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Investigating the Instability of a Retaining Wall: Construction Concerns and Design Revisions

Investigarea instabilității unui zid de sprijin: preocupări de construcție și revizuiri de proiectare

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Abstract :
This article examines a partially overturned retaining wall designed to protect the Taher Treasury Administration in Jijel Province, Algeria, from the sliding of the slope on which it is constructed. Although the wall remained intact, it led to the project being suspended by local authorities due to concerns about its stability. Despite several expert assessments, the root cause was not identified, necessitating in-depth analysis. Field investigations revealed unexpected results: the presence of a plastic sheet behind the wall hindered the drainage of rainwater and exacerbated the pressures on the surface layer of the backfill material. In particular, the wall's anchoring proved insufficient, barely affecting the intermediate layer. This configuration was critical for understanding the dynamics of soil pressures leading to superficial and partial sliding, which impacted the retaining wall's stability. Comparisons between field observations and our numerical model highlighted discrepancies in design dimensions and construction practices. Our revised model proposes a larger, deeper, and better-anchored retaining wall configuration, contrasting with the initial designs. This study concludes that errors in both design calculations and construction implementation contributed to the wall's instability, underscoring the importance of meticulous planning and adherence to geotechnical principles in future projects.

Keywords: Diagnosis, partial and superficial sliding, Retaining wall, lateral detachment, overturning, investigations, numerical simulation.

Introduction :

Geotechnical failures can have significant impacts on structures and represent a major challenge in civil engineering and risk management worldwide [1-5]. These often catastrophic events have devastating economic, environmental, and social consequences [6-10]. To improve the prevention, response, and mitigation of

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geotechnical disasters, it is crucial to understand the causes, mechanisms, and impacts of these failures [11]. The importance of geotechnical stability lies in its ability to prevent geotechnical and structural failures that can lead to disasters such as landslides, foundation collapses, or dam breaches [12-17]. To achieve this, engineers conduct geotechnical investigations [18-21], including in situ and laboratory tests [22-24], as well as numerical analyses to model the behaviour of soils and structures [25 - 29].

If a failure occurs, it is undeniable evidence that the engineering of the failed structure was incorrect or incomplete. Today, geotechnical engineers conduct thorough investigations of failures and hypothesize how the failure was initiated and progressed [30-32]. These investigations involve reverse engineering design and construction problems [33, 34], where the engineer must develop possible scenarios and test them through analysis [35, 36]. In many cases, failures are not due to a single deficiency but rather to an unfortunate combination of factors, making the prognosis even more challenging [37, 39]. In addition, the variability of soil parameters and the resulting uncertainty are fundamental aspects of geotechnical engineering [40]. Understanding and quantifying this variability and uncertainty is essential for improving the reliability of geotechnical predictions and making informed decisions in the design, construction, and management of geotechnical infrastructure.

The stability of retaining walls is crucial in geotechnical engineering to protect infrastructures against landslides [41]. Our article aims to study geotechnical stability and conduct a diagnosis to find the cause of the superficial and partial sliding that led to the overturning of a retaining wall, with the goal of proposing reinforcement solutions for this wall installed on the slope of the Treasury Project in Taher, Jijel Province in Algeria. Since the sliding has already occurred, the objective of this study is to analyze the causes that led to the instability, and then propose an adequate reinforcement system, while describing the procedure for its implementation. To carry out this study, our article focuses on the geotechnical, geological, and hydrogeological study of the sliding site. It also includes the verification of the initial study's calculations, which failed, using the PLAXIS 2D program, as well as an in-depth study of reinforcement options and verification of the sliding site's stability. Finally, our work concludes with a general summary of what we have learned and compiled in terms of study and reinforcement methods for landslides.

Position of the retaining wall and verification of slope stability:

The site intended for the construction of a municipal revenue office in the city of Taher, in the Jijel province of Algeria, is located at the foot of a small slope (figure1). This slope has experienced some movements, causing the existing retaining wall to partially overturn and shift laterally.

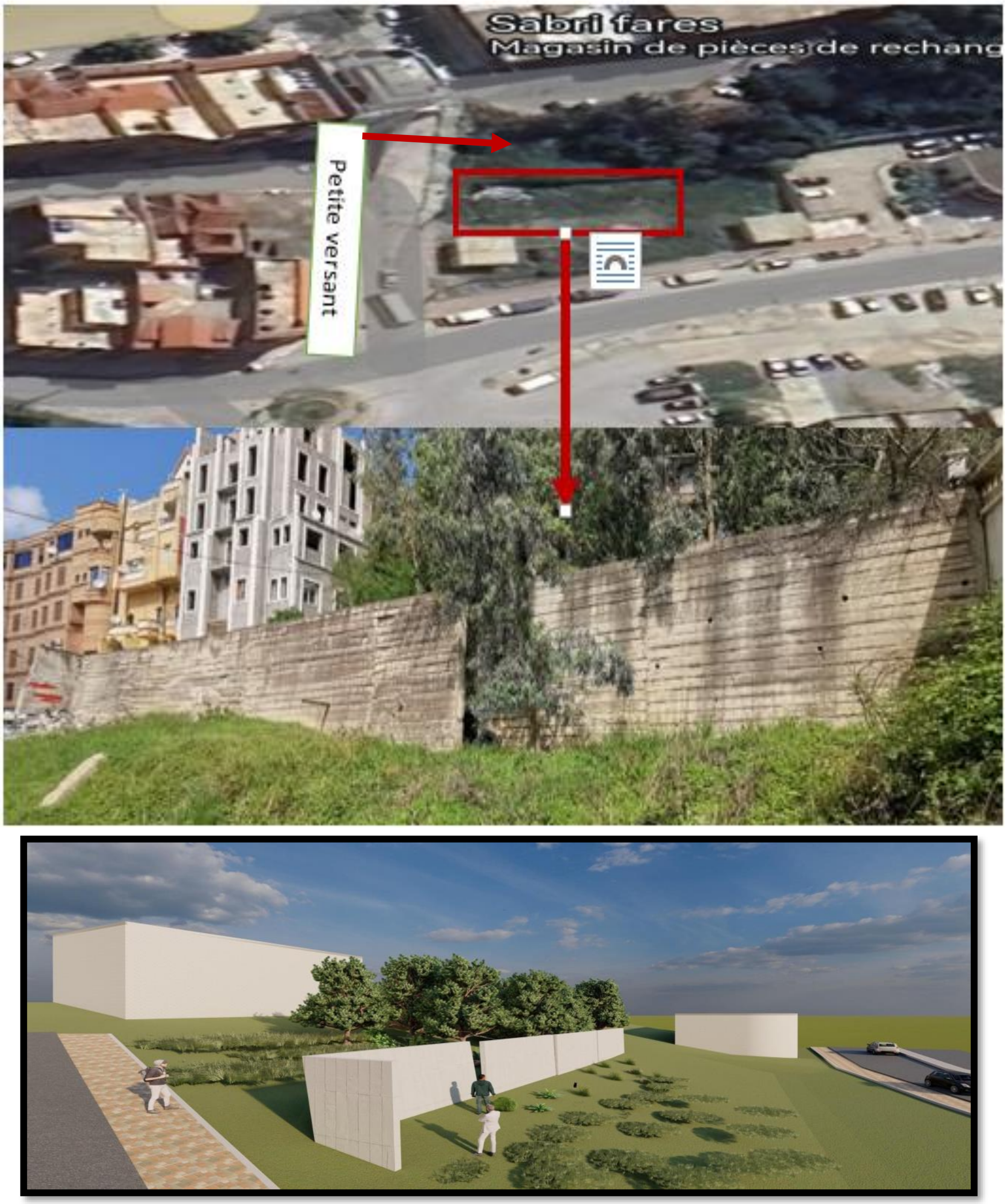

Figure 1 : position of the retaining wall.

The stability check consists of both checking the stability of the slope on the one hand and determining the direct causes of the movements of the existing retaining wall on the other hand (figure 2).

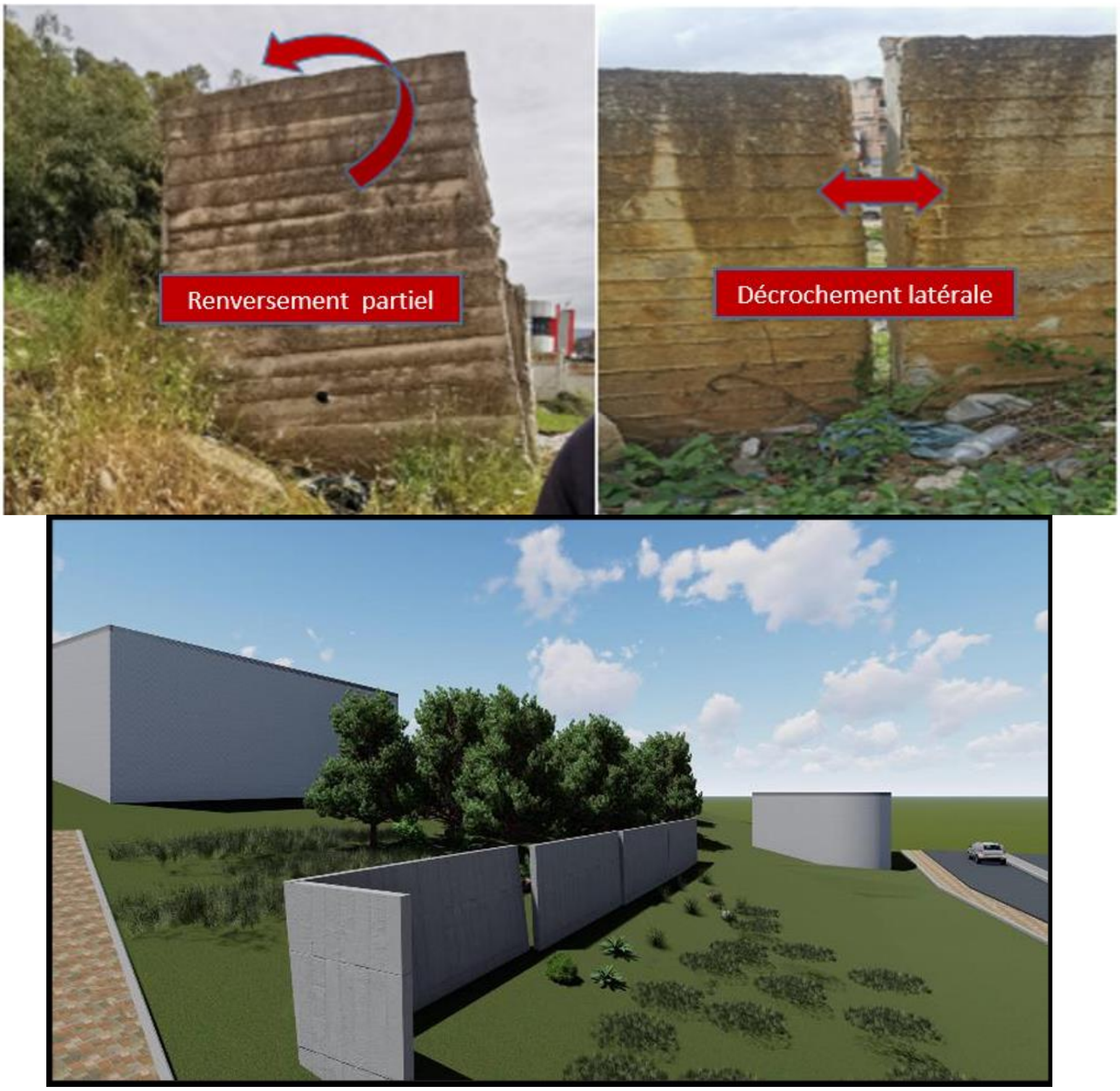

Figure 2: a) deformation on the retaining wall (partial overturning and lateral detachment) b): 3D photo of the site.

The lithology of the slope (figure 3) is characterized by a layer of backfill with a variable thickness ranging from 1.0 to 6.0m (the highest elevation is detected at the top of the slope), this deposit resting on a sandy clay and a clayey marl. Then in depth a marl is found.

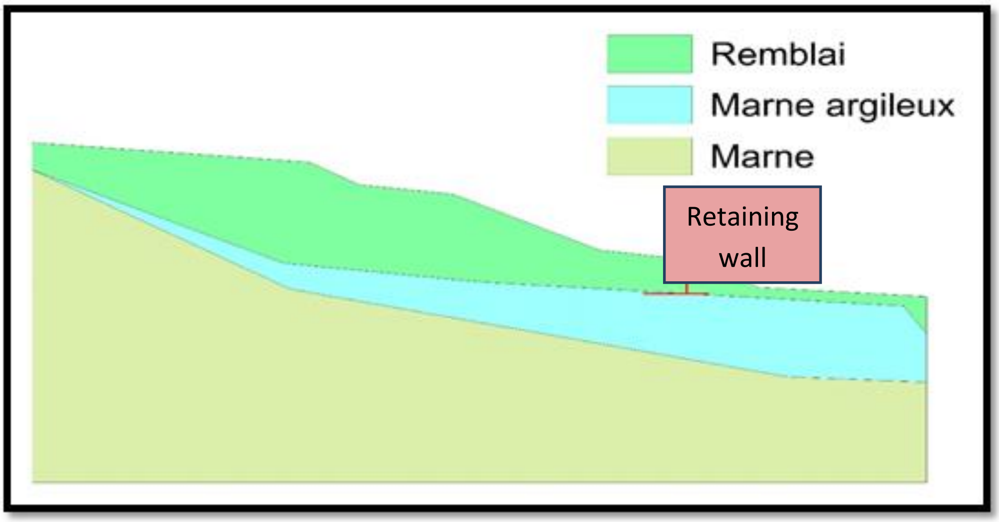

Figure 3 ; Lithological section of the project site.

The manual calculation of the stability of a slope (Figure 4) is done by trial and error, finding the most unfavorable slip line while strictly adhering to the geometry, geotechnical characteristics, and hydraulic properties of the embankments [51]. This section presents the manual calculation results of our slope failure using the Fellenius and Bishop limit equilibrium methods.

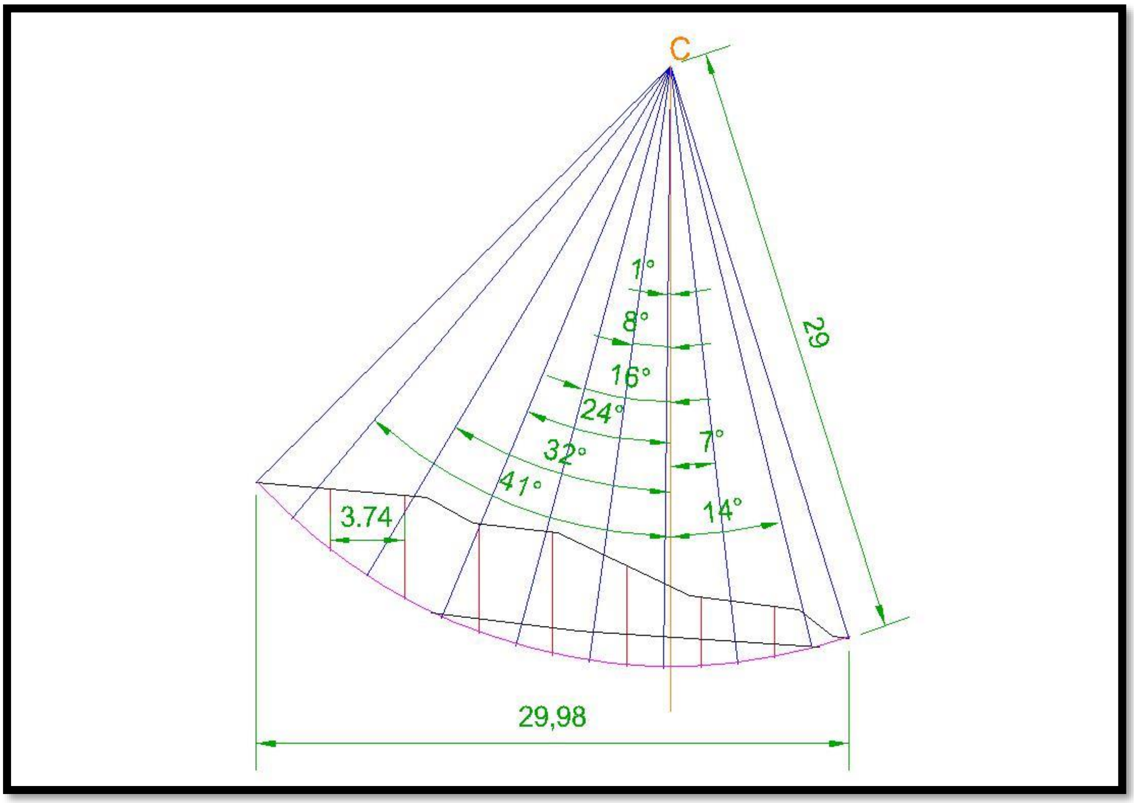

Figure 4 : Calculation of the slip circle by the slice methods, - Coordinates of the slip circle are; Center of the circle (25m; 38m), Radius 29m, B= 3.74 and L=29.98-.

The slice methods (Fellenius and Bishop) are the simplest and most practical due to their ease of implementation and reliability, always providing the best results for the safety factor [52, 53]. From the results obtained in Table (1), we observed that: The safety factor FS calculated by the Bishop method is higher than that calculated by the Fellenius method. Additionally, both safety factors FS (calculated by Bishop and Fellenius) are less than 1, indicating that the slope is unstable.

Table 1 :
Values of the safety factor calculated by different methods.

Fellenius	Bishop
0.70	0.81

Geology, Topography and Morphology of the Site, Climatological Context, Hydrology and Hydrogeology:

The set of topographic, morphologic, geological, geomorphological, and hydrological data allowed us to draw the following conclusions: the site intended for the construction of a municipal revenue office is located in the main agglomeration of the commune of Taher. Topographically, the site's relief has an irregular shape. The project is situated at the foot of a slope with an average to steep incline oriented towards the west, containing several small slopes. The overall slope ranges from 25% to 30%. The local geology consists of lithological formations from the Pliocene [54]. In terms of hydrology, the intensity of the hydrographic network is moderate, with a thalweg located at the foot of the slope, near the site, to the west. Climatologically, the study region is considered one of the rainiest. It has a temperate Mediterranean climate, with rainy and cold winters and hot, humid summers.

Correlation of dynamic penetration tests and borehole sections:

We found it useful to establish a correlation that allows for the comparison of penetrometric diagrams based on the penetration resistance criterion, with the depth of the resistant layer as it appears from the cross-sections made on the boreholes. The best way to proceed is to draw profiles that intersect the terrain in several directions. Thus, on the layout plan, profiles designated by double alphabetic letters are drawn: (A-A’): (S2, S3)-(P2, P4, P6)**.**

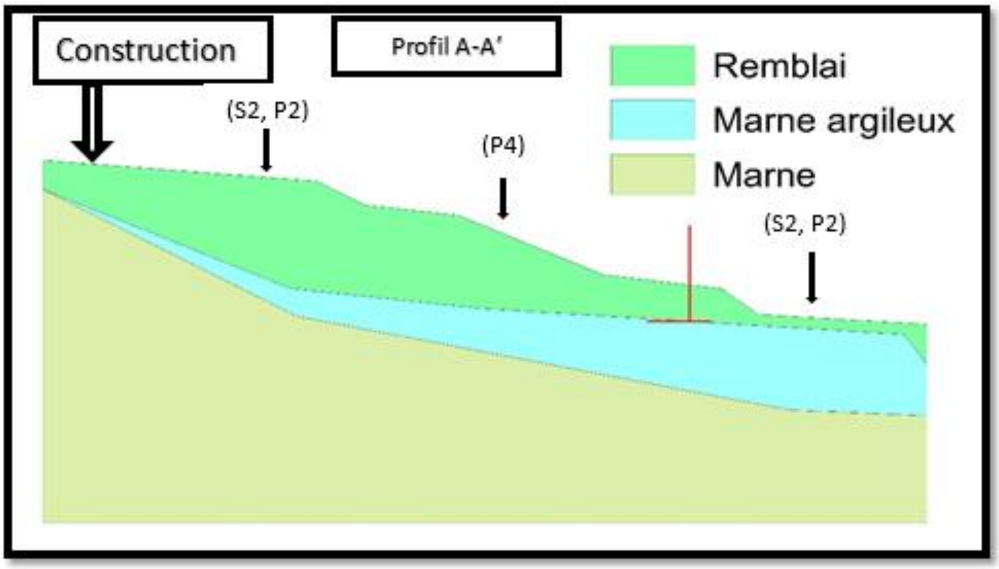

Figure 5: the A-A' profile.

The profile has a relief with a moderate to steep slope oriented towards the West. Geologically, it consists of clayey marl and sandy clays resting at depth on marls. This in-place formation is overlain by a thick layer of fill with a thickness of up to 6.0 meters. Dynamic penetration tests at this profile revealed variable resistances, very low in the fill with values of 10 to 20 bars, moderate in the clayey marl with Rp values ranging from 40 to 60 bars, and high in the marl at depth with values exceeding 100 bars. The piezometers installed at this profile revealed a low presence of groundwater, which consists of infiltration water from upstream areas.

Interpretations of physical parameters results

Water retention measurements (table 2) were carried out on different samples, yielding variable values. For clayey marl, the water retention is between 20% and 23%, and for marl, it is between 11% and 13%. The degree of saturation for clayey marl ranges from 68% to 79%, indicating a naturally moist state, while for marl, it is between 53% and 61%, indicating a moderately moist formation.

The dry density value for clayey marl is 1.50 t/m^3, while for marl, it is slightly better, ranging from 1.68 to 1.76 t/m^3. The apparent wet densities are as follows: for clayey marl, it ranges from 1.81 to 1.85 t/m^3, and for marl, it ranges from 1.88 to 1.97 t/m^3 (table 2). According to geotechnical standards, these soils are classified as semi-dense for clayey marl and dense for marl at depth.

Grain size tests were conducted for the different soil types. For clayey marl, the test showed that this formation has a fine texture, with 79% passing the 80 µm sieve and 94% passing the 2 mm sieve. For marl, more than 87% passes the 80 µm sieve (table 2).

Plasticity tests using the Casagrande apparatus on clayey marl gave a liquid limit value ranging from 42% to 44%, with a plasticity index between 22% and 23%. According to the Casagrande chart, this indicates a low plasticity soil. For marl, the liquid limit values are closer, ranging from 45% to 47%, and the plasticity index is around 24% to 25% (table 2). According to the Casagrande chart, these formations are of low to medium plasticity.

Soil activity is defined by the ratio of the plasticity index to the percentage of particles smaller than 2 µm. The soil consisting of marl has an average plasticity index of 24.97% and an average percentage of particles smaller than 2 µm of about 78.66%, giving an average activity (Ac) of around 0.31. According to Skempton, this indicates that the soil is inactive.

Table 2:

The results of the physical characteristics of the site layers.

Drilling		Identification							
N° drilling	depth (m)	**W** (%)	γ(t/m^3)	γ**d** (t/m^3)	Sr (%)	2 (mm)	0.08 (mm)	WL (%)	IP (%)
S1	2.5/3.0	28	1.78	1.37	83	/	/	/	/
	7.5/8.0	11	1.97	1.76	58	98	88	47	25
S2	7.5/8.0	13	1.93	1.70	61	97	89	46	24
S3	2.0/2.5	20	1.81	1.51	68	94	79	44	23
	5.0/5.5	12	1.95	1.72	60	95	87	45	24
S4	3.0/3.5	23	1.85	1.50	79	94	80	42	22
	6.0/6.5	11	1.88	1.68	53	97	88	47	24

Interpretations of mechanical parameters results

The characteristics of soils, including shear resistance and compressibility, directly determine the bearing capacity of soils in relation to acceptable deformation (settlement). To assess these characteristics, we used the Casagrande apparatus for shear tests and the Terzaghi oedometer for compressibility tests. Shear tests were conducted according to the standard NF P94-071-1, of the UU type, using a direct shear machine at a speed of 1.2 mm/min (see Table 3). Shear tests were performed on samples taken from the borings, using a shear machine and a Casagrande apparatus, where 60 mm diameter samples were sheared at a rate of 1 mm/min. The cohesion values for the clayey marls range from 0.30 to 0.35 bars, and the friction angle values range from 7° to 8°. These characteristics clearly indicate that this formation has a relatively medium cohesion and friction angle. For the deeper marl, the cohesion values range from 0.55 to 0.61 bars, and the friction angle values range from 8° to 10°. For the deposit, the intrinsic characteristics are low.

Table 3 :

Shear test results.

Drillig		Shear	
N° drilling	Depth(m)	C_{cc}(bars)	φ_{uu} (°)
S1	2.5/3.0	0.23	6
	7.5/8.0	0.57	10
S2	7.5/8.0	0.61	9
S3	2.0/2.5	0.3	8
	5.0/5.5	0.55	8
S4	3.0/3.5	0.35	7
	6.0/6.5	0.6	8

To assess the soil's propensity for settlement, compressibility tests were conducted on pre-saturated samples using the LPC procedure (incremental loading test). The consolidation pressure values for the clayey marls ranged from 1.64 to 1.71 bars, indicating that this formation is overconsolidated. The compressibility coefficients were moderate, ranging from 21% to 23%, which shows that the formation is moderately compressible. The swelling indices were below the 4% threshold, indicating that the formation is non-swelling. For the deeper marl samples, the values for consolidation pressure (Pc) ranged from 2.36 to 2.6 bars, the compressibility coefficient (Cc) from 15% to 18%, and the swelling index (Cg) from 2.6% to 3.6% (see Table 4). According to geotechnical standards, this formation is overconsolidated, has low compressibility, and is non-swelling.

Tableau 4:

Oedometer test results.

Drilling		Oedometer		
N° drilling	Depth (m)	σc (bars)	Cc (%)	Cg (%)
S1	7.5/8.0	2.48	16.59	2.6
S2	7.5/8.0	2.60	15.83	3.3
S4	2.0/2.5	1.71	23.37	3.6
	5.0/5.5	2.59	18.09	3.3
S5	3.0/3.5	1.64	21.61	3.3
	6.0/6.5	2.36	17.34	3.6

Interpretations of chemical results

Chemical analyses revealed a variable presence of carbonate, ranging from 31% to 48%. These analyses did not indicate any presence of sulfates; however, the soil is non-aggressive.

Tableau 5:

Presente l'interpretation des résultats chimique :

Samples	% of carbonates $CaCo3$	% of insolubles	% of gypsum $CaCo_4 2H_2O$	Sulfates SO4-103 mg/kg
S1(7.0/7.5m)	45.8	55	nil	nil
S2(7.0/7.5m)	47.5	52.4	nil	nil
S3(2.5/3.0m)	31.0	69	nil	nil
S4(3.0/3.5m)	33.5	66.3	nil	nil

The examination of all physical, mechanical, and chemical characteristics provides the following assessments: The analyzed soils are composed of formations with varying consistencies. The clayey marl formation is characterized by a fine texture, semi-dense dry density, and a naturally wet moisture content, indicating low plasticity. The marl found at depth is classified as slightly moist and dense. Mechanically, both the clayey marl and marl exhibit average cohesion and friction angle values. In terms of compressibility, these formations are overconsolidated, low to moderately compressible, and non-swelling. Chemically, the soils contain no sulfates, indicating no aggressiveness, thus normal cement is appropriate for infrastructure concrete.

Numerical simulation of the revised retaining wall:

The behaviour of soils under stress is often unpredictable and nonlinear [42]; this requires extraordinary efforts [43-45]. The undeniable advantage of numerical methods is their ability to solve problems that cannot be resolved analytically and to find approximate solutions [46-47]. The modelling of geotechnical structures using numerical methods is made possible by a set of assumptions regarding the geometry of the structure and its environment [48], the materials and their behaviours, the loads, the boundary conditions, and the initial conditions [49].

PLAXIS 2D is a two-dimensional finite element method (FEM) program specifically designed to perform deformation and stability analyses for various types of geotechnical applications. Real situations can be represented by a plane or axisymmetric model. The program uses a user-friendly graphical interface that allows users to quickly generate a geometric model and a finite element mesh based on the vertical cross-section of the structure to be studied [50].

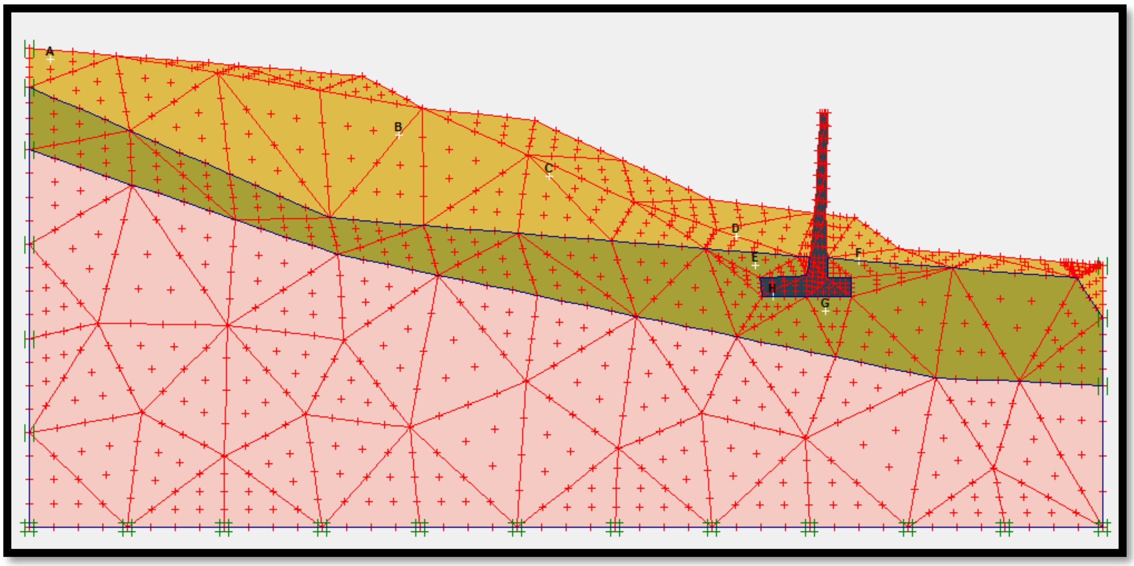

Figure 6: Designed Model and Behaviour *Tracking Points in our Model*.

Our modelling framework (Figure 6) has a total width of 40 meters and a height of 19 meters. It includes three different layers in its lithology, from top to bottom: a fill layer, a silty clay layer, and a clay marl layer, respectively (table 6). To revise the dimensions of the initial retaining wall, we modelled it with real dimensions instead of using plate elements to accurately observe its full-scale behaviour. The new height is 5.4 meters instead of the previous 4.6 meters, and the thickness is 0.4 meters instead of the previous 0.2 meters. Its foundation is anchored in the intermediate layer at a depth of 1 meter instead of the previous 0.4 meters in the superficial layer, with a front base of 2 meters and a rear base of approximately 1 meter. Our model is generated using a 15-node triangular mesh and is constrained by a system of boundary conditions to control the distribution of stresses and deformations, as well as to avoid undesirable effects at the model's boundaries. Our numerical modelling proceeds through four main phases to approximate real-time behaviour: Phase 1 involves the initial plastic phase, where there is no retaining wall over a period of seven (7) days. Phase 2 represents the installation of the retaining wall, adjusted over ninety (90) days, including the application of building loads above the slope (16 kN/m²) and traffic loads (6 kN/m²) below the slope. Phase 3 is the consolidation phase, adjusted for one hundred and twenty (120) days, to calculate the deformations of our system. Finally, Phase 4 involves calculating the Phi/c reduction to determine the safety factor of our system.

Table 6 :

The characteristics of the different soil layers.

Parameters		Backfill layer	Clayey marl	Marl
Behaviour		Mohr-Coloumb		
Type		Drained	Undrained	Undrained
ɣunsat	[kN/m³]	15,000	13,700	16,800
ɣsat	[kN/m³]	18,000	17,800	18,800
kx	[m/min]	8,000E-03	8,000E-03	8,000E-03
ky	[m/min]	8,000E-03	8,000E-03	8,000E-03
Eref	[kN/m²]	2,660E+04	6,400E+04	6,450E+04
Eoed	[kN/m²]	3,941E+04	9,483E+04	1,035E+05
ʋ	[-]	0,33	0,33	0,350
Gref	[kN/m²]	1 E+04	2,406E+04	2,389E+04
cref	[kN/m²]	5	12	20
ϕ	[°]	20	38	15
ψ	[°]	0	0	0

To monitor the deformations in our numerical model, we selected points A, B, C, D, and E on the surface of the fill layer, as well as points F and G below the retaining wall, to thoroughly understand the deformations around the retaining wall (Figure 6). Additionally, to track the stresses in our model, we adopted points I, J, K, L, M, and N in the fill layer, as well as points O, P, and Q around the retaining wall in the intermediate layer.

Results :

After the calculation, we found a total deformation of 444.73 x 10^3 with a scale effect of 2.10^-6, which gives us a total deformation of 0.89 m. It can be observed that this displacement occurs only in the surface layer of the backfill, with a direction going from the top of the slope to the foot of the slope, before the retaining wall, without any effect on the wall itself (Figure 7).

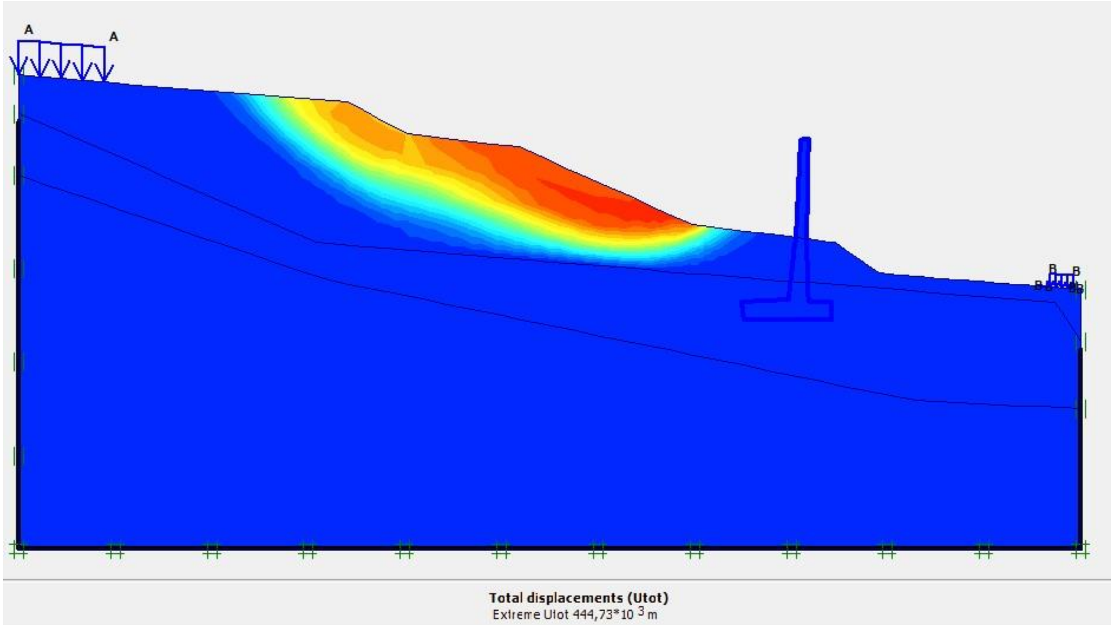

Figure 7: Total deformation of 0.89 m.

Thus, we observed that significant displacement occurs at tracking point A, located above (at the head of) the slope in the surface layer, and at point D, located directly behind the retaining wall. The other tracking points in the model can be considered negligible (Figure 8).

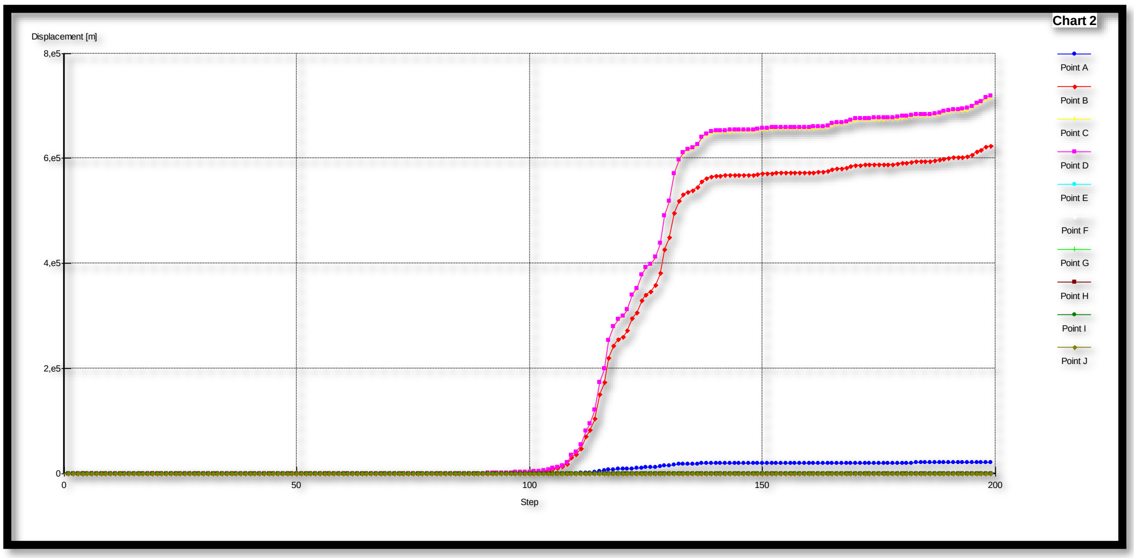

Figure 8: Deformation curves at the different monitoring points.

The safety factor calculated before the installation of the retaining wall is 0.58 (the red curve in Figure 9), which required us to find a solution to stabilize the slope. We chose the same solution proposed by the initial design office to determine if the problem lay in the study or in the execution, and thus conclude on the appropriate solution based on our expertise. The safety factor calculated by our model after the installation of the retaining wall is 2.11 (the blue curve in Figure 9). This safety factor value indicates that the retaining wall is largely stable and that the deformations occurring behind the wall do not affect its stability, which lends credibility to our model.

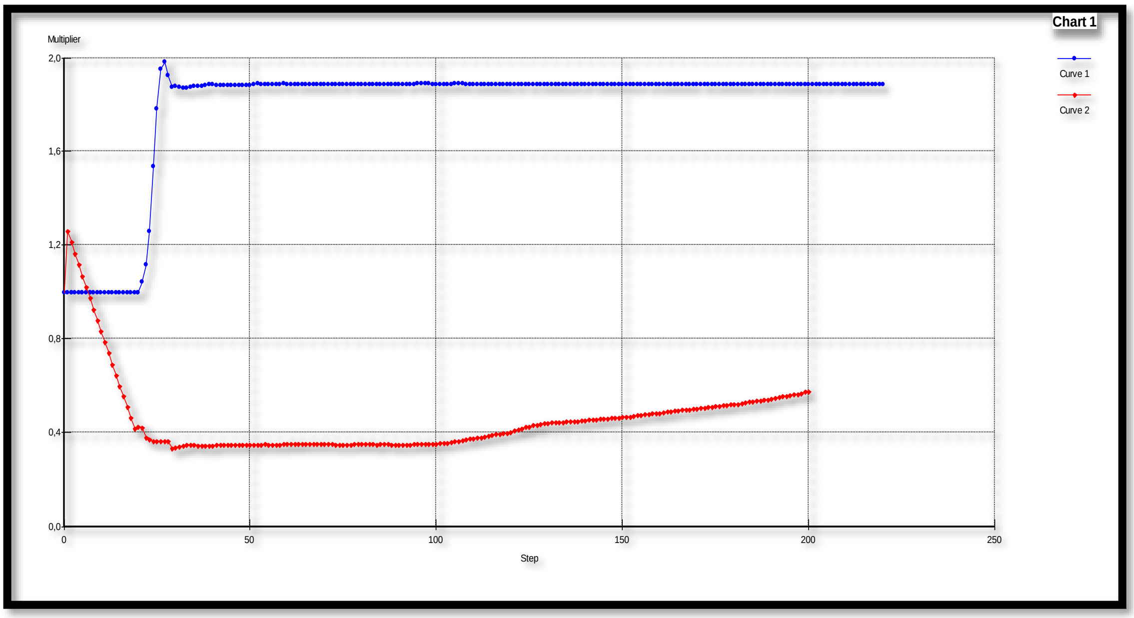

Figure 9: Safety factor before installation (red) with a value of 0.58 and after installation of the retaining wall with a value of 2.11.

Discussion

The wall intended to support the administration of the Treasury of Taher in the province of Jijel in Algeria against the sliding of the embankment behind the Treasury is partially overturned without visible deformation, which led the project owner (APC of Taher) to suspend the project by freezing the allocated funds and to open an inspection to determine the real cause of the problem, whether in the initial study or in the poor execution. Despite the counter-expertise carried out by several engineering offices and civil engineering experts on the ground, no satisfactory solution has been found to solve this problem. This prompted us to undertake investigation to identify the real source of the problem.

Figure 10: the investigation site.

Upon visiting the site and conducting our diagnostic of the wall (see photos in Figure 10), we discovered the presence of a plastic sheet - plastic tarpaulin - behind the wall, along half of the slope, sealing the surface layer of the backfill and blocking the dissipation of rainwater upwards. This created pressure on the wall in the surface layer, particularly at monitoring point D in our model. We also noticed that the retaining wall is anchored only in the backfill layer and barely touches the intermediate layer. This led us to realize that the pressure exerted by the soil under the rear footing of the retaining wall after significant rainfall can cause superficial and partial sliding in the area of point D in our model, resulting in a partial and slight overturning of the left part of the retaining wall without visible deformation (see Figure 11 for photos of the plastic sheet and the toppled wall).

Figure 11 : plastic tarpaulin.

By comparing these observations with the predictions from our model, we find a logical correlation, particularly at point D as previously mentioned, as well as at point A where we observed deformations in the foundations of buildings at the top of the slope (see Figure 8 for deformation curves and Figure 12 for photos of the deformed houses).

Figure 12 : distorted house photos.

It should be noted that our retaining wall model is larger than that of the initial design office, with an increased height of 0.80 m, a greater thickness of 0.20 m, and a deeper foundation of 0.60 m, anchored down to the intermediate layer. This provides greater stability to our model compared to that of the design office.

Figure 13 : photo of the adjacent administration.

In summary of our diagnosis and expertise, we identify errors both in the study and in the execution. In the study, the design office failed to correctly size and provide adequate anchoring for the base of the wall, which should have been deeper down to the intermediate layer. Additionally, there appear to be potential errors in the execution, particularly concerning site preparation and the inappropriate use of a plastic sheet instead of a geotextile to facilitate the drainage of the backfill. This conclusion is supported by the intact state of the adjacent administration, visible in the condition of its services, which shows no signs of sliding (see Figure 13 for photos of the adjacent administration). It is noteworthy that this administration is located lower than the Treasury and had excavated a thinner superficial layer of backfill.

Conclusion:

This in-depth study on the stability of a retaining wall in Jijel Province, Algeria, reveals several critical aspects in the design and execution of urban constructions. The partial overturning of the wall, without apparent deformation, was a major concern that led to the suspension of the Taher Treasury Project. Evaluations highlighted deficiencies in both the initial study and on-site execution. The identification of a plastic sheet behind the wall, hindering proper rainwater drainage, was a significant discovery. This situation exacerbated pressures on the superficial layer of the backfill, contributing to the observed superficial sliding. Additionally, the insufficient anchoring of the wall in the intermediate layer compromised its stability under local geotechnical conditions.

The comparison between field observations and our full-scale, real-time numerical model—different from the initial failed model, which was only plate-element based—emphasized the importance of a robust design with adequate dimensions and appropriate anchoring. Our proposed revised model, larger, deeper, and better anchored, aims to rectify the identified shortcomings, offering better resilience against slope sliding risks.

Ultimately, this study underscores the crucial importance of rigorous geotechnical planning and adherence to standards in urban projects. The lessons learned from this analysis should guide future decisions to avoid similar failures, ensuring stability during construction and the durability of buildings in morphologically challenging and geotechnical complex urban areas.

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