Back Analysis and Solutions of Slope Instability in Algeria: Case Study

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ABSTRACT: During the construction of a pumping station in Elbouira Zone, a landslide was observed due to the high cut slope in the weak clay deposits over sand base. The slip surface appears behind the station as cracks and creep towards the station. Back analysis using FEM was run to calculate the horizontal force required for the slope stability and optimizing a vertical piling solution. This paper documents the investigation conducted to understand the ground conditions and the unstable surfaces shapes, the analysis to identify the potential reasons for the instability, and the proposed solutions to stabilize the affected region.

1 INTRODUCTION

Landslides in hilly places are influenced by two main factors: geological environment and groundwater conditions. The geological environment is a complex and dynamic system that changes throughout time through slow and deep processes. Landslides are generally caused by structural-tectonic conditions and the geological composition of strata. The formation of a specific type of landslip is preceded by a long period of preparation, which includes rearranging the slope site, decomposing geological formations in the slope massif, and changing key parameters of the pressure field, structure, condition, and properties of the slope's breeds. After exceeding peak resistance of breeds to shift destruction in a zone of landslip displacement, the process of deformation at a stage of basic displacement typically transitions to a catastrophic character with high destructive energy.

Landslides are most frequently caused by changes in groundwater conditions. Excessive rainfalls or interference with natural drainage can cause groundwater levels to rise. Landslides and precipitation have a strong correlation, as seen by the increasing frequency of initiated or reactivated events. Groundwater can affect slope stability by raising the weight of saturated materials, causing pore pressure, weakening soft rocks and soils, and decreasing the effective normal stress along the slip surface. A decrease in effective stress reduces the shear strength of the geomaterial along the slip surface.

2 SITE NATURE

2.1 *General*

The present site is a water pumping station (SP6) that is located at Ain El Hadjer – Bouïra - Algeria, Figure 1. The study area is located at the northern part of Algeria between longitudes 3°47'14.61"E to 3°47'20.56"E, and latitude from 36°23'16.27"N to 36°23'20.70"N. A landslide was observed at the site (surficial cracks and damage to the constructions) along with the construction of the station as presented in figure 2 a & b. In this paper, the landslide is documented, its cause is investigated, and the possible consequences on future earthwork are discussed.

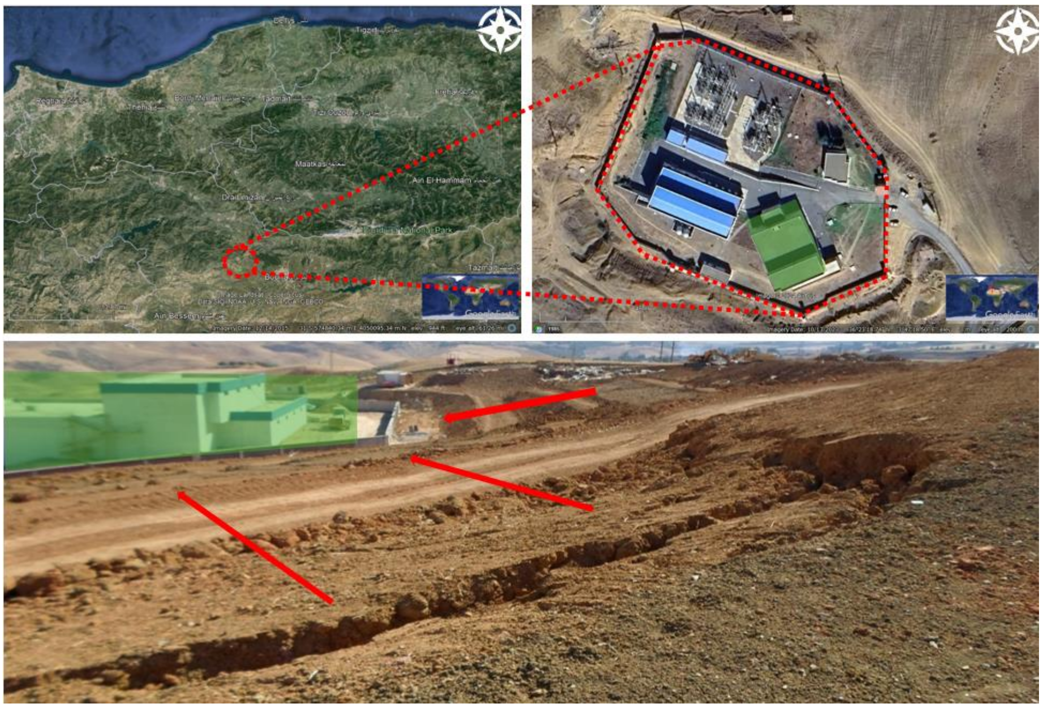

Figure 1 :Location map of El Bouïra Water Pumping Station, Algeria.

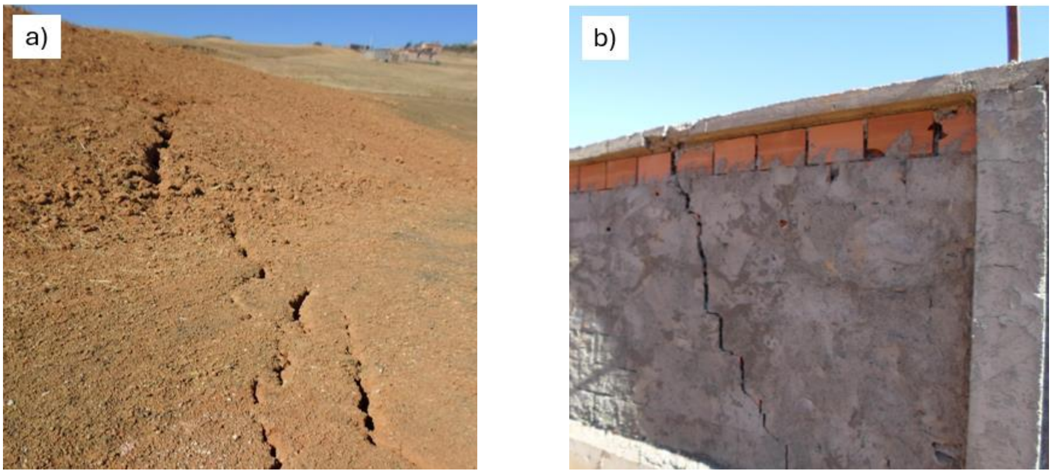

Figure 2 :a) surficial cracks of the slope, and b) cracks in the fence at the slope toe (El Bouïra Water Pumping Station, Algeria)

2.2 *General geology of site area*

The northern Algeria area is made up of young mountains formed during Tertiary times by the Alpine orogeny. Alpine Algeria consists of a number of structural-sedimentary units, from north to south (Fig. 1):

• Off-shore Algeria is a reduced continental shelf bearing Tertiary and Quaternary sediments (1000 to 3500 m, Mio-Pliocene target) overlaying a metamorphic basement;

• The Tellian Atlas is the nappe domain, with mountain basins (i.e. Chelif Basin), and a sedimentary column ranging from Jurassic to Miocene;

• Hodna Basin is a foredeep basin whose sedimentary infill begins with Eocene and Oligocene continental deposits overlain by marine sediments of Miocene age.

• The High Plateaus are the foreland of the Alpine range bearing a thin sedimentary cover (Liassic target). Local distension mechanisms allowed the formation of intra-mountain basins;

• The Saharan Atlas was formed from an elongated trough pinched between the High Plateaus and the Saharan Platform. During the Mesozoic times, the trough was filled in by thick (7000 to 9000 m) sedimentary deposits. Later, the Tertiary compressive tectonic stresses modified the former extension trough into a number of reverse structures which led to the creation of the mountain range. The main target of this area is the Jurassic.

Geologically, the study area is belonging to the Tellian Atlas at northern part of Algeria. The Tellian Atlas, a complex area comprising a succession of nappes set in place during the Lower Miocene. Late Neogene basins such as the Chelif Basin or the Hodna Basin were deposited over these nappe structures. The Tellian zones constitute the External Zones of the belt and overthrust the Atlas foreland (Vila, 1980; Bouillin, 1986; Frizon de Lamotte et al., 2000). The flysch units (Massylian, Mauretanian, and Numidian) and the Internal Zones of the belt, made of Hercynian or older basement and its sedimentary cover, overthrust the External Zones (Fig. 3).

Landslides are a major hazard in the mountainous areas of northern Algeria and are often triggered by heavy rainfall (Bougdal 2007) or earthquakes, even of moderate magnitude (e.g., Machane et al. 2009; Guemache et al. 2010). For instance, the north Algeria area is well known for its mountainous reliefs and the instabilities lying there. The Bouira 1/50,000 geological map (Ficheur,1911) indicates to the right of the site and its vicinity the multicolored clay formations of the lower Miocene to Oligo-Miocene, as well as the Quaternary old alluvial deposits of the low areas and alluvial fans.

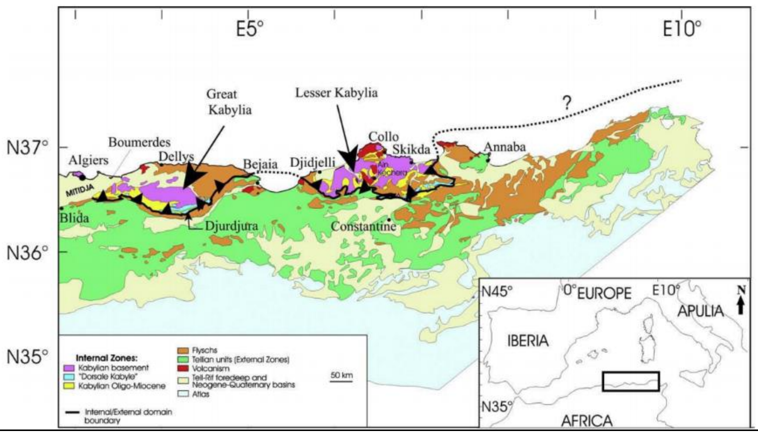

Figure 3: Geological setting of the NE part of Algeria (modified from Domzig, 2006). Dark line is the suture zone between the Internal and External Zones of the Al pine belt. Dashed line indicates the assumed offshore extent of the thrust front, which is not identified.

2.3 *Ground conditions at slide region of El Bouïra*

Based on the topographical data of the current study area, a 3D visualization has been created to show the slope nature and geometry of the slope at the station as demonstrated in figure 4, along with the executed boreholes, inclinometers, and piezometers. Four boreholes were conducted to investigate the lithological nature in view of the area. The area consists mainly of Clay and Sand layers. The description of the soil profile is as following:

- 0.00 – 5.00 m: silty clay layer blackish brown with sandy gravel (Weathered layer)
- 5.00 - 15.00 m: friable consolidated clayey sand with greenish and whitish spot
- 15.00 - 20.00 m: thick layer of consolidated sand with yellowish always whitish nodules and presence of stones and centimeter blocks

Figure 5 shows a simplified geotechnical section in the water pumping station area of El Bouïra Region along the failure zone. The piezometer readings were used to locate the ground water table in the slope.

According to the boreholes, the clay and the sand have the following properties:

- Clay: Saturated unit weight = 2.17 t/m3, Clay Fraction = 33-53%, Natural water content (at the time of the investigation) = 10-31 %, Liquid Limit = 39-69%, Plasticity Index = 21-40%, Degree of saturation = 91%.
- Sand: Bulk unit weight = 2.07 t/m3, Clay Fraction = 16%, Natural water content (at the time of the investigation) = 15 %, Liquid Limit = 34-50%, Plasticity Index = 17-27%, and Degree of saturation = 97%.

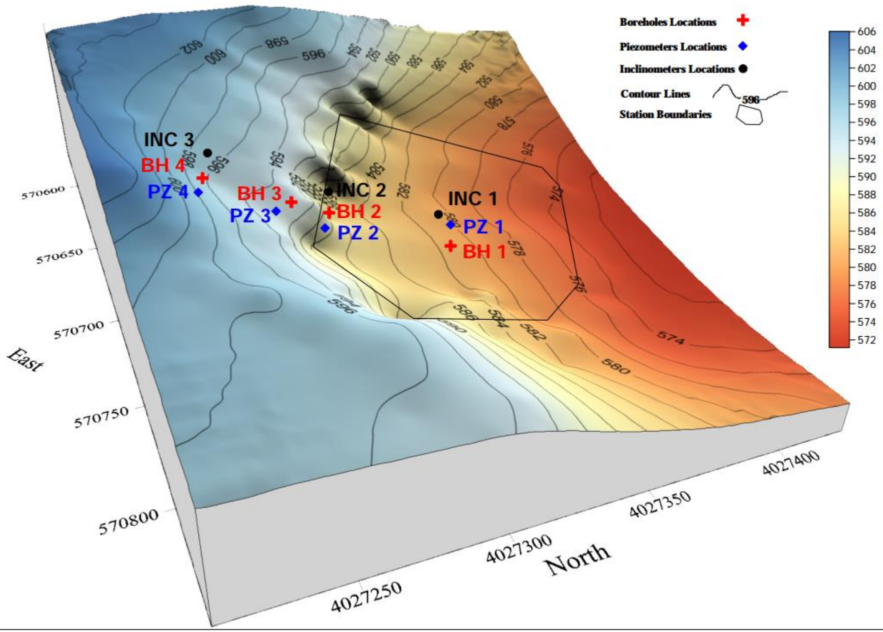

Figure 4: 3D visualization of the slope and stratification and site investigation at El Bouïra slide area.

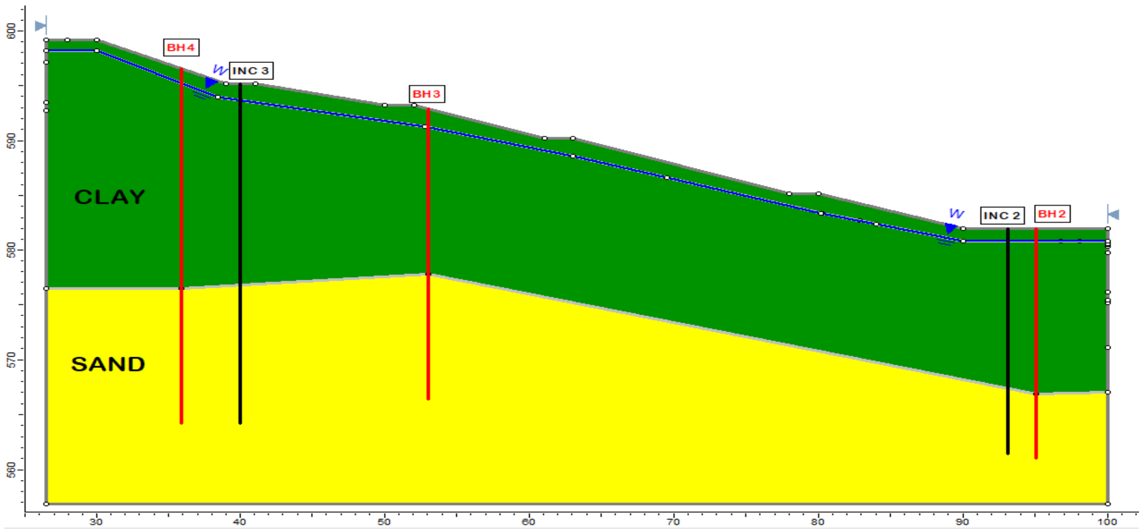

Figure 5: Cross section of the slope and stratification at El Bouïra slide area.

2.4. Climate conditions

The climate of the region of Bouira is typically Mediterranean with a dry and warm summer and a cold and rainy winter and sometimes snowy in high mountains. The Bouira station shows that average monthly temperatures vary between 12 °C in winter and 27 °C in summer. Average precipitation ranges between 400 and 660 mm per year. The cold season, the months of November, December, and January received on average 201 mm or 40% of the annual volume. These rains are often quite intense for example, it fell 50 mm in 10 h on November 7, 2002 (Slimi and Larue 2010). These precipitations play a morphogenic role, that is to say, that can modify local topographic forms (Hugonie 2004; Slimi and Larue 2010).

The dry season, from June to September, favors decohesion and cracking of clay soils, exceptionally high-intensity thunderstorms, such as that of August 24, 2002, which discharged 27 mm in 3 h, occurring on soils that lost their cohesion during the summer drought. Mud cracks that appear during this dry season facilitate the infiltration of rainwater, leading to loss of cohesion of slippery sediment s in this zone. The autumn showers, more abundant than in summer, multiply the possibilities of ravines and sliding slowly or brutal (Hugonie 2004). Thus, autumn concentrates the greatest risks.

Meziani et al. (2016) concluded that the Bouira area shows that the landslide is on a flank whose slope is estimated at more than 20°. This side forms the eastern flank of an anticline fold, the heart of which is formed of Senonian formations on which lie schistose clays, favorable to sliding. This means that the layers dip in the direction of the topographic slope, thus conforming to the slope of the topography, which promotes instability at this site.

3 EXPLANATION FOR MECHANISM OF THE OBSERVED LANDSLIDE

The presence of the overconsolidated clays and weathered soil over sand as observed and reported in above are the main explanation for the failure. At El Bouïra sliding area, observations from the inclinometers, matching with the boreholes, the slip surface starts from the cracks at the surface at the 4^{th} berm from the station (about 50m from the cracked fence wall) and extends down to a depth of about 6.5m. It can be anticipated that cutting slope for station construction has exceeded the safe slope of the soil nature. The strength to be used along the shear failure surface is the residual drained strength. Intact to fully soften drained strength should be at locations other than the failure surface.

4 ESTIMATING THE SHEAR STRENGTH OF SLIDING GEOMATERIAL

The shear strength at the plane of failure may be obtained by analysing the geometry immediately prior to failure (Figure 5) with a factor of safety of unity. The failure surface's residual strength is predicted to be the back-calculated mobilised strength. The outcome of this investigation might not have much bearing on upcoming assessments of slope stability in the region, which might not be influenced by this surface's strength. But the analysis needs to be done because, if the failure surface strength is realistic and falls within acceptable bounds of such strengths, the back analysis could be helpful for verifying the anticipated mechanism. Such an approach was used by many researchers in literature (e.g. Mesri and Shahien, 2003 and Hamza and Shahien 2004)

Mesri and Shahien (2003) provided empirical correlations for estimating the secant residual and fully softened friction angles using plasticity index at effective normal stresses of 50, 100, and 400 kPa. The correlation was based on laboratory testing results database based on (Eid, 2001) and back calculations of mobilized shear strength from about 100 case histories of slope failures and landslides. Mesri and Shahien (2003) suggested the following form of power equation to describe the non-linearity of fully softened and residual shear strength envelopes:

$$s = \sigma_n' \tan\,[\varphi_s']_{100} \left[\frac{100}{\sigma_n'}\right]^{1-m} \qquad (1)$$

where $[\varphi'_s]_{100}$ = secant friction angle (fully softened or residual) at σ'_n= 100 kPa, σ'_n is effective normal stress, (1-m) = slope of $\log(\tan[\varphi'_s]_{100}/\ \tan[\varphi'_s]_{\sigma'n})$ versus $\log(100/\sigma'_n)$, and $[\varphi'_s]_{\sigma'n}$ = secant friction angle (fully softened or residual) at any σ'_n. The component n is calculated based plasticity index. The plasticity index of the clay is used to estimate the secant residual and fully softened friction angles at effective normal stresses of 50, 100, and 400 kPa. Thus, the shear strength envelopes could be estimated. Figure 6 shows the estimated fully softened and residual shear strength envelope of the clay based on an average PI = 29% considering the values range.

Figure (7) estimates shear strength envelopes without considering particle aggregation's impact on empirical correlation (Mesri, 1991). Consider utilizing Eid (2001) in a future edition of the work to account for such influences.

5 SLOPE STABILITY BACK ANALYSIS

5.1 *General*

This work analyses slope stability using Janbu's generalised method of slices (1957) and (1973). Janbu's method of slices is an iterative procedure that uses vertical slices and any form slip-surface. The rigorous approach satisfies all equilibrium conditions, including vertical and horizontal force equilibrium, slice moment equilibrium, and sliding mass equilibrium. Interslice forces were estimated by considering the moment equilibrium around the centre of the base of each slice. According to Janbu (1973), the position of the interslice forces' line of thrust is an imaginary line that is drawn through the points where the forces operate, making the overall moment equilibrium implicitly satisfied and the problem statically determinate. The possible failure mass is separated into a number of vertical slices during the process, and both the equilibrium of each slice and the equilibrium of the total mass are taken into account. Two methods were examined for back analysis: the first involves accounting for stress dependency using nonlinear shear strength envelopes, and the second involves using the software's parameter sensitivity analysis to find the representative values at failure, which could only be found using the Mohr-Columb model. The analysis is two dimensional considering the large area and length of the slope with respect to its depth. Thus, the potential influence of the three-Dimensional effect of the end lateral sides of the slipping mass can be neglected as reported by Baligh and Azzouz,1975. In addition, as reported by Leshchinsky and Baker,1986, For soil with zero cohesion, the potential slip surface and slope surface coincide. Thus, in this case there are no end effects.

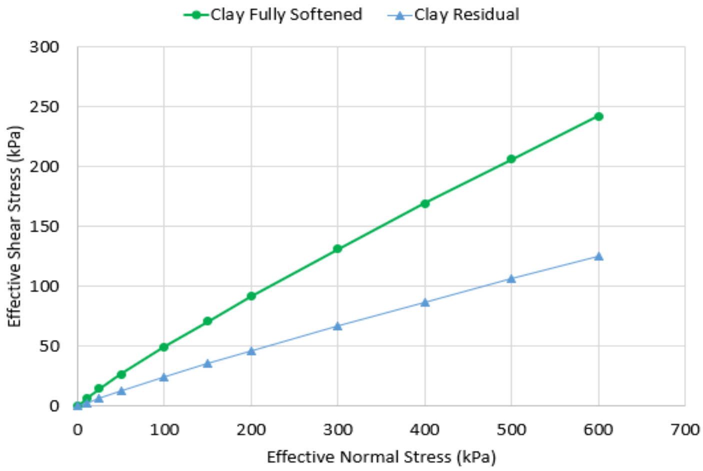

Figure 6 : Fully softened and residual shear strength envelopes of the clay layer.

5.2. Nonlinear shear strength envelope

The cross section of the landslide in Figure 5, together with the shear strength envelopes in Figure 6, are used in the analysis. The fully softened strength is assigned for the clay layers, except for the shear failure surface zone. The failure surface is modeled as a fictious weak layer with relatively small thickness of 0.25m for modeling purposes. This thickness is in the typical range as reported by Cornforth, 2005, and the weak layer properties are assigned as the clay residual shear strength values. In searching for the minimum factor of safety, two surface options were investigated. The first is through the circular surface option to check whether the surface will pass through the residual zone, while the second is using the non-circular polyline option to enforce the slip surface to pass through the residual zone. The results of the two cases are presented in figures 7 and 8, results show that both two cases reveal almost a failure condition, i.e., safety factor of approximately 1 using the aforementioned soil properties. Considering the range of PI values and the corresponding shear strength variation accordingly, the values are deemed acceptable to represent a failure condition.

5.3. Sensitivity analysis using Mohr-Columb model

After assuring the failure surface location, a sensitivity analysis has been run to investigate the optimum values of friction angles, i.e., fully soften for the Clay layers and the residual friction angle at the failure zone. The friction angles were initially estimated from (Mesri and Shahien ,2003) considering an average effective overburden 100kPa and varied between ±10°. The sensitivity analysis results for both circular and non-circular failure surfaces are presented in figures 9 and 10. Results reveal that the failure condition occurs at the fully softened friction angle for the clay layer and the residual friction angle at the failure surface zone, taking into account the range of the PI measured values and the reported variation of the friction angles from (Mesri and Shahien, 2003).

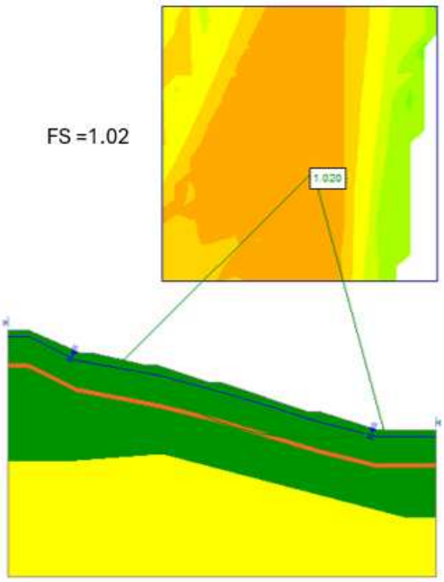

Figure 7: Slope stability FS results (Circular Surface)

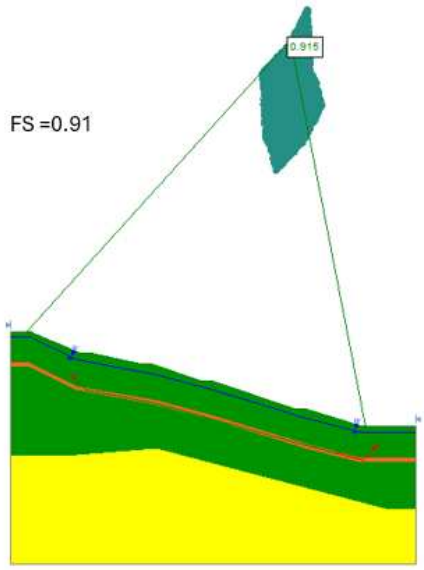

Figure 8: Slope stability FS results (Non-Circular Surface)

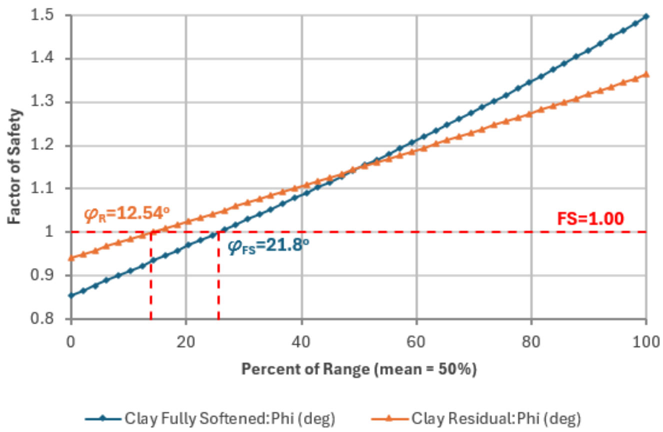

Figure 9: Sensitivity Plot (Circular Surface)

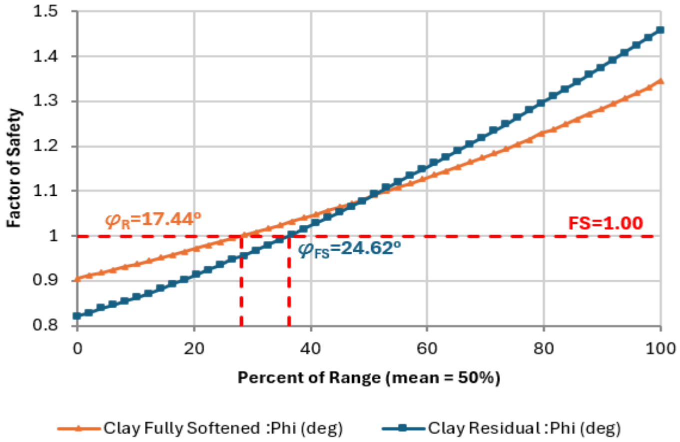

Figure 10: Sensitivity Plot (Non-Circular surface)

7 INVESTIGATING-CONFIRMING THE EFFECT OF RAINFALL

It is anticipated that a critical trigger for the slides is the rise of ground water at rainy season (following month to investigation time). Such a rise increases the water pressure resulting in a decrease in the effective normal stress along the slip surface. The decrease in effective stress leads to a decrease in the shear strength of the geomaterial along the slip surface. This effect was investigating, resulting in a further decreased safety factor of 0.91 and 0.84 for circular surface and non-circular surface respectively as presented in figures 11 and 12. The results indicate the necessity of considering the rainfall effect on the long-term slope stability.

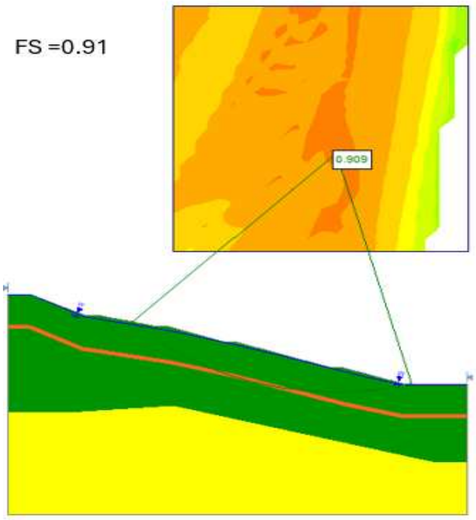

Figure 11: Slope stability FS results for rainfall case (Circular Surface)

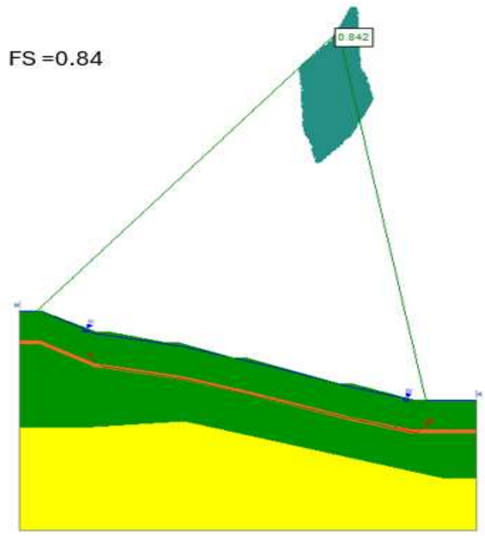

Figure 12: Slope stability FS results for rainfall season (Non-Circular Surface)

8 SLOPE STABILIZATION REMEDIAL

The required force for slope stabilization at the toe (in front of the station fence wall) has been calculated analytically through wedge analysis force equilibrium and verified by the SLIDE 2D software. For a safety factor of 1.5 according to the US Army Corps of Engineers for long term conditions. The principle of using piles for slope stabilization as presented in figure 13 has been widely reported by many authors, e.g. Poulos,1999; Howe, 2010; Yang et al. 2011; Sobhey et al. 2017 and Hajiazizia et al.,2018 .Hence, the required piles dimensions to achieve the passive stabilizing force has been calculated and a system of RC contiguous pile row has been adopted in front of the fence wall, with length 17m, diameter 0.8 m, and spacing 80cm as staggered pattern to resemble a continuous embedded wall, the piles are constrained with a capping beam of depth 1.2 m and width 1.8m as shown in figure 14. The large diameter piles have been adopted as Poulos (1995) found that having fewer large-diameter drilled shafts leads to better stabilization than having more small-diameter shafts. In addition, 30 monitoring points have been installed to observe and record future ground motion. Photographs from google earth along time, figure 15, confirms the efficiency of the piles system in slide mitigation and preservation of the pumping station from further damage.

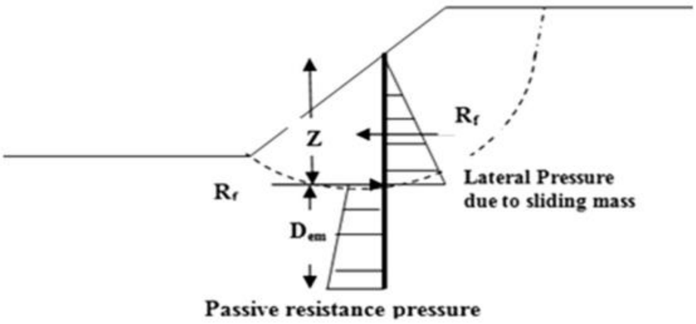

Figure 13: The lateral pressure acting on row of piles in a clay slope (Poulos,1999)

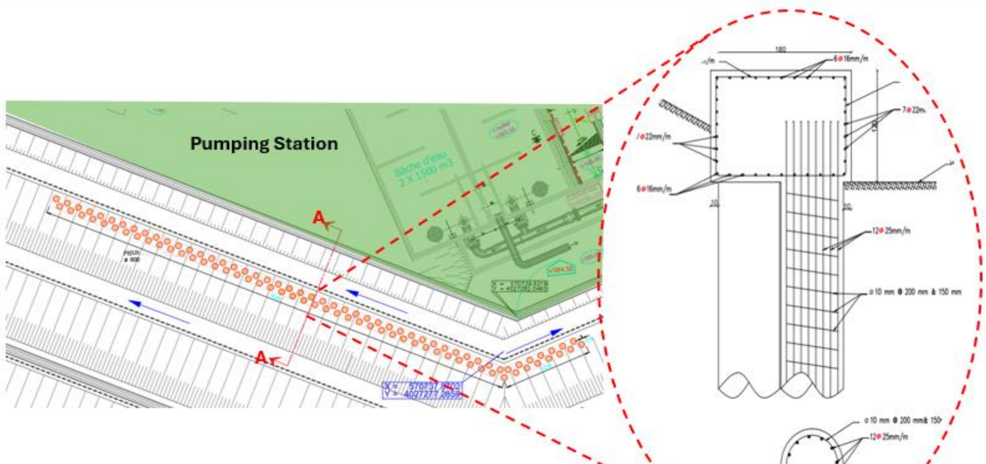

Figure 14: The contiguous pile rows at the fence wall

Figure 15: Google earth photos of the landslide zone from 2017 to 2023

9 CONCLUDING REMARKS

1) This paper records a case history of slope or existing landslide instability that endangers a pumping station in North Africa.
2) The inclined interface beneath the weathered soil zone has an inclined mass that has the potential to fail, and this geological situation indicates that only residual shear strength may be available along this failure surface.
3) The validity of using the shear strength envelope along the failure surface estimation from the empirical correlation developed by Mesri and Shahien (2003).
4) The stability analysis presented in this paper validates the expectation that heavy precipitation events may cause bypass flow through desiccation crack systems, resulting in rapid reduction of effective stresses at critical slip surfaces and subsequent slope deformations caused by a decrease in shear strength.
5) This paper documented the remedial of the landslide using contiguous staggered RC pile walls that has proven its effectiveness along time from field monitoring.

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