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Groundwater Conditions and Drainage of a Large Slide

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ABSTRACT

A large slide of great complexity, occurred on February 18, 1995 near the village of Malakasa, 36 km North of Athens, and disrupted road and rail traffic between Athens and Thessaloniki. The material in which the slide occurred can be described as a chaotic mixture of schists and breccias which have been highly disturbed by tectonic events, by ancient landslide activity and the prevailing high groundwater pressures. An understanding of the groundwater conditions in the Malakasa landslide was essential in designing remedial measures to stabilize the slide. The presence of a permanent spring and high water levels measured near the toe of the slide affects the sliding conditions. To better evaluate the impact of the groundwater on the overall stability of the slide, this paper presents the results of an in depth 3D study of the groundwater flow in the vicinity of the rupture surface and examines several draining options at the toe of the slide.

KEYWORDS

Slide, Groundwater, Pumping wells, Pumping schedule, rupture surface.

INTRODUCTION

This report presents the outcome of a study of the prevailing groundwater conditions and possible drainage alternatives at the Malakasa landslide. The material in which the slide occurred can be described as a chaotic mixture of schists and breccias highly disturbed by ancient landslide activity. A continuous failure surface now exists in this slide mass and is exposed in the lower part of the slope as shown in Figure 1. Because of the presence of the hard angular rock fragments, the overall residual friction angle for the entire failure surface is likely to lie in the range of 12 to 20° but friction will, without any doubt, depend on the prevailing water pressures. Therefore an understanding of the groundwater conditions in the Malakasa landslide is essential in designing remedial measures to stabilize the slide. The presence of a permanent spring and high water levels measured near the toe of the slide confirmed that the water pressures acting on the rupture surface near the toe were high affecting the sliding conditions.

Figure 1. Aerial view of the landslide
To better evaluate the impact of the groundwater on the overall stability of the slide, the present paper focuses on the modeling aspect of the groundwater flow in the vicinity of the rupture surface and examines several draining options at the toe of the slide.

All data in this report are taken from the site investigations and monitoring activities performed by Geomechaniki Ltd. Edafos Ltd. The EIS/GWM platform was used to quantify the various natural processes prevailing at the site.

OBJECTIVES

The objectives of the study were the following:

- Review groundwater monitoring results from 10/23/1996 to 1/9/96 and from 1/9/96 to 3/4/1996. (Reports provided by Geomechaniki Ltd, Edafos Ltd, and Gamma 4 Ltd.)
- Design and setup 3-D simulation models under the EIS/GWM platform to investigate different flow parameters (1/Loading conditions due to incoming groundwater flow up-gradient of the slide, 2/ Influence of rainfall conditions, and 3/ Local variation of hydraulic conductivities due to inhomogeneous geologic formations.)
- Examine several draining options. (Option 1: Single horizontal drain at the toe of the slide, Option 2: Two horizontal drains in the horizontal plane, and Option 3: Two horizontal drains in the vertical plane)

BACKGROUND ON LANDSLIDE

The following sequence of events was provided as background information on the development of the slide:

1. April-May, 1993: Excavations were carried out for the widening of the Athens Thessaloniki highway. A north-facing 1:1 excavated slope face was created as part of these excavations.
2. May, 1993: Local failure of this slope occurred immediately after completion of the excavation.
3. Site investigations were not carried out and, on the assumption that the failure was a first-time slide, remedial measures were designed. These consisted of flattening the slope and providing drainage ditches above the slope.
4. These remedial measures were only partially applied. The Contractor flattened the slope to 1:2 and constructed a toe wall.
5. January, 1994: During a period of continuous heavy rain and snow, a second failure of the excavated slope occurred. A number of cracks occurred behind the crest of the slope, the toe wall cracked and was slightly displaced and some heaving of the pavement and gutter occurred.
6. January-March, 1994: A program of site investigations was carried out and designs for remedial measures were prepared.
7. March, 1994: The designs for the remedial measures were submitted. These included flattening of the cut slopes (2:3 and 1:2) with a 4 m intermediate step and the improvement of drainage measures. In addition, a row of 12-14 m long, 1 m diameter reinforced concrete piles were placed a 3 m center to center at the toe of the slope, behind the toe wall.
8. November to December, 1994: Excavation for flattening the slope and construction of the piles was carried out.
9. February 18, 1995: During the early hours of the morning, a large landslide occurred, disrupting both the highway and the Athens-Thessaloniki dual railway line. There had been no rain in the period immediately preceding the failure. The slide extended approximately 350 m up-slope and was approximately 300 m wide at its widest point.

GEOTECHNICAL CONDITIONS CONTROLLING THE SLIDE

The material in which the slide occurred can be described as a chaotic mixture of schists and breccias which have been highly disturbed by tectonic events and by ancient landslide activity. An outcrop in which intense folding and juxtaposition of different material was evident in many locations of the site. This was also observed in many drilling cores in which sections of breccia were interspersed with sections of good quality limestone and very weak black schist. A question that was raised was whether or not a continuous failure surface existed in this slope before the 1995 failure and whether such a surface could have been detected by a site investigation program.

The site evidence shows that it is probable that such a surface or at least a substantial part of such a surface
did exist. This surface could have been caused by a much earlier episode of landslide activity which produced the slopes which, until the early 1990s, have shown no significant signs of instability. However, a full topological description of this pre-existing surface was impossible because of the nature of the recovered materials. Indeed the core recovery was difficult and the interpretation of the core logs was complicated by the chaotic orientation of the structural features and the mix of materials. We must also point out that the known history of the slope and the fact that the railway operated successfully across the slope for 130 years would not have given any reason to suspect the presence of a pre-existing failure surface.

What was beyond any doubt was the presence of a continuous failure surface in this slide mass. This rupture surface, was exposed in some places close to the toe of the landslide. A clay gouge collected from the rupture surface gave residual friction angles as low as $8.5^\circ$ in laboratory tests. Because of the presence of the hard angular rock fragments, the overall residual friction angle for the entire failure surface is likely to lie in the range of 12 to $20^\circ$ and is greatly affected by the water pressures at the level of the failure surface. Figure 2. shows the thickness of the sliding mass across the site, and in the background the contours of the failure surface.

![Figure 2. Thickness of landslide mass.](image)

These distributions were obtained using a generalized kriging procedure and clearly delineated the zone where water pressures govern the slide stability. As it can be observed this zone is located close to the slide toe and requires an in depth study of the water pressure variations.

**GROUNDWATER CONDITIONS IN THE SLOPE**

An understanding of the groundwater conditions in the Malakasa landslide was essential in attempting to interpret the causes of failure and in designing remedial measures to stabilize the slide. The presence of a permanent spring and a high water level in a well, both near the toe of the slide as indicated in Figure 3, confirmed that there was an abundance of water in the slide mass.

![Figure 3. Observed groundwater pressures.](image)

Evidence of the level of the water table was obtained from an examination of the records of the depth at which water was encountered in site investigation boreholes drilled at various location around the perimeter of the slide. These water levels are illustrated in Figure 3 indicating that the water table is generally 5 to 20 m below the surface, above the failure surface. There is an indication that some local draw-down of the water table may exist along the water supply tunnel which was constructed in the 1940s. This is not surprising since the construction damage to the rock mass surrounding the tunnel creates a zone of higher permeability along the tunnel. There is also evidence to an upper 'perched' water table in the slide mass and a lower water table in the bed rock much deeper than the failure surface.

A number of simple standpipe piezometers were installed during the early site investigation complemented by several seated Casagrande type piezometers installed across the landslide, providing important information about the water pressures at different depths in the slide. These piezometers were sealed Casagrande type, with one piezometer per hole, providing the most reliable information for the design.
of a drainage system and for long term monitoring of drainage performance. Note that the locations of greatest interest were those adjacent to the slide failure surface since it is the water pressures acting on this surface that have the greatest impact on slope stability. Hydraulic conductivities were determined by in situ tests on these piezometers. Figure 4 shows the distribution of the hydraulic conductivities in the vicinity of the failure surface. Different measurements were obtained with depth and properly implemented in the 3D groundwater model.

Figure 4. Hydraulic conductivities close to the failure surface.

A number of additional tests were be carried out at different locations in the slide mass. These tests were an extremely important component of the general effort to understand the groundwater conditions in the slope. They will indicate how long drainage may take and how effective it will be and what drainage system is required taking into consideration the prevailing groundwater storage conditions.

Indeed storage changes in a saturated aquifer system under conditions of decreasing water levels result from three primary processes: (1) the draining of pore spaces as the water table declines with the installation of the pumping system; and (2) the compression of the aquifer skeleton caused by increasing effective stresses in the aquifer below the water table. The site conditions clearly indicated the presence of layers of compressible clayey beds susceptible to inelastic compaction. Storage changes caused by the draining of the pore spaces or by inelastic compaction must properly accounted for in the 3D modeling of the landslide mass.

**THE EIS/MIFLOW MODEL**

The standard ground-water flow model accounts for changes in storage caused by water-table fluctuations, elastic compression of the aquifer skeleton, and expansion of water. To simulate inelastic compaction one needs to update the storage values over time. This is done automatically in the present algorithm by accounting for storage changes from inelastic compaction of interbeds within an aquifer and calculating the resultant land subsidence during the model simulation.

The general form of the flow equation is (MiFlow, 1992)

\[
\frac{\partial}{\partial t} \left( K \frac{\partial h}{\partial t} \right) + \frac{\partial}{\partial x} \left( K_{xx} \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_{yy} \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( K_{zz} \frac{\partial h}{\partial z} \right) - W = S_N \frac{\partial h}{\partial t}
\]

where

- \( h \) is hydraulic head [L];
- \( W \) is volumetric flux per unit volume of sources and (or) sinks of water [T\(^{-1}\)];
- \( S_N \) is specific storage of aquifer material [L\(^{-1}\)]; and
- \( t \) is time [T].

The term on the right-hand side of this equation describes the flow rate of water into storage in the aquifer per unit volume. The storage term can vary over the modeled area depending on the storage properties of the aquifer materials but the values are assumed constant in time, except for the case where a confined aquifer becomes water table or vice versa. To account for changes in storage caused by compaction of fine-grained interbeds, an additional term must be added to the ground-water flow equation. Assuming that changes in head result in equal but opposite changes in effective stress, the term can be expressed as:

\[
q_t = S_{sk} \frac{\partial h}{\partial t}
\]

where

- \( q_t \) is rate of flow per unit volume of water flowing into storage in compressible interbeds [T\(^{-1}\)] and
is skeletal component of specific storage of interbeds a function of previous maximum effective stress [L\(^{-1}\)].

Equation 2 represents a volume average flow into or out of storage per unit volume of interbeds. The skeletal specific storage value in equation 2 varies between an elastic and an inelastic value depending on the relation of the head in the cell to the preconsolidation head. The preconsolidation head corresponds to the previous maximum effective stress (previous lowest head). The term \(S_{sk} \)' is the elastic skeletal specific storage whenever the head in a cell is greater than the preconsolidation head. The term is the inelastic skeletal specific storage whenever the head in a cell is less than the preconsolidation head. The preconsolidation head can also change during the simulation as it is assigned the most recent lowest head value.

For an aquifer with multiple interbeds (as it is the case of the studied landslide) of differing specific-storage values, a single elastic and a single inelastic skeletal specific storage value can be determined to account for changes in storage in all the interbeds. For a system with \(n\) interbeds with specific-storage values \(S_{s1}, S_{s2}, ..., S_{sn}\) and with thicknesses \(b1, b2, ..., bn\), a single equivalent specific-storage value, \(S_{ssystem}\) is given by (Jorgenson 1980, equation 67):

\[
S_{ssystem} = \frac{S_{s1}b_1 + S_{s2}b_2 + ... + S_{sn}b_n}{b_1 + b_2 + ... + b_n} \quad (3)
\]

Individual thicknesses and elastic skeletal specific-storage values of the interbeds are combined by equation 3 to compute a single elastic specific-storage value for the \(S_{sk} \)' function. Similarly, the thicknesses and inelastic specific-storage values can be combined by equation 3 to compute a single inelastic specific storage value.

EIS/MiFlow uses dimensionless storage-coefficient values for each layer. The value for a confined layer is the specific storage assigned to a model cell multiplied by the thickness of the cell. The storage coefficient is then multiplied by the cell area in the program to create a Storage capacity. The storage-capacity value for each cell is used in the basic finite-difference equation for ground-water flow. The program described in this report also requires specification of the storage coefficient. Storage coefficients for individual layers of sediments can be combined into an equivalent value in a manner analogous to combining specific-storage values in equation 3. The expression for computing the equivalent storage coefficient, \(S_{system}\) is given by Jorgenson (1980, equation 67) as

\[
S_{system} = S_{s1}b_1 + S_{s2}b_2 + ... + S_{sn}b_n \quad \ldots (4)
\]

The skeletal storage coefficients (elastic and inelastic) are used to estimate elastic and inelastic components of subsidence in addition to the flow of water into and out of storage. The dimensionless storage coefficient is the constant of proportionality between compaction and head change. This proportionality is used in the calculation of storage changes and compaction in the computer program.

Model Calibration

Several runs were needed to calibrate the model and reproduce the groundwater levels of the recorded pumping tests. Figure 5 illustrates the 3D results.

Figure 5. Results of the modeling calibration

The discrepancy within the landslide limits were smaller than a 1 meter, 7 days after the start of
pumping tests. These results were considered satisfactory to further pursue the study of an optimum drainage system.

STUDY FINDINGS

At this stage several draining systems of the landslide were investigated with the following findings:

- The simulated groundwater flow regime exhibits an interesting pattern. The water levels above the surface of rupture of the landslide decrease with time as the pumping wells control the flow. This is true for the eastern portion of the slide where hydraulic conductivities are high. Water levels drop from 20 m to an average of 12-15 meters above the rupture surface with an increase of the factor of safety against sliding (See Figure 5).

- This pattern is different at the west side of the landslide where the water levels slightly increase due probably to the low hydraulic conductivities prevailing at this location. However, the water levels do not exceed 15 m and are below the high water levels of 21 m observed in November of 1995.

- The rain clearly affects the water level above the surface of rupture. The main loading mechanism due to the rain is included in the model through the water levels measured upwards of the site. These effects constitute a real hazard for the slope stability. They will probably increase the water pressures at the rupture surface near the toe of the slope.

- The compressibility of the layered soil medium due to pumping favorably affects the overall stability reducing in the long run the water pressures.

- This hazard can be considerably reduced by placing horizontal drains near the toe of the slope thus reducing the potentially dangerous water pressures. Figure 6 shows the drawdawn due to the presence of the horizontal drain. The water level will essentially drop at the elevation of the horizontal drain.

- The configuration with two horizontal drains gives the best results in terms of adequately reducing the water levels on the surface of rupture. Ideally these drains should be placed one meter above the existing surface of rupture.

Figure 6. Effects of horizontal drains

REFERENCES

Geomechaniki Ltd, Edafos Ltd., and Gamma 4 Ltd, February 1996, "Evaluation of the Monitoring Results of the Temporary Vertical Drainage System for the stabilization of the Malakasa Landslide".


