggering of sliding movement and the evolution of the landslide were:

1. The period of stoppage of works, which enhanced instability phenomena, allowing the action of environmental agents, in combination with the initial small scale displacements and cracking that are inevitable in earthworks of such dimensions within soft rocks, due to stress relief.
2. The systematic and constant action of rain water. In addition to causing deterioration of upper marl quality, water also contributed by a gradual increase of pore pressure along various preceding sliding surfaces, after infiltrating in the generally impermeable formation through open and constantly widening cracks.
3. The existence of a distinct weak contact surface between the weathered and the underlying compact marl. This specific interface has been often shown to contribute to failures as a common sliding surface, not only from similar incidents in the area (Prountzopoulos et al., 2010) but also from statements of the on-site engineers of the specific project who attested that this surface had caused small scale failure incidents in the same coal mine before the specific landslide.
4. The clayey, weak nature of marl, especially in shallow depth, as well as its sensitivity to the action of environmental agents. The progressive transition of marly formations to residual strength conditions, due to combination of the action of water and small scale displacements, has been repeatedly reported in the Greek territory and has been related to landslide phenomena (Cavounidis & Sotiropoulos 1980, Dounias 1988, Prountzopoulos et al. 2014).
5. The important height of the mine slope cuts (>60 m), which was not timely retained through deposition of non-comercial excavation materials, due to the stoppage of works with adjudication.

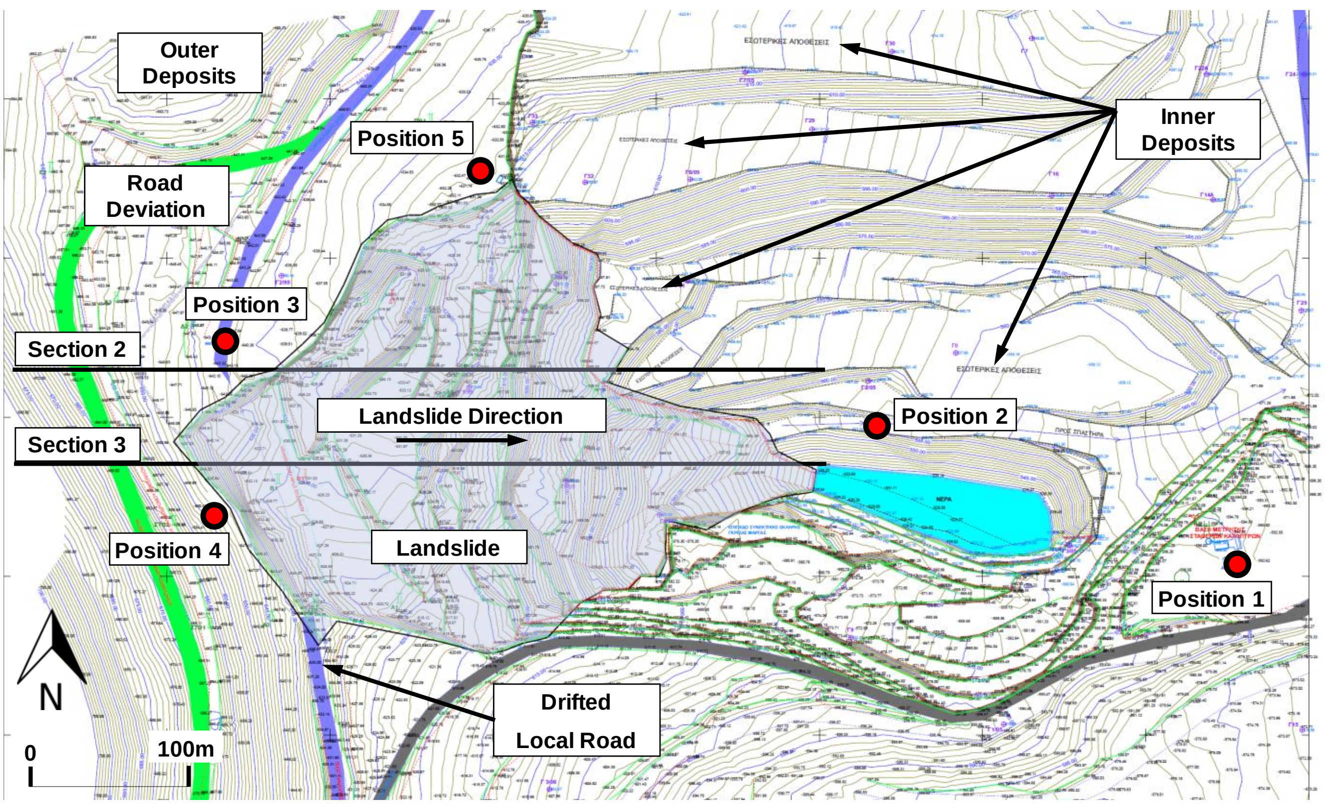

Figure 2. Topographic mapping (plan view) of the coal mine after the landslide with demarkation of some key features

Summarizing, the sliding initiated due to the long-term effect of environmental agents on the high mine slope cuts. Creep related phenomena in combination with the existence of the weak contact surface between the weathered and the compact marl, caused initial small scale displacements and slides. These initial movements resulted in cracking around the slope crest, ingress of water in the initially impermeable marly formation and an initial reduction of marl's strength. Action of water in the weaker formation caused widening of cracks and additional evolution of displacements, which favoured deeper infiltration of water and additional transition of marl's shear strength towards its residual value. This process continued accelarated until the final large scale failure.

Figure 3 illustrates a general aspect of the landslide from its E and NE side (Positions 1 and 2 in Figure 2) and a view of the contact surface between the weak and the competent marl, which played a major role in the landslide manifestation. Figure 4 illustrates various aspects of the landslide and its main features.

The landslide had a clearly three dimensional character, due to the lateral "constraints" posed by the non-commercial deposits of the mine at its north part and the exploitation limit at its south part. The failure surface at the landslide crest at its NW boundary had a considerable (subvertical) inclination and was steeply dipping towards the weak contact surface between the weathered and the underlying compact marl (Figure 4a left and 4c). On the contrary, at the west boundary of the landslide crest, the failure surface had a milder inclination (Figure 4a and 4b right) and seemed to have travelled a considerable length within the shallower weak marl formation. Within the landslide, large blocks of the marly formation were almost monolithically displaced (see Figure 4a and 4d) with limited cracking and detachments within their mass, giving the impression of a combination of rock slide and typical rotational landslide to the incident.

Actions for the enhancement of the stability of the area and the rehabilitation of the landslide had to be taken immediately. Clear signs of instability migration outside the boundaries of the landslide were observed, in the form of new cracks uphill of the landslide crest. A possible extension of the instability to the W-SW would seriously jeopardize the alternative route (already under construction) of the drifted local road and affect areas outside the expropriation boundary of the coal mine. In that sense, evaluation of the landslide and commencement of stabilization works had to be performed within the shortest possible time period.

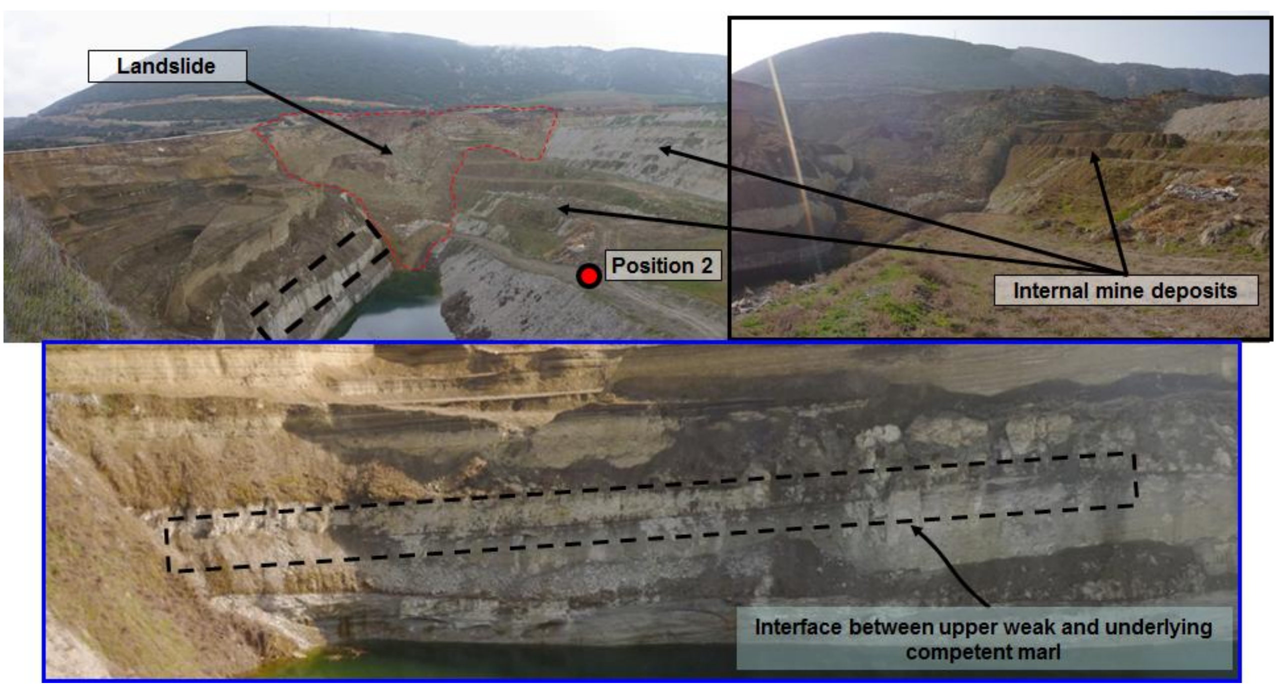

Figure 3. Aspect of the coalmine and the landslide from Position 1 (upper left), the landslide from Position 2 (upper right) and the contact surface between the weak and the competent marl (bottom)

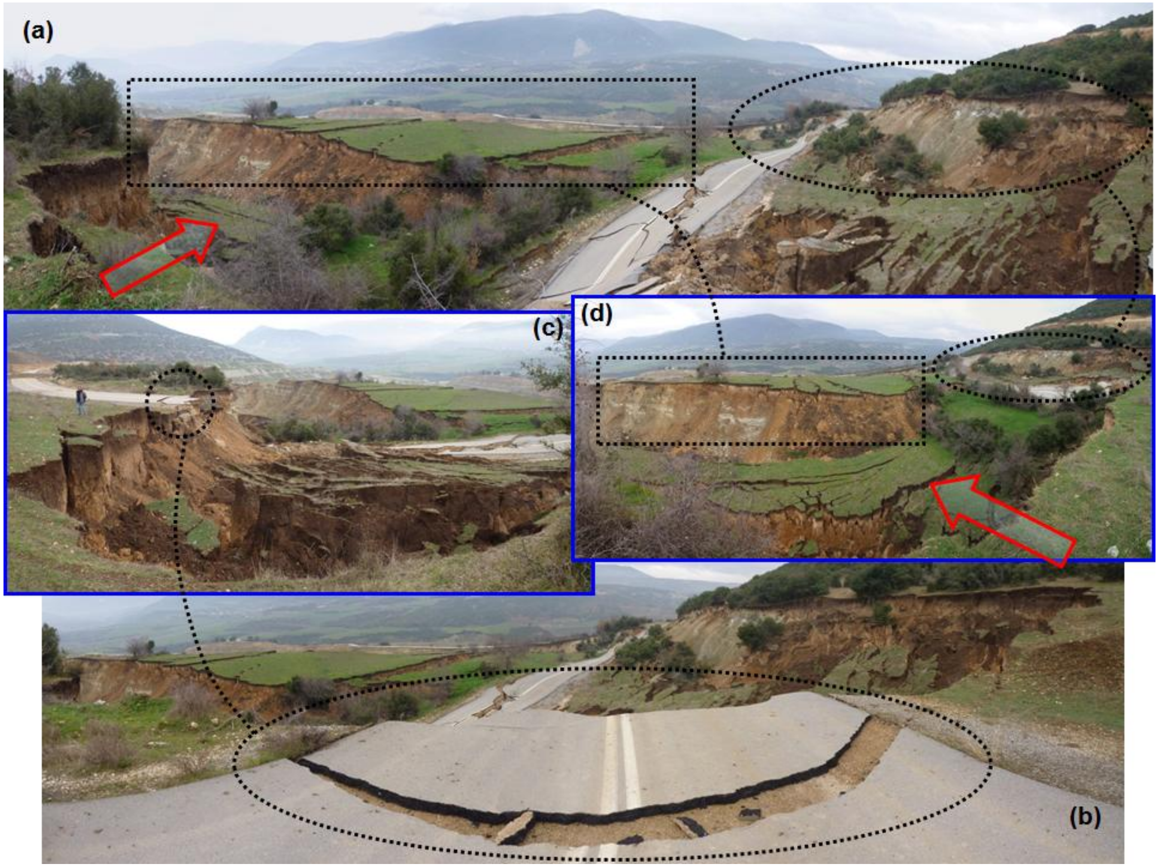

Figure 4. Aspect of the landslide from Position 3 (a & b), Position 4 (c) and Position 5 (d), the landslide crest (oval shape in a & d), the block that moved almost monolithically downstream (rectangle in a & d) and the mass that sank behind the block (arrow in a & d).

EXISTING GEOTECHNICAL INVESTIGATION

The main part of the investigation that had already been performed in the coal mine was focused on the exploitation design. Therefore, only two sampling boreholes were available that were accompanied by some limited in-situ and laboratory testing for the estimation of the physical and mechanical characteristics of the in-situ formations. These boreholes were executed for the design of the alternative route of the road that was later drifted by the landslide. Due to (a) the scope of the boreholes and the corresponding laboratory testing, which practically was restricted on the foundation support of the new local road and (b) their location (towards the outside part of the sedimentation basin), their value was really limited for the evaluation of the landslide event.

Given the limited time frame for the commencement of stabilization works and the unavailability of proper drilling equipment and specialized geotechnical laboratory in the area, it was decided to proceed with the existing experience from a similar event in an adjacent coal mine (Prountzopoulos et al., 2010) and, mainly, the use of back analyses of the landslide itself. A summary of the estimated shear strength parameters (values of the residual friction angle) for different values of the pore water pressure ratio R_u from the available case history in the area is provided in Table 1.

Table 1. Range of geotechnical parameters' values in the area of the coal mine (Prountzopoulos et al., 2010).

Parameter	R_u=0.10	R_u=0.20-0.40
Residual friction angle of weak marl φ_{marl}	15° - 17°	17° - 18°
Residual friction angle of contact surface between weak and competent marl φ_{surf}	10° - 11°	12° - 17°

BACK ANALYSES OF LANDSLIDE

The back analysis of a landslide is a complex procedure due to the usually large number of influencing parameters, i.e. geometry - morphology, geotechnical properties, groundwater conditions and additional loads and therefore different approaches can be found in literature. Chandler (1977) presented an integrated approach for the back analysis of landslides using the case record of Barnsdale landslide in Rutland (East Leicestershire). Skepmpton (1985) highlighted the role of the residual strength of clayey materials in the invetigation of landslides, whereas Castellanos et al. (2015) presented an extensive review of the role of fully softened shear strength in landslides and provided guidelines for use in practice. Harris et al. (2012) performed a set of back analyses of a slope failure using FEM considering the available rainfall data, in order to calibrate a numerical model that was used for the development of an early warning system for rainfall induced landslides. Zhao et al. (2015) describe an approach for the back analysis of landslides adopting reliability analysis methods that allows the estimation of the statistical distribution and the correlation of the geotechnical parameters.

Objective and Description of Procedure

The objective of the back analyses was the estimation of (a) the shear strength parameters of the weak upper marl formation and of the contact surface between the weak and the underlying competent marl, which represented the lower boundary of the landslide and (b) the groundwater conditions during failure. Some initial rational admissions were made, which were necessary to counterbalance the lack of geotechnical data, groundwater information and investigation on the landslide mechanism and sliding surface (e.g. through boreholes, inclinometers and piezomenters). In other words, it was necessary to assume some of the parameters of the problem, since the only site-specific quantitative information available was the occurrence of the landslide itself, i.e. the value of safety factor marginally below unity for the initial morphology. The initial assumptions specifically concerned:

- <u>Shear strength parameters</u>. Due to already progressing movements and displacements and the nature of the marl formation, it was assumed that along the total failure surface of the landslide, residual strength conditions were dominant. Therefore, cohesion (a) of the weak marl and (b) along the contact surface between the weak and the underlying compact marl was assumed to be zero. Hence, the analyses objective was limited to the estimation of friction angle for the weak marl and the contact surface. The specific values of the friction angle to be approached by the back analyses concern only the zones close to the total failure surface and any satellite or smaller scale failure surfaces within it (between the monolithic blocks that were detached) and do not correspond to "intact" parts of the marl formation that were part of the total mass that slid.

- <u>Failure surface geometry</u>. The failure surface shape seemed to vary between the sections considered at different parts of the landslide widthwise. This was obvious during on-site visits and was also mapped on the surveying sections of the area after the landslide. Therefore, the failure surfaces selected for the back analyses on each section were different with respect to the assumed range of their slope at the landslide crest. The sections considered are shown on the coal mine area plan view of Figure 2. Specifically, along Section 2, the failure surface that was revealed at the crest of the landslide had a sub-vertical slope. On the other hand, the crest in the case of Section 3 showed a much gentler sliding surface. Additionally, based on available geological information and mapping data from the engineers of the mine, the position of the contact surface between the weak and the underlying competent marl was assumed approximately 20 m above the roof of the coal layer (which was closely mapped from the exploitation investigation program) and parallel to it. Due to the process of sedimentation, contact surfaces between successive layers practically have the same slope and slope direction within an area of limited extent in a non-folded, post-alpine environment. In that context, the inclination of the slide at its crest, as well as its lower boundary along the weak contact surface were well defined from site information and landslide topographical mapping.

- <u>Underground water regime</u>. Due to the mechanism of water ingress in the mass that slid, through open cracks that gradually became wider and deeper, no uniform water table was considered and the water effect was taken into account via the pore water pressure ratio R_u.

The range of examined parameters in the back analyses is shown in Table 2. In the absence of available laboratory testing that could define a narrow range for the residual friction angle of the weak marl and the contact surface, a wide initial range was considered, based on the experience of the consulting team and the relevant results from the similar case in the adjacent coal mine (Prountzopoulos et al., 2010) that are given in Table 1. The examined range of the pore pressure ratio was the maximum feasible (R_u=0 corresponds to dry conditions whereas R_u=0.5 practically means water table up to the surface), as it was considered that no correspondence with previous experience could be assumed.

Table 2. Examined range of parameters' values during back analyses.

Parameter	Range of Values
Friction angle of weak marl φ_{marl}	15° - 25°
Friction angle of contact surface between weak and competent marl φ_{surf}	10° - 20°
Pore pressure ratio R_u	0 – 0.5

Given the three dimensional character of the landslide, as mentioned earlier, two analysis sections were systematically checked (Sections 2 and 3) so as to estimate some rational parameters for the assessment of rehabilitation proposals. Figure 5 illustrates the analysis sections along with their main features. The process that was adopted is summarized in the following steps:

1. After in-situ inspection and study of available surveying mapping, conclusions were reached concerning the general shape of the failure surface along Section 2 (subvertical near the crest) and Section 3 (with more gentle slope near the crest). Based on these conclusions, a set of possible failure surfaces was considered for each Section, with varying inclination and part of the surface passing through the weak marl formation and along the contact surface between the weak and the competent marl.

2. For each Section, the value of the safety factor was computed for the selected possible failure surfaces and for the range of geotechnical parameters listed in Table 1. Specifically, analyses were performed for 6 R_u ratio values (0-0.5, at steps of 0.1). For each R_u value, the safety factor was calculated for 5 values of φ_{marl} (15°, 18°, 20°, 22°, 25°), whereas for every analysis with constant φ_{marl} value, the value of φ_{surf} varied from 10° to 20°, using the sensitivity analysis tool offered in Slide software.

3. From every analysis, the minimum safety factor resulted for each combination of R_u, φ_{marl} and φ_{surf}.

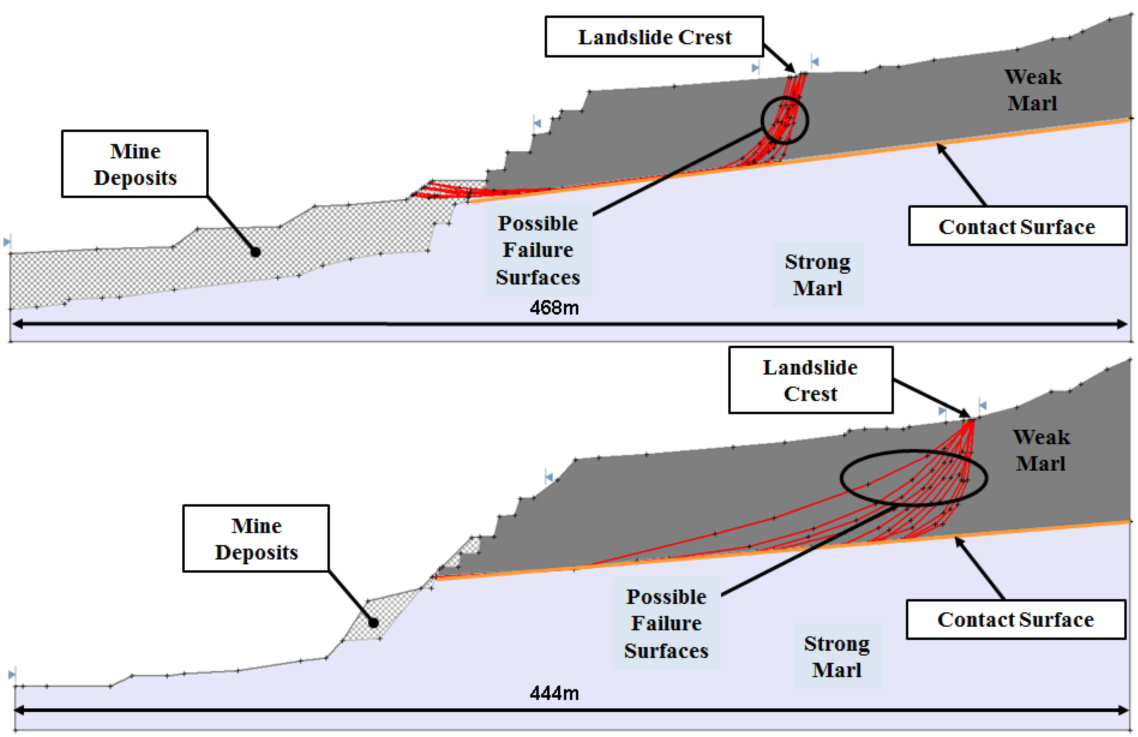

Figure 5. Analysis Section 2 (up) and 3 (down). Slide software model.

Analyses Results

The following figures (Figures 6 to 8) illustrate the results of back analyses performed for Section 2 and Section 3 and various values of R_u ratio. Examination of results led to the following remarks:

1. Section 3 yielded somehow higher values of the safety factor compared to Section 2 for the same geotechnical and groundwater conditions, which is attributed to the different geometry of these sections, both in terms of topography and failure surface geometry. The real (three dimensional) safety factor, when 2D analyses are used, can only be approached indirectly, as a weighted average of its values from various analysis sections (Cavounidis, 1987 & 1988). In that sense, the representative range of geotechnical parameters values should be such that results in safety factor values slightly more than 1.0 for Section 3 and slightly less than 1.0 for Section 2.

2. Values of the safety factor for values of pore pressure ratio R_u=0-0.1 generally lie above unity for both Sections analyzed (see Figure 6), especially for φ_{surf} >13° and were therefore excluded. This result was actually anticipated, since water seemed to have had a major role in the landslide triggering.

3. Values of the safety factor for values of the pore pressure ratio R_u=0.4-0.5 were generally well below unity for both analysis Sections (see Figure 8), especially for φ_{surf} <17°-18° and were not considered likely. Values of the safety factor for φ_{surf} >18°, given the nature of the specific surface and the fact that are assumed to refer to residual strength, were not considered reasonable.

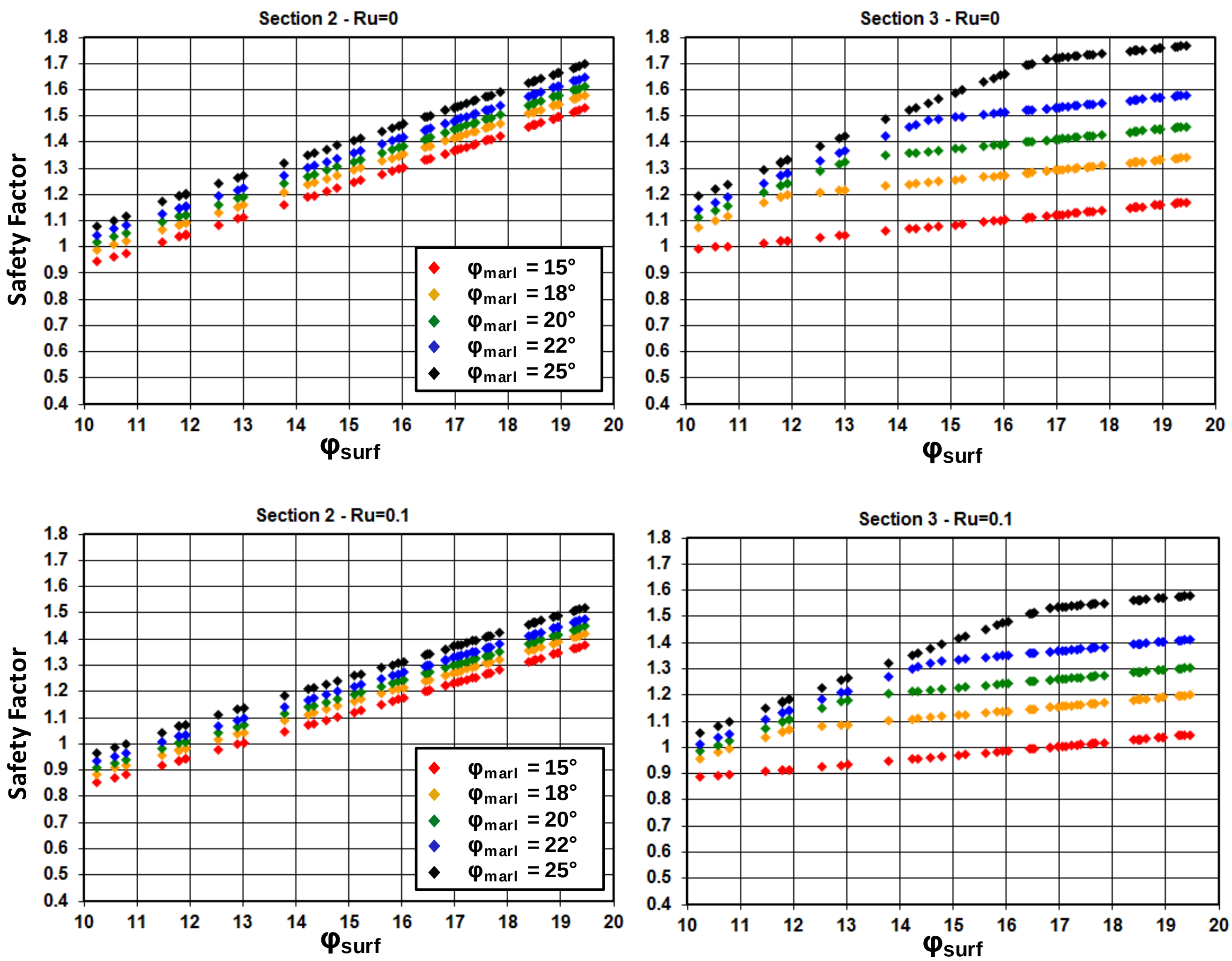

Figure 6. Variation of safety factor for different values of φ_{marl} and φ_{surf} for R_u=0.0 (up) and R_u=0.1(down), as computed for Section 2 (left) and Section 3 (right)

4. Due to the range of examined parameters, the surface yielding the minimum safety factor value per Section may vary. Therefore, the resulting curves showing the variation of the minimum safety factor value vs the variation of φ_{surf} for each value of φ_{marl} in Section 3 did not have a constant slope. Points of slope change mean transition from one critical surface to another. Specifically, the lower the slope of the curve of variation of the minimum safety factor value vs the variation of φ_{surf}, the less significant the effect of φ_{surf}, meaning that the major portion of the failure surface was through the weak upper marl. Given that the failure surface around Section 3 has a generally gentle slope and therefore a large part inside the weak marl formation, it is obvious from Figure 7 that values of φ_{surf} <13° are not possible, as they correspond to an increased slope of Safety Factor vs. φ_{surf} curves.

5. In-situ observations of the shape of the failure surface along Sections 2 & 3 and corresponding selection of possible failure surfaces during the back analyses were reflected on the resulting Safety Factor vs. φ_{surf} curves. Specifically, for Section 2, where the major part of the landslide was along the contact surface between the upper weak and the underlying rocky marl, the effect of φ_{marl} on the safety factor value is not significant. On the contrary, the results from Section 3, where a considerable part of the failure surface seemed to be included in the upper weak marl formation, show a quite significant effect of φ_{marl} on the computed safety factor.

6. Considering the value of φ_{marl}=15° marginal and not very possible, the sets of φ_{marl} - φ_{surf} values resulting for each case are presented in Table 3.

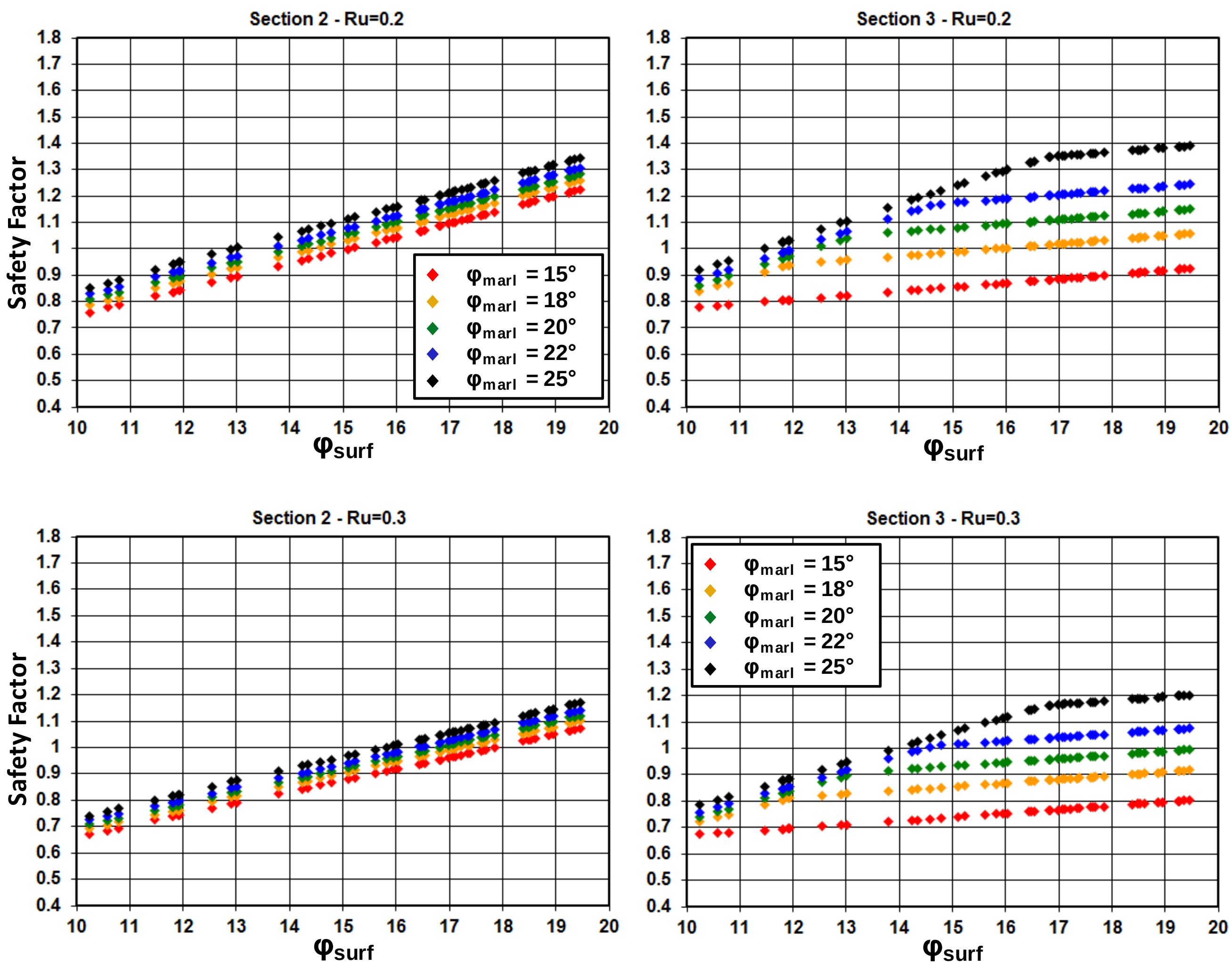

Figure 7. Variation of safety factor for different values of φ_{marl} and φ_{surf} for R_u=0.2 (up) and R_u=0.3(down), as computed for Section 2 (left) and Section 3 (right)

Table 3. Geotechnical parameters range from the back analyses.

Section	R_u ratio	φ_{surf}(°)	φ_{marl}(°)
2	0.2	13-15	18
	0.3	15-17	18-20
3	0.2	14-16	18-20
	0.3	15-16	22

Based on the available experience from similar geomaterials in the area, the following set of representative parameters was selected for the rehabilitation design of the landslide:

- Pore pressure ratio R_u=0.20-0.25
- Residual friction angle of the contact surface between weak and competent marl φ_{surf} =14°-15°
- Residual friction angle of the weak marl φ_{marl} =18°-20°

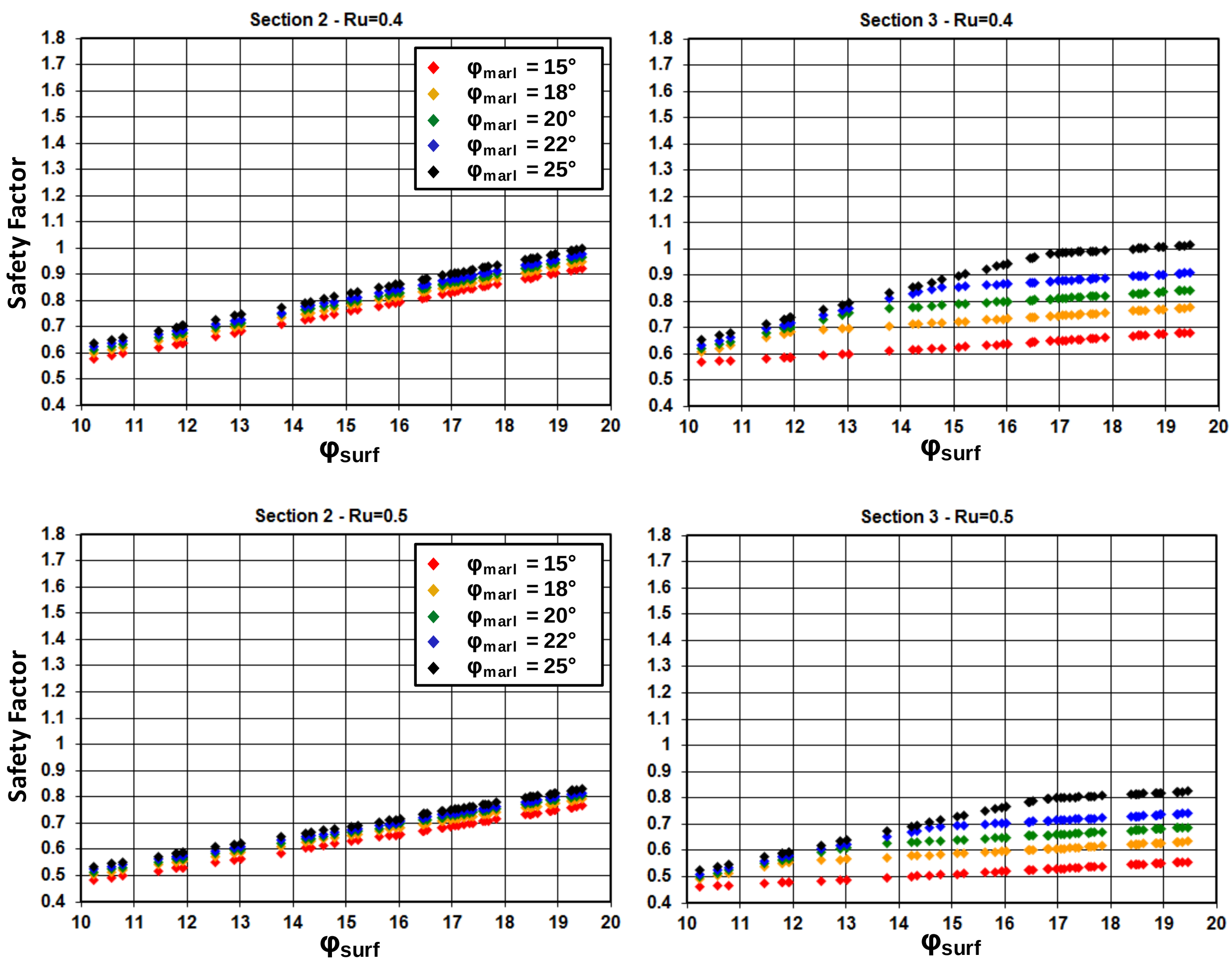

Figure 8. Variation of safety factor for different values of φ_{marl} and φ_{surf} for R_u=0.4 (up) and R_u=0.5(down), as computed for Section 2 (left) and Section 3 (right)

REHABILITATION PROPOSAL AND STABILITY CHECKS

Evaluation of Landslide Stability

The existing (before any intervention) situation of the landslide in terms of stability was initially assessed, using the geotechnical and groundwater parameters resulting from the back analyses (R_u=0.25, φ_{surf}=14° and φ_{marl}=19°). Use of values that were generally on the conservative side within the resulting range was considered necessary, so as to counterbalance the lack of proper geotechnical and groundwater investigation. Corresponding analysis models from Slide software are illustrated in the figures below.

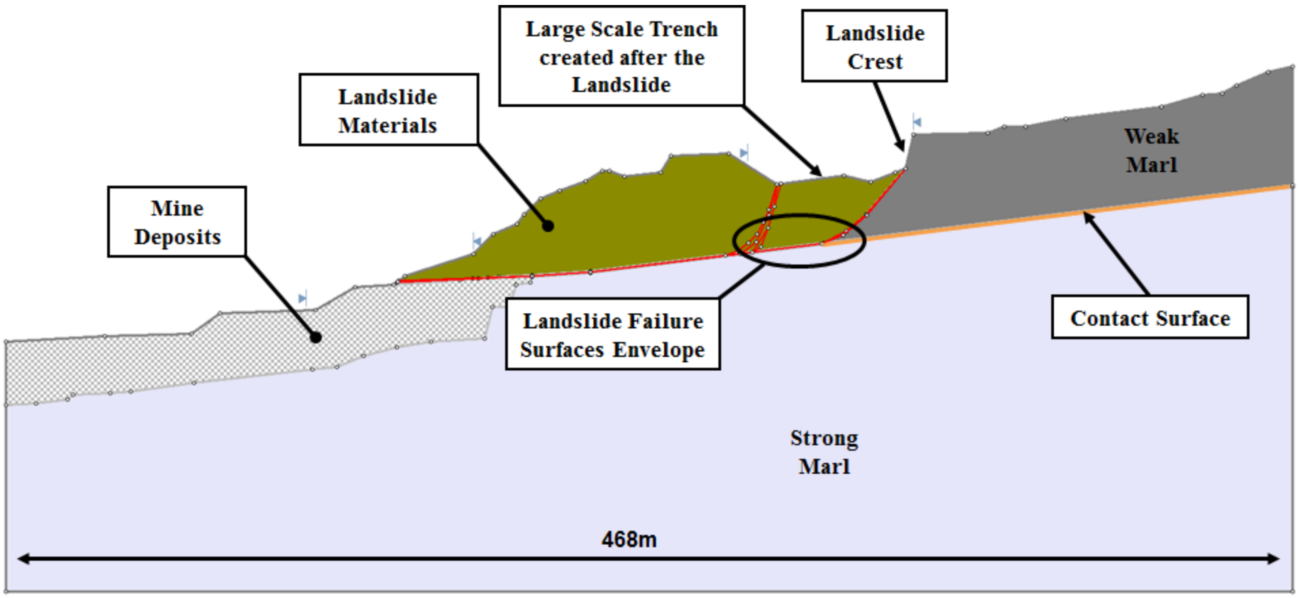

Figure 9. Analysis Section 2 after the landslide

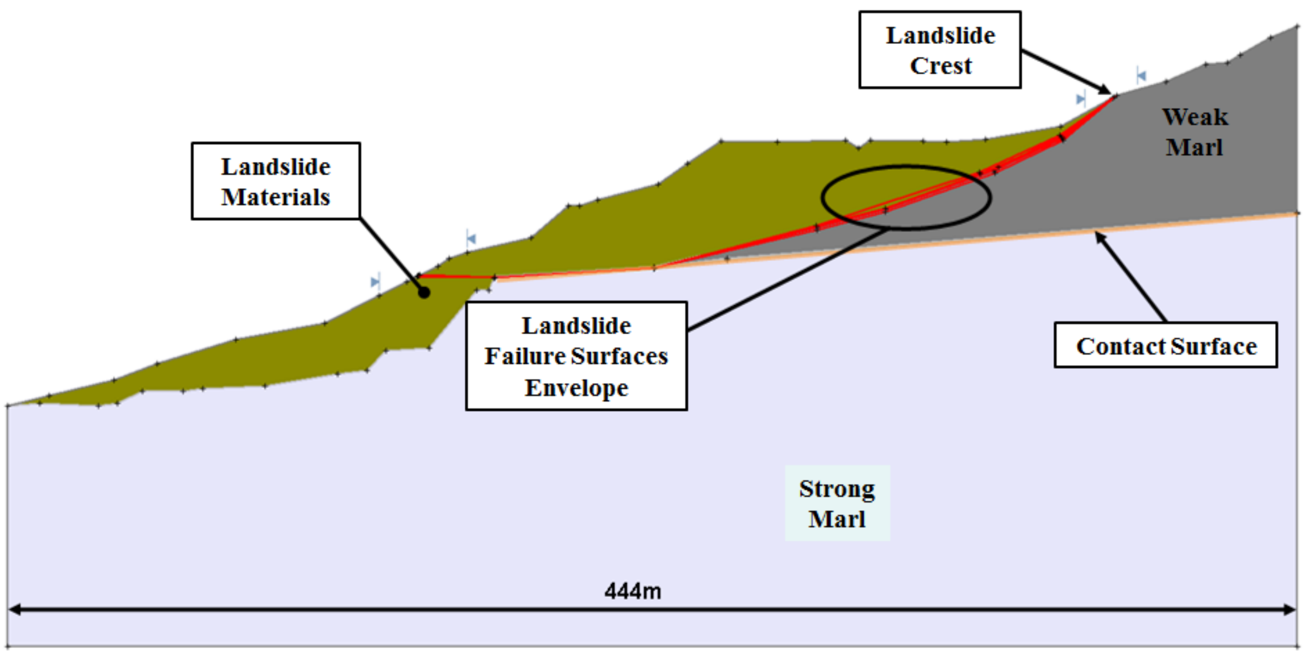

Figure 10. Analysis Section 3 after the landslide

To evaluate the stability conditions of the landslide before any intervention, it was assumed that the critical failure surfaces either pass through the total failure envelope of the event, or through zones where significant shearing and relative movements were observed between “intact” parts of the marl formation, such as the zone downslope of the large scale trench that is shown in Figure 4.

The relevant analyses yielded results of the safety factor between 1.06 and 1.11, which were considered marginal, given the possible further deterioration of the shear strength of the materials involved in the slides, due to ongoing movements, especially during the rainy season in the area. Specifically, analyses of Section 2 resulted in a minimum value of the safety factor equal to 1.11, whereas the corresponding minimum value from Section 3 was 1.06. Analyses with free selection of surfaces by the software were not performed, as these would yield unrealistically low values of the safety factor (considerably <1), under the assumption that the total mass included in the landslide envelope had been downgraded to its residual strength (zero cohesion and low friction), which was not the case due to monolithic, undisturbed blocks existing within the slid mass. The fact that the resulting safety factor values were rational, i.e. marginally larger than unity, indicating a temporarily stable situation after a main landslide event, enhanced our confidence in the assumptions that were made.

Rehabilitation Proposal

Due to the magnitude and the location of the landslide (inside a coal mine), rehabilitation had to be of purely geotechnical nature, i.e. to be based on rearrangement and deposition of soil masses for the enhancement of the total stability of the slid mass. Therefore, placement of non-commercial deposits was proposed at the toe of the landslide. Configuration of the deposits for landslide retaining action was with slope inclination of 2:3 (v:h), slope height of 12 m and 10 m wide benches. The vertical thickness of the deposits at the central part of the landslide had to exceed 15 m, especially around the level of the weak contact surface between the superficial weathered and the underlying competent marl, which suggested the major lower limit of the landslide, so that the deposits could really act as a counterweight and not only as a surficial cover at the toe of the landslide.

The design of the deposition configuration had to serve an additional purpose, other than the stabilization of the landslide. As the event took place within the exploitation limits of the coal mine (the road that was drifted was to be deviated by a road already under construction uphill – marked in green in Figure 2), landslide materials needed to be carefully and safely removed, so that the exploitation could extend to the North. Therefore, the proposed deposits would have to serve the purpose of access to the main body of the landslide (which was not possible from the crest) so that removal of materials could commence, specifically from the large monolithic block shown in Figure 4a and 4d. This requirement imposed that the maximum elevation of the deposits would have to be (at least locally) approx. 35 m above the contact surface between the weak and the strong marl, suggesting an absolute elevation of approx. +620 m).

Shear Strength Parameters of Counterweight

The proposed rehabilitation configuration primarily had to ensure the stability of the counterweight deposits. As these would of course be constructed from non commercial, marly products of the mine, estimation of the corresponding shear strength parameters was based on back analyses of the existing slopes in the north part of the mine (see upper right image of Figure 3), created from the same materials with the same deposition and compaction method. These existing non commercial deposits, during stoppage of works, showed no sign of instability or time dependent deformational response, even during the excessive rainfalls that preceded the landslide. Aiming to a conservative aproach, a partial slope of approx. 30m height and 36° inclination was selected for the back analyses and the value of the safety factor was set to slightly >1.0. Upstream and downstream of this slope, wide benches had been graded so that a potential formation of a deeper slip surface would have to involve the marly “bedrock”, which was not considered possible.

Some assumptions were again necessary, since the number of unknowns (cohesion c, friction angle φ and R_u ratio of deposits) was high compared to the only available “equation” (safety factor close to unity). Due to the marly nature of the deposits, the good quality of compaction and the absence of open cracks on the slope surface, R_u was assumed to be zero. Friction angle of this material is expected to generally lie below 30°, whereas for well compacted deposits unit weight was assumed equal to γ=20kN/m^3. Based on these assumptions and the conducted back analysis, a representative set of shear strength parameters, i.e. cohesion c=15kPa and φ=28° was selected. Geometry of designed counterweight was such that stress levels within the deposits were not expected to significantly differ from the corresponding stress levels in the back analysed slope. Therefore, the above mentioned representative parameters could be used for the stability analysis of the proposed rehabilitation scheme.

Proposed Rehabilitation Stability Checks

Stability analyses were performed to quantify the increase of the safety factor due to deposition – retaining action at the toe area of the landslide. The required height of the retaining deposits, i.e. the final retaining deposition elevation, was parametrically examined. As already mentioned, the vertical thickness of the deposits was always considered more than 15 m. The geotechnical and groundwater parameters that resulted from the back analyses (R_u=0.25, φ_{surf}=14° and φ_{marl}=19°) were used for the stability verification analyses. Increased value of the pore pressure ratio R_u to account for possible more unfavourable groundwater conditions was not considered necessary during stability checks. The landslide took place after a very heavy rainfall of exceptional duration for the area. Therefore, being a temporary configuration (once access would be possible to the body of the landslide, the landslide materials would be removed), the proposed rehabilitation proposal was considered adequate and indeed somehow conservative, since works started during spring and were completed before the next rainy season.

The parametric analysis results showed that deposition of non commercial materials against the landslide toe and up to an elevation of approximately 25 m above the contact surface between the weak and the competent marl, corresponding to an absolute elevation of +610 to +615 m, was adequate for the stabilization of the landslide mass. Only locally, for access reasons to the main mass of the landslide from the east, the upper elevation of the stabilization deposits had to be increased by 10-15 m (absolute elevation of +620-625 m). Indicative analysis results for Section 3 and different final levels of stabilization deposition, are illustrated in the following figures.

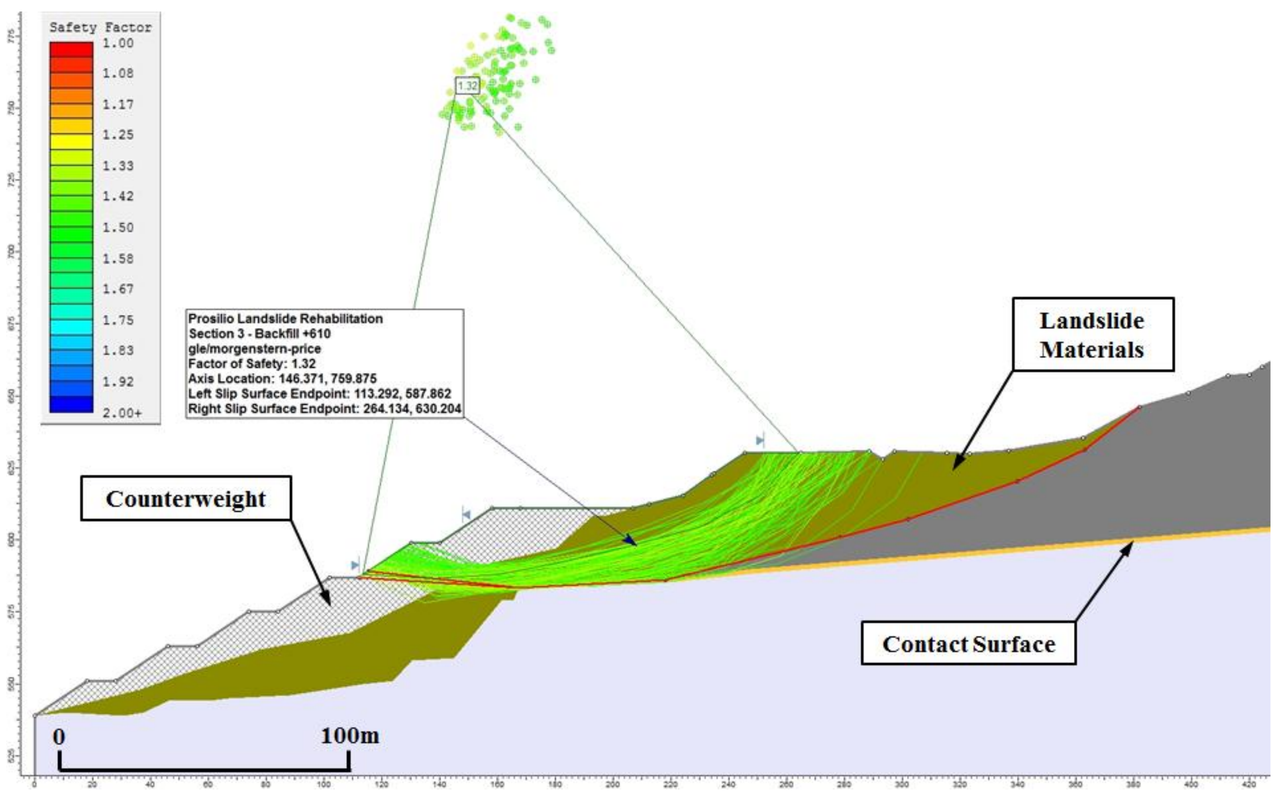

Figure 11. Analysis Results for Section 3 with backfill at the toe up to elevation +610 (F.S. = 1.32)

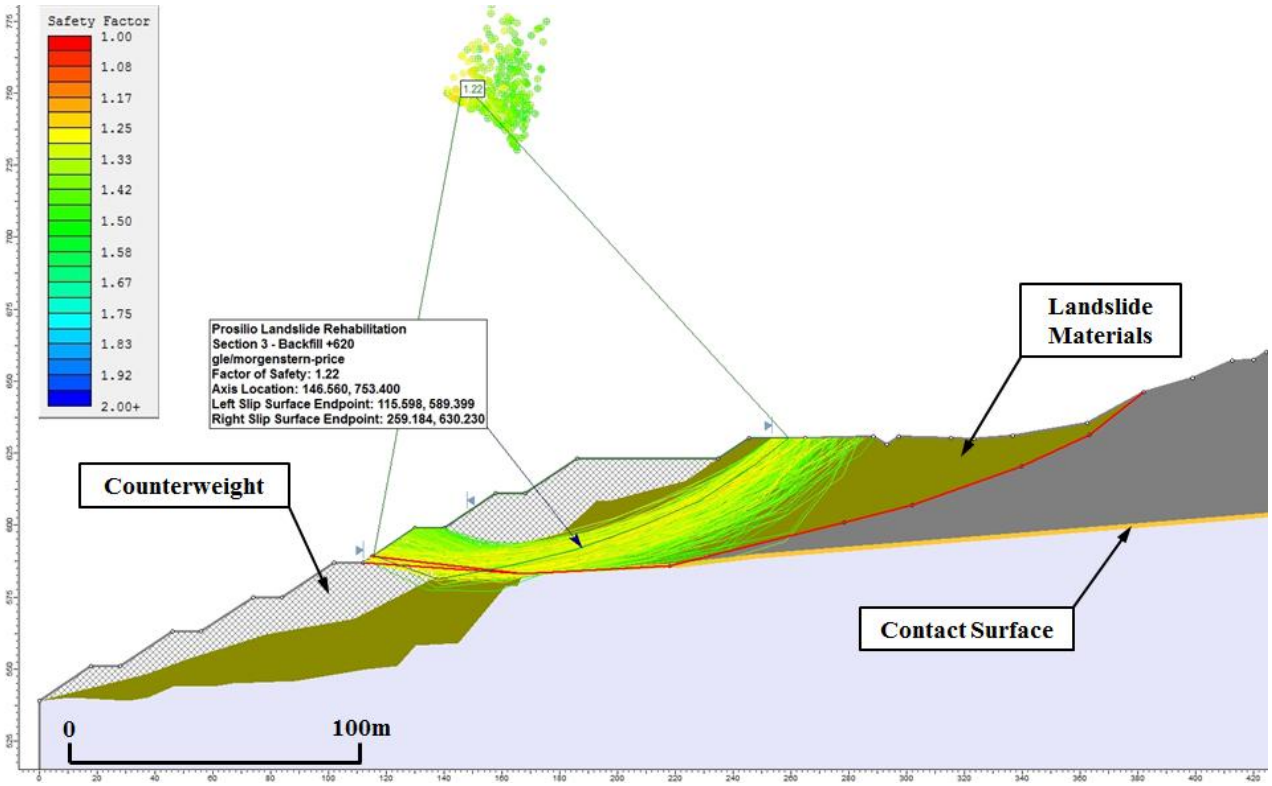

Figure 12. Analysis Results for Section 3 with backfill at the toe up to elevation +620 (F.S. = 1.22)

Summarizing, analysis results showed that:

1. Placement of non commercial deposits at the toe of the landslide up to an elevation approximately 25 m above the contact surface between the upper weak and the undelying competent marl (~+610 m absolute evelation), significantly increased the safety factor. Specifically, considering the envelope of the landslide or surfaces along obvious large scale movements within the total landslide mass as potential failure surfaces, the computed values of the safety factor ranged between 1.52 and 1.63. Assuming that any failure surface within the total landslide envelope was possible (a quite conservative approach, since residual shear strength parameters were assigned in the analysis model) the safety factor ranged between 1.32 and 1.56. These values, even with the second, conservative approach, were considered adequate.

2. Placement of non commercial deposits at the toe of the landslide up to an elevation approximately 35 m above the contact surface between the upper weak and the undelying competent marl (~+620 m absolute evelation), did not significantly differentiate the safety factor. Specifically, considering the envelope of the landslide or surfaces along obvious large scale movements within the total landslide mass as potential failure surfaces, the computed values of the safety factor ranged between 1.56 and 1.70. Assuming that any failure surface within the total landslide envelope was possible, the safety factor ranged between 1.22 and 1.57. Therefore, it was shown that the additional placement up to the specific level was only to facilitate the landslide removal and exploitation procedures and was not necessary for the stabilization of the landslide. In fact, locally, due to the existing geometry and morphology of the landslide, this additional placement could somehow reduce the available safety factor, as part of the deposits would actually increase the sliding moment for certain potential failure surfaces (pose additional loading to the landslide - see Figure 12 above).

The proposed rehabilitation scheme was succesfully applied in order to stabilize the landslide before any intervention to its mass. The operation of the coal mine has continued without significant instability incidents ever since. The following figures show an aerial photo of the coal mine area and an aspect of the same from within the mine area, after completion of landslide counterweight construction and leveling of the landslide materials.

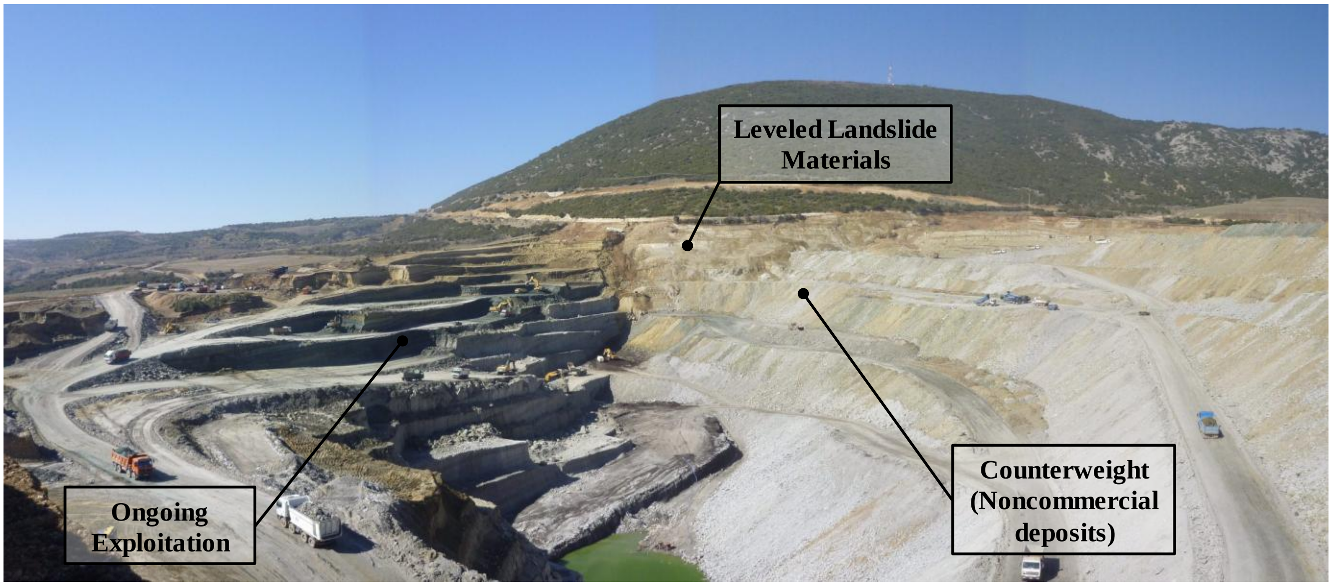

Figure 13. Aspect of the coal mine area after completion of counterweight construction and leveling works (March 2012).

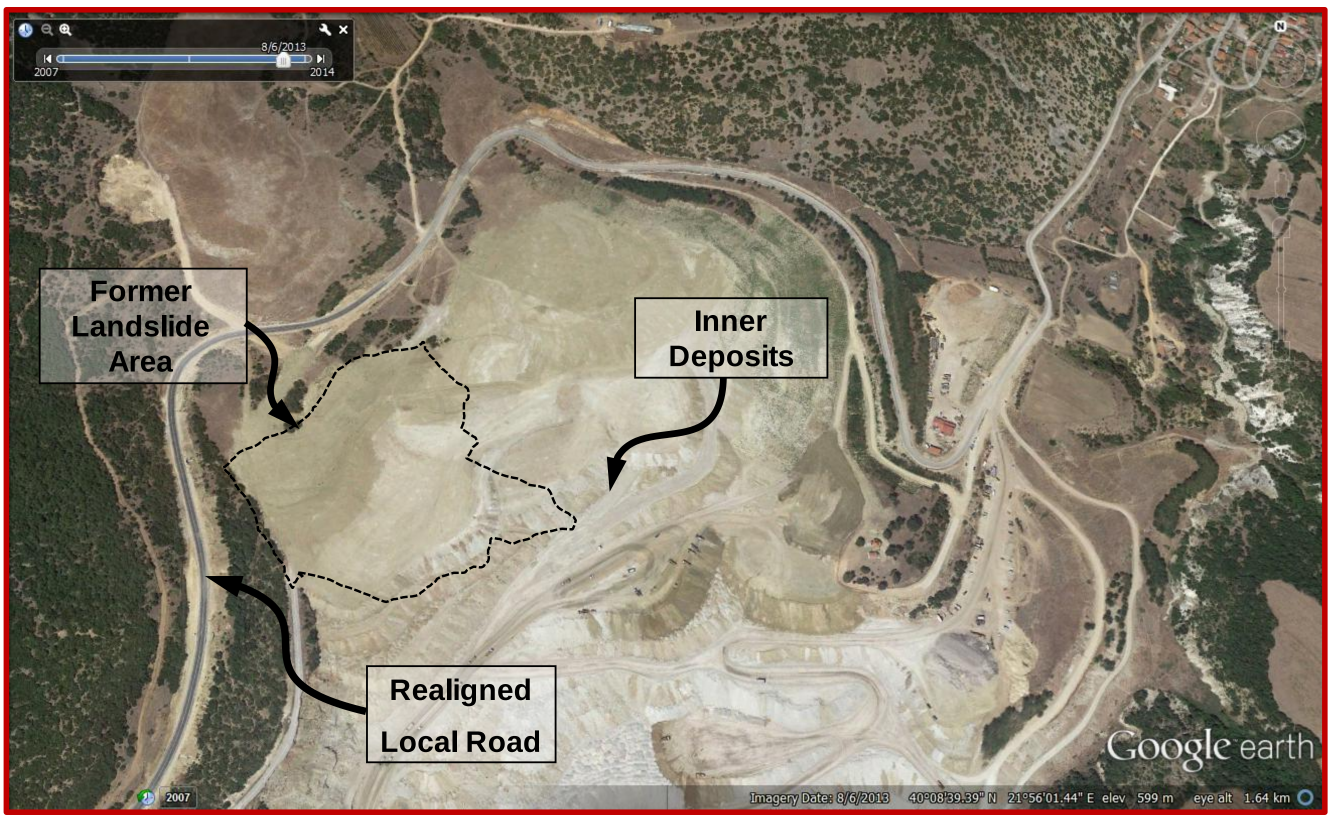

Figure 14. Aerial photo of the coal mine area after completion of counterweight construction and leveling works.

CONCLUSION – LESSONS LEARNED

A large scale landslide within marly formations in a coal mine in northern Greece is discussed in the present paper. The landslide has been a combination of a typical failure through a weathered soil-like mass and a sliding along a weak, pre-existing plane, whereas its manifestation was a result of numerous factors and parameters. Evaluation and study of all relevant data and not only of the geotechnical and groundwater conditions in the area and time of the event, was necessary. Historical data on the condition of slopes, the sequence of works and the period of their stoppage, as well as observations of instability-related features from people involved with the project were valuable contributions to the understanding of the failure mechanism. The timing of the final, large scale landslide event, was preceded by a long period of stoppage of works due to relevant adjudication. These works however involved actions for the stabilization of the slopes that finally failed. This case history highlights the importance of, and need for, a technical evaluation of the long term stability conditions of a project, whose results have to be accounted for before any decision on its stoppage.

The framework for the provision of a solution for rehabilitation of the landslide actually imposed the necessity to deviate from the conventional evaluation approach. Available time was very limited and project conditions (time and budget, availability of appropriate geotechical investigation agencies in the area) could not allow for a timely executed and adequate geotechnical investigation. Therefore, landslide evaluation had to be practically based on (a) only minor already available geotechnical data, (b) experience and evaluation results from similar cases in the wider area of the event, (c) in-situ observations of geomaterial response and landslide characteristics and (d) existing references in the bibliography. In such cases, a rationally increased conservatism has to be applied during evaluation and design of rehabilitation, to counterbalance the lack of adequate geotechnical information. Logical assumptions are necessary, since the number of unknowns is significantly higher than the “available equations”. However, assumptions have to be based on careful in-situ observations, generally lie on the conservative side, and be reflected in the back analyses results, both qualitatively and quantitatively.

The critical geotechnical and groundwater parameters of the problem (the residual shear strength of the surficial, weathered marl formation and its contact surface with the underlying strong, rocky marl and the pore pressure ratio R_u) were approached via back analyses, under time pressure for stabilization action. The “unknown” parameters to be aproached by the back analyses were multiple and therefore rational assumptions and simplifications, as well as careful examination of the landslide’s failure mechanism were necessary in order to maximize the benefit from the only geotechnical “equation” (Factor of safety marginally below unity at the time of the event). The evaluation of the back analyses results had to be combinatorial in order to limit the range of representative parameters. Consideration of in-situ observation of failure surface geometry in specific sections of the landslide widthwise, especially the part of the failure surface within (a) the surficial marl formation and (b) its contact surface with the lower rocky marl, assisted the elimination of possible strength and groundwater parameters’ values. The existing experience from similar, albeit of smaller scale, landslide events in similar materials and indeed in the wider area of the event, assisted the selection of the initial range of parameters to be parametrically examined via stability analyses.

Results of the analyses were in general agreement with the in-situ observations and the basic geotechnical assumptions, e.g. the water had a major role in the event, however this was through open cracks in the generally impermeable marl formation and not in the form of a continuous water table. Consideration of more than one analysis sections of the landslide widthwise has been helpful in order to estimate the 3-dimensional safety factor and validate the analyses results and assumptions. These results were used to propose a geotechnical rehabilitation scheme with placement of a counterweight as retaining action on the toe of the landslide. The geometry and dimensions of the counterweight resulted from the co-evaluation of geotechnical and constructional parameters, in order to achieve an acceptable level of stability conditions and facilitate access to the landslide area and continuation of exploitation. Attention had to be paid so that part of the counterweight deposits did not pose additional loading to the landslide in order to provide access to its main mass for leveling and exploitation advance works. Application of the proposed solution led to the stabilization of the landslide and the proper mine operation.

ACKNOWLEDGMENTS

The authors would like to acknowledge the company ME.T.E. S.A, for its cooperation and for providing all the necessary data for the present paper.

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