Specific slope stability study of Aomar region—characterization and proposal of reinforcement measures

Tawfiq Boufrina
University of Blida-1 Saad Dahleb, Blida, Algeria, tawfiq.boufrina@univ-blida.dz

ABSTRACT: The 162 km High-Speed Railway project connecting the two Algerian cities Thenia in the north and Bordj-Bou-Arerridj in the east encountered a lot of engineering challenges, among which the 10 km landslides that occur in the region of Aomar, district of Bouira. This paper presents a specific slope stability analysis of one of these landslides. The region is historically well known of being the centre of many landslides (the landslide area of "Djabahia" region located next to Aomar is witnessing many landslides and still presenting a major risk for the East-West Highway). In order to a better understanding of the Aomar's landslides typology, a slope stability analysis is carried out with an emphasis on geomorphological, geological and hydrogeological conditions in addition to geotechnical data. Slope stability analysis was conducted using correlations between laboratory and/or in-situ tests with soil strength parameters in both drained and undrained conditions. The General Limit Equilibrium method (GLE) was then used to estimate the safety factor of the slip surfaces using the commercial code TALREN V5.2. The proposed reinforcement measure consists of a range of peripheric piles along the foundations of the bridge to be constructed in the sliding area. This measure will completely isolate the structure from the soil movement and keep the railway platform safe. Finally, practical findings and conclusions related to this important case study are highlighted.

Keywords: Landslide; case history; General Limit Equilibrium Method; safety factor; reinforcement.

1. Introduction

The village of Aomar (Bouira, Algeria) is being affected by many landslides due mainly to the geological and geomorphological characteristics of the area. The tectonic-stratigraphical setting of Aomar area is characterized by the presence of marls of Miocene age, mainly clayey, lying in intercalation with metric sandstone benches. Several distinct landslides can be recognized in the study area. The largest landslide area (subject of this study) evolving along the southern slope exhibits rotational slides with deep slip surfaces (>15m) in the upper part of the slope, and earth-flows (creep phenomenon) that involve the middle-low portions of the slopes. The slip surfaces can be detected in the contact between clayey layers and sandstone metric benches. Although a retrogressive activity, testified by tension cracks, is now involving also areas where the sandstones outcrops, most of the accumulation areas produced by rotational slides, represents the alimentation zone of slow earth-flows that can affect the entire slopes from the top to the river valleys. These complex landslides, accompanied by a progressice increased in time, caused severe damage to the village that has been gradually abandoned by the population.

2. Geology of the region

The Aomar region of Post-Miocene age (Fig. 1) belongs to the chain of 'The Maghrebides 'also called Al pine of northern Algeria in the geological unit of the Tellian Atlas (Alpine orogenesis from -66 to -3 M years ago). This orogenesis took place in several stages, with essentially an anterior Miocene phase, during which the Cretaceous and anterior lands were tectonized (faulted and folded). In the Miocene, the beginning of mountain erosion led to the accumulation of clayey deposits (Helvetian), which became the Miocene marls, with sandy-sandstone intercalations.

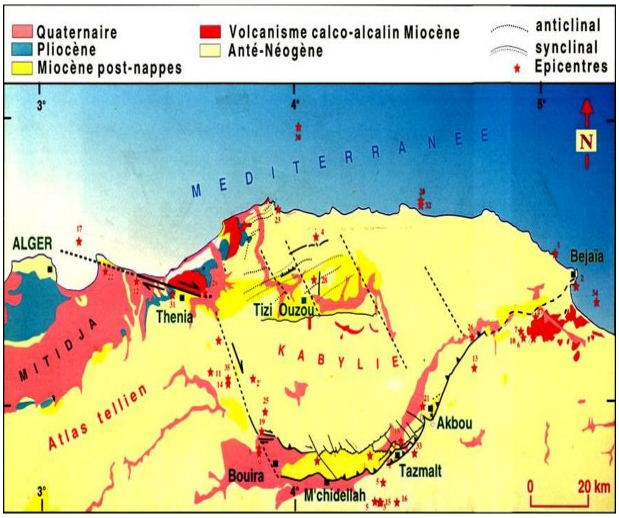

Figure 1. Geological structural map of the region

These Miocene marls are more tectonized during the last phase of the Atlas orogeny.

Then, during the Quaternary period, 3 million years ago, the landscape of today is gradually being drawn. The large valleys are deepening, and the banks of the wadis are eroding, the slopes are the result of various erosion processes, inducing that they are in a state of limit equilibrium.

Soft soils with a clay component (clays, marls, schists) will give rise to landslides. These landslides concern the weathered part of the marls, over a thickness that can vary from 5 m to 30 m. In some special cases, landslides can be deeper, and occur along weak areas within the rock (faulted or very schistose areas).

2.1. Instabilities description

The new line crosses the slopes of the "Aomar" region between the kilometric points PK36 and PK46 (Fig. 2) and continues along a hilly topography with slopes of more than 15%. This area is affected by many instabilities areas which generally follow a parallel movement to the direction of the slope.

The geological mapping of the region has classified these instabilities according to the geometry of the movements into: Sliding areas , Potentially unstable areas and Solifluction areas [1].

We were interested in this work to study the most important sliding area located between PK36+300 and PK37+900 from many aspects mainly geomorphology, geology, hydrogeology and geotechnics.

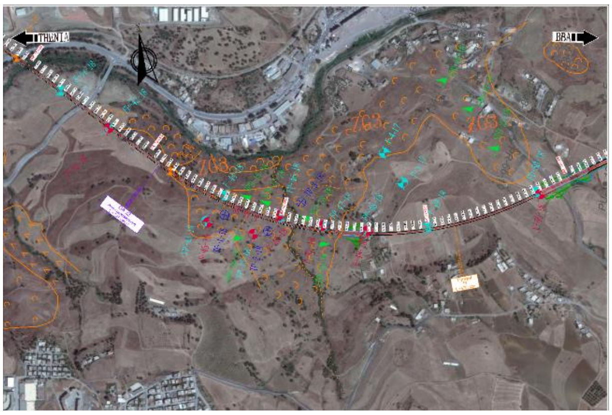

Figure 2. Railway line crossing the sliding area between PK36+300 and PK37+900

2.2. Sliding area between PK36+300 and PK37+900

This clayey marl area has undergone landslides in three different directions, it is characterized by significant slopes exceeding 40%. Several significant landslide areas have been identified.

Detailed geological mapping of the sliding area (Fig. 3) has made it possible to identify the area in motion, to properly locate and delimit unstable parts, as well as the geological, hydrogeological and morphological environment in which they have evolved.

Figure 3. Topography between PK36+600 and PK37+050: View of the foot of the slope, with the sandstone bench

3. Geomorphology

The area is located in a hilly area with morphological irregularities and slope breaks (red and white lines on the Google Earth images below). These slope failures can result either from large and old landslides or from the process of differential erosion.

Between PK 36 + 300 and PK 36 + 600 (Fig. 4), the terrain is relatively flat, then the natural slopes of the terrain gradually increase, reaching 20° in some places. The section between PK36 + 700 and PK36 + 800 is characterized by the presence of a cliff of approximately 30 meters high, corresponding to a sandstone bank (continuous orange lines - visible bench, discontinuous line - probable bench).

From a morphological point of view, the existence of this sandstone bench makes it possible to dissociate the upstream and downstream areas. The upstream area affected by the landslides are limited in their lower part by this stable slope. This makes it possible to assess the depths of the fracture surface at about 10 m at the level of the bridge foundation. And also to consider that there cannot be a deeper surface, which would extend from the top of the slope to the bank of the wadi (rejected hypothesis).

Figure 4. 3D satellite image from PK 36+300 to PK 37+150

From PK 37+150 (talweg) to PK 37+500 (Fig. 5), the slope is generally regular, with slight humming. To the right of the railway line, begins a very flat and sloping large surface (this point corresponds to the entry of the TU09).

From PK37+500 to PK37+700, there is a large flat surface, surrounded upstream by an old tearing pattern, which would suggest a sliding crown, suggesting that the flat surface would correspond to the upper part of a very large sliding body. However, this morphological interpretation is not compatible with the flatness and inclination of the surface.

Beyond the slope break at PK 37+700, we descend to the road at PK 37+900, exit of the tunnel TU09. This slope is marked by a direct erosion morphology, which indicates the presence of sandstone marl. These benches extend laterally quite far, indicating that they have a regular dip.

This observation leads to the conclusion that the previous large flat surface is in fact a stratigraphic structural surface. The crowning all around would then represent the scar of an old landslide and total flattening of the entire rock cover. Such a slide in sandstone marl, with clayey marl intercalations, is only possible in an earthquake situation. If this hypothesis is correct, the downstream slope would correspond to the accumulation of residues from this large, abrupt rock slide. It turns out that in this area, there are many sandstone blocks scattered on the slope, which support this hypothesis.

This means that the tunnel TU09 will enter the ground in these deposits, over to a relatively long length, close to 100 m horizontally.

This also means that the tunnel would only encounter mainly solid marlstone materials, in situ, until it exits, which makes this solution interesting.

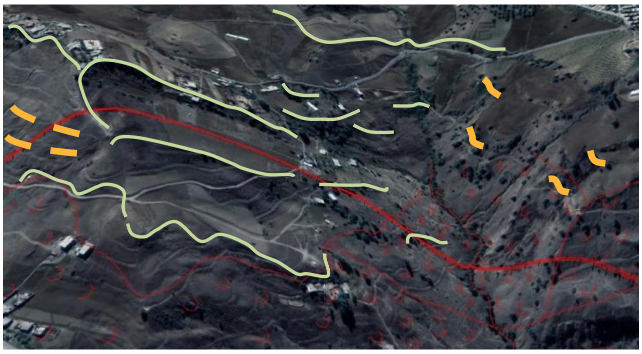

Figure 5. 3D Satellite image between PK 37+150 and PK 37+900

4. Geology survey

Geological surveys in this section on Aomar region (Fig. 6) have shown that the soils consist of Miocene marl interspersed with friable sandstone benches. Those sandstones are characteristic of the Aomar region.

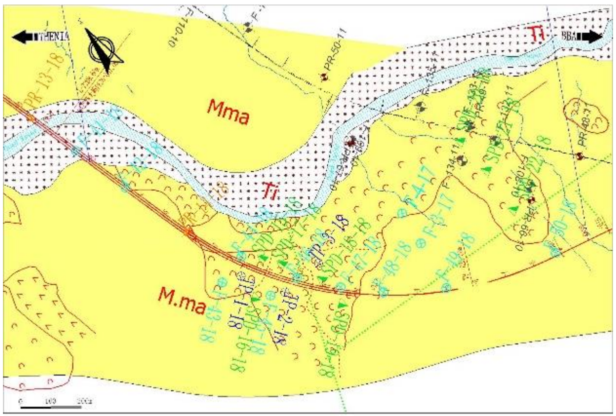

Figure 4. Geological survey between PK 36 and PK 38 (Miocene marl of Aomar region)

5. Geotechnical investigation

A set of geotechnical investigation has been conducted in the area of landslide including boreholes, SPT, DPT and PMT tests. Fig. 7 and Fig. 8 shows sandstone outcrop and Aomar marl. Fig. 9 and Fig. 10 presents some soil samples.

Figure 5. Sandstone bench outcrop between PK 36+700 and PK 36+800

Figure 6. Outcrop of weathered clayey marl

Figure 9. Fully weathered clayey marl

Figure 10. Compact marl interspersed with sandstone

5.1. Ground water level

The piezometers are installed in holes F-41, F-42, F-43-18, F-44-18, F-45-18, F-46-18, F-47-18, TP-1-18, TP-2-18, TP-3-18, F-48-18, F-49-18 and F50-18, and groundwater level measurements were carried out over a period of approximately nine months from 18/06/2018 to 15/03/2019. Some piezometric readings are shown in table 1.

Table 1. Piezometric measurements

Borehole	PK	Date of meas-urement	Water level (m)	Date of measure-ment	Water level (m)
F-41-18	36+140	18/06/18	7.5	15/11/18	5.2
F-42-18	36+345	24/06/18	5.7	15/11/18	4.5
F-43-18	36+775	15/11/18	3.4	15/03/19	1.4
F-44-18	36+795	15/11/18	4.2	15/03/19	16.1
F-45-18	36+930	16/11/18	3.8	1/03/19	1.2
F-46-18	36+995	15/11/18	7.3	15/03/19	2.2
F-47-18	37+145	15/11/18	1.3	15/03/19	1.0
TP-1-18	36+820	27/06/18	8.5	16/11/18	3.3
TP-2-18	36+970	22/06/18	6.0	16/11/18	11.5
TP-3-18	37+055	18/06/18	1.4	16/11/18	5.2
F-48-18	37+290	15/11/18	2.5	15/03/19	26.7
F-49-18	37+500	15/11/18	31.0	15/03/19	31.1
F-50-18	37+885	16/11/18	1.3	15/03/19	1.8

Groundwater is omnipresent in this sliding area (fig. 11). The groundwater level recorded in the piezometers is located in the weathered clayey marl and varies from 1.0 to 9.50 m from ground level.

It is important to note the presence of several wells along this section with a practically water level on the surface. The small dam located upstream (100m in vertical drop upstream) of the line could be part of the source of supply for these wells, especially in dry periods.

Figure 7. Several wells located in the sliding zone

5.2. Inclinometers

Inclinometer measurements determine the magnitude and the direction of the horizontal movements of the slope over time, they provide a very reliable means of locating the depth of landslide planes within the slope. Ten inclinometers were installed according to the French norm (NF-94 156). Table 2 gives the dates of the first and the last readings.

Table 2. Dates of first and last inclinometer measurements

Inclinometer	Lecture "0"	Lecture "5"
F-42-18	30/07/2018	26/11/2018
F-43-18	16/08/2018	29/11/2018
F-44-18	30/07/2018	26/11/2018
F-45-18	16/08/2018	29/11/2018
F-46-18	16/08/2018	29/11/2018
F-47-18	15/08/2018	28/11/2018
F-48-18	15/08/2018	28/11/2018
F-50-18	15/08/2018	28/11/2018
F-03-17	20/03/2018	-
F-04-17	20/03/2018	-

At this stage of the study, it can be noted that the inclinometer F-43-18 showed a very small displacement of about 1mm at 21m depth, which seems to show that a fairly thick slice of the weathered marl is affected by the slide.

Inclinometer F-47-18 showed also a very small displacement at 11m depth, corresponding to the interface between the weathered and the compact horizon of the marl. The rise of the slip surface is related to the presence of sandstone bench observed.

The other inclinometers did not show any significant displacements. The absence of displacement of the inclinometer tube may be explained by the absence of movement during the dry period. This, at least, allow us to say that the aquifers that feed the landslides are linked to seasonal rains, otherwise there would be continuous movements, even if they were slowed down during the dry period.

5.3. Landslide geometry

The morphology of the ground surface indicates that the slope on this section is affected by landslides. These landslides may be old and/or current. According to their morphology, and the absence of active indices, they are slow and progressive, and therefore relatively deep (sliding surface at more than 5 m).

The assumption that the sliding surface could be detected at a depth of 20 m, which corresponds approximately to the thickness of the weathered marl is quite interesting. It makes it possible to consider, for the entire railway line located on Miocene marls, that these potential slides can take place in these geological units, when the slope gradients are greater than 20° and in the presence of water.

On this section, the weathering thickness is about 20 m, up to the plumb of the line. However, the railway line is located on a likely flat area due to the existence of the stable sandstone bench described above. As a result, the failure surface is forced to rise to the surface, to be between 5 m and 10 m deep at the level of the supports of the viaduct. At this stage of study, the 10 m slide depth hypothesis will be chosen for the design of the protection structures for the viaduct piers, in order to ensure safety.

5.4. Results of in-situ tests

The results of the in situ-tests (essentially the pressuremeter test PMT) performed in four boreholes PR-14-18, PR66-10 and PR-49-11 and PR-48-11 in weathered and compacted marl are summarized as follows:

a. The pressure limit obtained for the weathered marl of miocene age (named MHa) varies from 4.4 to 35.6 bar (according to PMT test PR-14-18), attesting to the soft (weathered) nature of this marl roof over a section of land about 8m thick. The Menard modulus at this marly roof varies from 48 bar to 600 bar.
b. Above 8m depth, the overall recorded limit pressures exceed 50 bar with Menard pressuremeter modulus of around 2500 bar, these results highlight the compact nature of the marl at depth.

5.5. Results of laboratory test

Laboratory tests on samples consist of the identification tests (physical and chemical) and mechanical tests (direct shear and uni-axial compression).

The results obtained showed the extent of the weathered marl roof, which is vulnerable to instability, for a depth up to 20 m (F-42-18 to F48-18), except for boreholes F-49-18 and F-50-18 executed along the tunnel and in the east side portal which showed a thick depth of the marl up to 42m.

The wet bulk weight (γ_h) vary according to the depth investigated between 2.0 and 2.50 t/m^3, indicating the dense to very dense nature of this marl (consolidated state).

The moisture content decreases with depth, its ranges from 12 to 18% on shallow depths reaching very weak values in the range of 5% in the depth. The variation in soil moisture with drilling depth did not reveal a clear correlation of a significant change in soil moisture in the sliding surfaces.

The drained shear parameters deduced at the direct shear test on weathered marl samples are estimated as follows:

A drained cohesion, c' varies between 32 and 38 kPa (an average of 36 kPa)

A drained friction angle φ' varies between 14° and 18° with an average of 16.50°.

The residual shear parameters of the weathered marl roof deduced from the altered consolidated drained shear tests are presented as follows

A residual drained cohesion varies between 0 and 20 kPa (average of 7 kPa)

A residual drained friction angle varies between 9° and 14° (average of 11.00°).

For the compact part of the marl, the shear tests could not be carried out because of the friable rock aspect of this layer.

However, simple compression tests were carried out on the compact rocky marl, the simple compressive strength values obtained are scattered, they vary according to the degree of fracture between 1.10 MPa and 8.50 MPa (average 4MPa), attesting to the poor quality of this rock.

Chemical analyses of $CaCO_3$ (calcium carbonate) carried out on samples of this Miocene marl provide carbonate contents between 9.00 and 30.24%, confirming the marliness of this layer.

6. Geological and geotechnical profiles

On the basis of the collected data, the geological surveys carried out and the results of the geotechnical investigation carried out, geological sections of the slide along and across the axis are established to perform a stability analysis. For this purpose, the french code TALREN [3] was used to conduct the numerical analysis.

7. Slope stability analysis and reinforcement measures

Part of the bridge OA22 between PK36+300 and PK37+210 (12 piers between P32 and P43 + abutment C02) crosses the sliding area. Its deck is composed of a 50m span continuous steel-concrete composite beam. The height of the piles at this varied area from 5 to 35 m. Pile foundation were adopted, the foundation of the piers are composed of a group of 4 x 3 piles of 1200mm diameter and of 30 m length.

The piles are anchored at least (5m) at the level of the compact marl characterized by a limit pressure $Pl > 50$ bars and Menard modulus $E_M > 2000$ bars below the identified slip surface.

7.1. Back analysis

Back analysis is carried out using the french software TALREN by the limit equilibrium calculation method according to the simplified Bishop formulation and/or the method of perturbations, Fig. 12 shows the results obtained from back analysis performed on one of the four selected critical sections.

For seismic conditions, a pseudo-static analysis was conducted according to the Algerian National Code RPOA 2008 (The Algerian Seismic Rules Applicable to Bridges and Tunnels) [4], the coefficient of acceleration “A” was taken aqual to 0.25, the horizontal and vertical components of the seismic load are given as follows:

$$k_h = 0.5 \times A = 0.125, k_v = 0.3 \times k_h = 0.037 \quad (1)$$

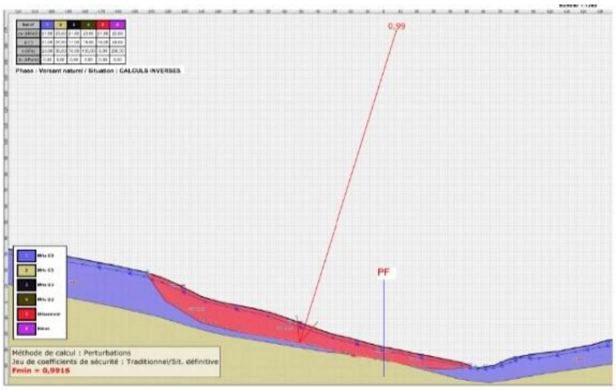

Figure 12. Back analysis section 2-2' at PK36+800 (Pier 35)

7.2. Bridge foundation stability analysis Post-treatment stability check

The stability analyses are carried out in provisional and permanent situations (static and pseudo-static), and consisted in the construction of the pile nailing followed by the construction of the pile foundation.

The solution adopted for the stabilization of the landslide affecting the piers P32 to P43 and the abutment of the bridge is the construction of a retaining structure consisting of a pile wall on "U obtuse shape". This reinforcement proposed (Fig. 13) is composed of 1200mm diameter piles, 30m long, with a spacing of 1.5m, stiffened by a reinforced concrete footing on the contour of the foundation and justified with regard to the stability to landslide in provisional and permanent and static and seismic (pseudo-static) situations.

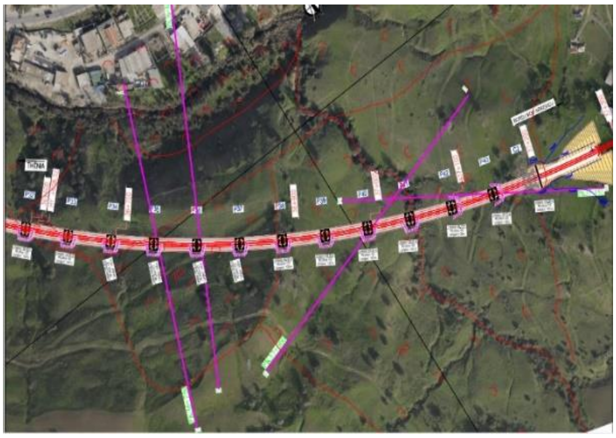

Figure 13 . Reinforcement system for the bridge piers P32 to P43

Table 3 summarizes the results of the stability calculations after the installation of the reinforcement measures on the 4 critical sections.

Table 3. Results of the stability calculations for the Sliding area

Section	Pier	Construction/ calculation steps	Calculation Situation	FOS (Required) [2]	FOS
1-1’	P35	Pile installation	Provisional	1.1	1.11
		Pile installation + bridge foundation	Static permanent	1.2	1.26
		Pile installation + bridge foundation	Seismic	1.0	1.05
2-2’	P36	Pile installation	Provisional	1,1	1.15
		Pile installation + bridge foundation	Static permanent	1.2	1.26
		Pile installation + bridge foundation	Seismic	1.0	1.01
3-3’	P40	Pile installation	Provisional	1,1	1.09
		Pile installation + bridge foundation	Static permanent	1.2	1.28
		Pile installation + bridge foundation	Seismic	1.0	1.03
4-4’	P43/ Abuttm ent	Pile installation	Provisional	1,1	1.17
		Pile installation + bridge foundation	Static permanent	1.2	1.41
		Pile installation + bridge foundation	Seismic	1.0	1.01

8. Conclusion

Detailed geology mapping of the sliding area of Aomar region has allowed to describe and delineate the sliding areas crossing the railway line, morphological and hydrogeological conditions explained some aspects of the Aomar landslide (some other aspects are still unknown and could be tectonic movements, faults, etc.) of the instabilities. the analysis of a rich geotechnical data (In-situ tests, laboratory tests, instrumentation) leads to the stability analysis and the proposal of a reinforcement measure for the structures crossing the landslide area.

Acknowledgments

The author thanks “CCECC-Algérie” for providing data for a the purpose of a scientific study.

References

[1] Philipponnat, G. & Hubert B. 2006. “Fondations et ouvrages en terre”, (Foundations and earthworks structure), 2nd edition, Eyrolles, Paris, France, 2000. https://www.eyrolles.com/BTP/Livre/fondations-et-ouvrages-en-terre-9782212144871/

[2] Laboratoire Central des Ponts et Chaussées, LCPC, (Central Laboratory of Bridges and Roads), “Guide technique pour la stabilisation des glissements de terrain, techniques et méthodes des Laboratoires des ponts et chaussées”, (Technical Guide for landslides stabilisation , Techniques and methods of the laboratories of Bridges and roads), Ministère de l’équipement, du transport et du Logement, (Ministry of Equipment, of Transportation and of Housing), Paris, France, 1998. https://structurae.net/fr/litterature/rapport-technique/stabilisation-des-glissements-de-terrain-guide-technique

[3] Terrasol "TALREN V.5.2", [TALREN] Available at: https://www.terrasol.fr/en/catalogue/talren-v5

[4] RPOA 2008 : Règles parasismiques applicables au domaine des ouvrages d’art (the Algerian Seismic Rules Applicable to Bridges and Tunnels)