

HOW THE “BIREH” LANDSLIDE QUIT MOVING

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Abstract: Every year, a recurring landslide damages a portion of the road stretching about 50m in Bireh, north of Lebanon. This has led to many car accidents and injuries in addition to costly damages to the infrastructure and power lines. Several attempted remedial procedures proved to be inefficient over the past five decades.

A well-engineered solution became necessary before the inauguration of the new road passing through Bireh. The solution was based on site explorations, theoretical models with back-analysis approach, construction, and monitoring phases. This solution consisted of installing a support system with 3 rows of vertical and battered piles reaching the bedrock at the bottom, connected at the top by a solid concrete slab, and complemented by deep drainage trenches. Monitoring during the past winter season proved that the maximum displacement of the slab did not exceed 3 mm, which represents about 10% of the predicted displacement for the most critical conditions. This optimized solution finally solved the problem and is kept under monitoring for evaluating future performance. This paper summarizes all phases of the solution and gives new insight to what could be the reason behind landslide occurrences and how to stop them.

INTRODUCTION

Project Description

In the summer of 2004, the Council for Development and Reconstruction (CDR) awarded a 12 million dollar construction project for widening and improving the main road connecting the vilalge of Halba to the mountainous village of Qobayat, in north Lebanon. The work started that summer with great enthusiasm and political support. Towards the end of that road sits the village of Bireh which is very well known by locals for its treacherous landslide. The landslide has been actively moving over the past five decades and possibly earlier, occurring at the same location around station 4+780. A general view from the North side, which is the valley side, is shown in Figure 1. The landslide slope averages 19 degrees. Its area affects about 50m of the road length and extends uphill from the road about 80m. The slope failures appeared almost invariably every winter season, especially after heavy rains. Due to the large displacements, as shown in Figure 2, road crossing in this zone became hazardous, and led to many car accidents and injuries, in addition to costly damages to the infrastructure, like drainage ditches, sewer conduits, telephone and even power lines. Quantitatively such landslide would have a relatively high hazard score of 225 according to the Ohio DOT system (Liang *et al.* 2006). In the past, all concerned parties called for quick remedial procedures like backfilling and asphaltting, or surface soil replacement, or surface drainage; which all proved to be insufficient solutions. In the winter of 2005, the project of construction of the new road reached the landslide zone. In order to successfully stabilize this critical portion of the new road, a specialty contractor known for its creative and well engineered design/build solutions (Ballouz 2005), was hired for this job.



Figure 1. General View from North Side (see Figure 3)



Figure 2. View from West Side (see Figure 3)

Soil Formation

The site is located in Northern Lebanon, about 30 km north of Tripoli. The geological formation in this region is a dolerite Basalt deposit of the Pliocene era. A soil investigation was conducted by SOMECA with 5 boreholes taken along the landslide centerline, 3 above and 2 below the main road, as shown in the plan view of Figure 3, with depth varying from 10 to 12 meters. The P on Figure 3 designates the exploration being done with Cone Penetration Test, and S designates drilling with coring and in situ pressuremeter testing. Three layers were detected: 1- silty clay (3 to 5m), 2- conglomerate of basalt fragments and clayey sand (8 to 10m), and 3- basaltic bedrock. Under the road it seems that about 5m of gravelly backfill had been installed in the past and in stages to level the displaced road. Lab and in-situ testing provided insight to the soil parameters and characteristics, with representative values shown in Table 1.

The water table in May 2005 varied between 7.5m at the landslide top (S1), and 2.75m at (S2) to about 1m near the road (S5). It is possible that during a heavy rainfall this perched water could saturate the top soil layer all the way to the surface. This condition would be considered the most critical one in this study.

Table 1. Soil Data

Soil ID	Description	Thick (m)	Modulus E (Mpa)	Limit Press. p_L (Mpa)	TCR %	RQD %	Cone tip R_p (MPa)	Moist. w_n %	LL %	PL %	Cohesion c (kPa)	Friction ϕ (deg)
1	Silty Clay	3 to 6	5	0.5	15 to 30	0	20 to 50	25	55	20	10	20
2	Backfill	~5m of gravelly backfill under the Road only						Assumed :			0	30
3	Conglomerate	7 to 9	25	1.5	30 to 50	0 to 20	50 to 150	30	-	-	10	35
4	Basalt Rock	>10	>40	>4	60 to 80	0 to 34	Refusal	-	-	-	50	35

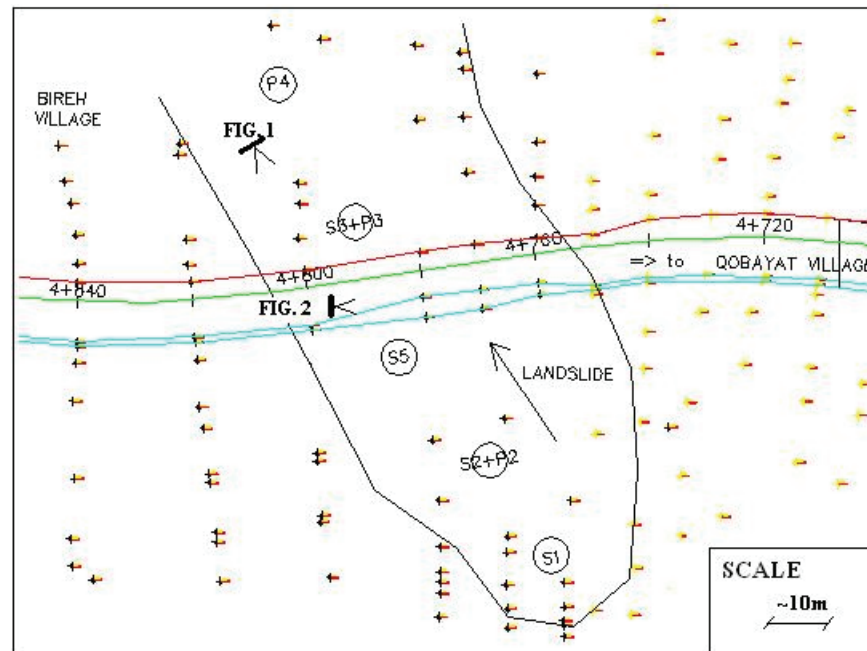


Figure 3. Plan View of Landslide Area

DESIGN APPROACH

With the limited budget available, the challenge was to find an optimized solution within the right of way limits since adjacent terrains are private lots and could not be altered. The solution should be capable of resisting the lateral forces of the landslide as well as the vertical loads of the roadway traffic. The solution proposed by is illustrated in Figure 4 and consisted of actually nailing the top weak soil layers to the lower strong strata. This can be achieved by using a combination of battered and vertical piles, connected by a rigid platform at road level, and complemented by a drainage system. This multi-directional approach gives a hidden redundant safety that might be needed under extreme conditions. In the author's opinion, when failure is already present such approach is highly recommended. This multi-directional approach had its efficiency proven lately on another similar landslide problem (Rasplicka *et al.* 2004). Additionally, Laudeman *et al.* (2004) demonstrated that multiple rows are more efficient than a single row of piles in resisting slope failure.

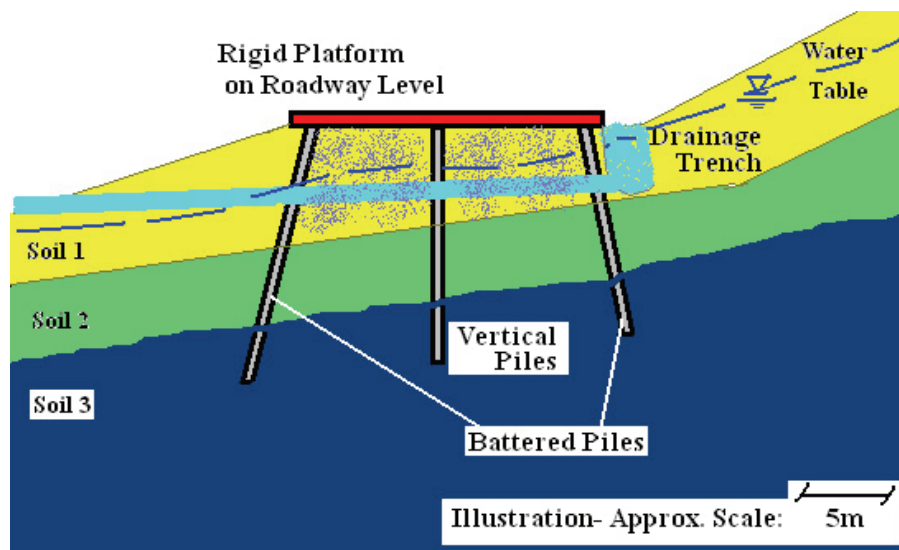


Figure 4. Proposed Solution

The design was made by solving two modes of possible failures: the slope stability, and/or the structural integrity of the system. In the following paragraphs, both issues would be treated in order to ensure the long term viability of the solution.

Slope Stability

The slope stability was modeled and checked utilizing the TALREN software by *Terrasol*. This software is guided by the French recommendations presented in the research program CLOUTERRE, (1991). It analyzes the limit state equilibrium to predict the safety of a slope stability using in this case the well known Bishop's method of slices.

The initial step was to recreate the present situation with a safety factor close to one representing the slope at the edge of failure. In fact, by recreating the geometry of the slope along the landslide centerline, using the soil parameters presented in Table 1, a model was prepared to represent the actual situation in May 2005 with the water table set at that date. This model is presented in Figure 5. As Figure 5 shows, when setting all the partial safety factors equal to one, the global safety factor ϕ_{\min} obtained was $\phi_{\min}=1.51$. This value indicates that the landslide under unsaturated condition is safe, which was the case in the spring of 2005.

Another model was prepared to simulate a storm with heavy rainfall by saturating the slope to the surface. This model is presented in Figure 6. In this model the global safety dropped below the failure envelope with $\phi_{\min}=0.94$; which, theoretically explains why the Bireh landslide is active only after heavy rains, occurring at a transient stage between the conditions presented in Figures 5 and 6. Recently L'Heureux *et al.* (2006) studied this transient flow using numerical analysis for a slope subjected to heavy rainfall in order to investigate a slope failure in Norway. This approach of comparison between actual and critical conditions is also known as a back-analysis approach that permits validating the sensitive variables of the problem like the soil parameters and pore pressures. Slingerland *et al.* (1982), Desai *et al.* (1995), Chen *et al.* (2002), Tiwari *et al.* (2005) and L'Heureux *et al.* (2006), amongst others, have relied on back-analysis methods for solving slope stability problems.

A third slope stability model was put together for the design analysis. This time and according to the French recommendations CLOUTERRE 1991, the partial safety factors were

applied and the global stability factor had to exceed the value of $\gamma_{min}=1$. The hunt for the optimal geotechnical design was determined by fixing the pile length, and varying the pile diameter and spacing. The TALREN model was prepared as shown in Figure 7 to represent the optimized solution. Optimization was attained when the global safety fell slightly above the requirement ($\gamma_{min}=1.02$), under the most critical conditions; that is, heavy rainfall and full soil saturation.

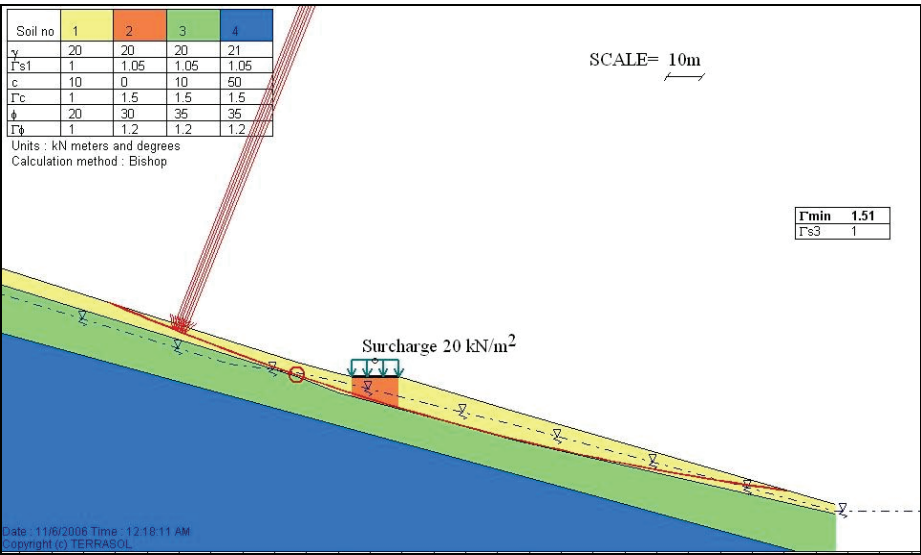


Figure 5. Actual Situation

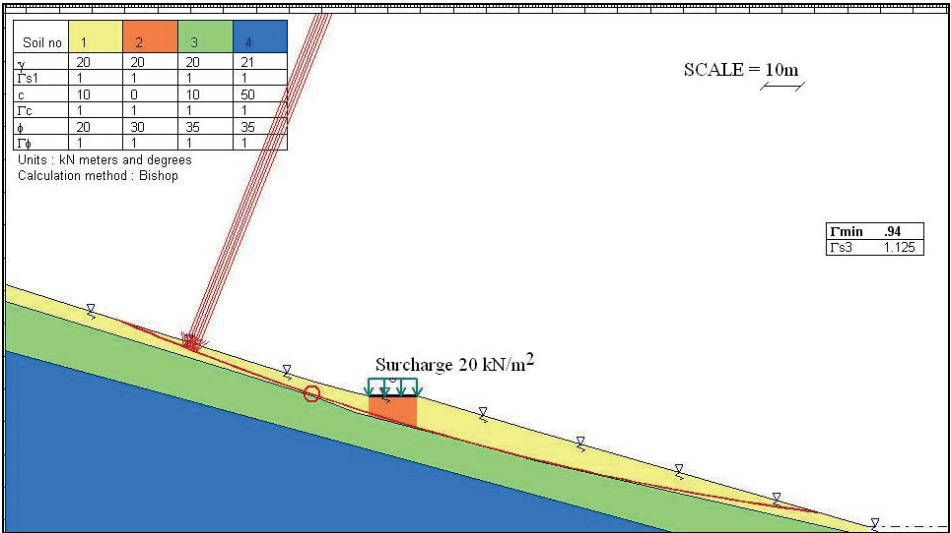


Figure 6. Saturated Condition

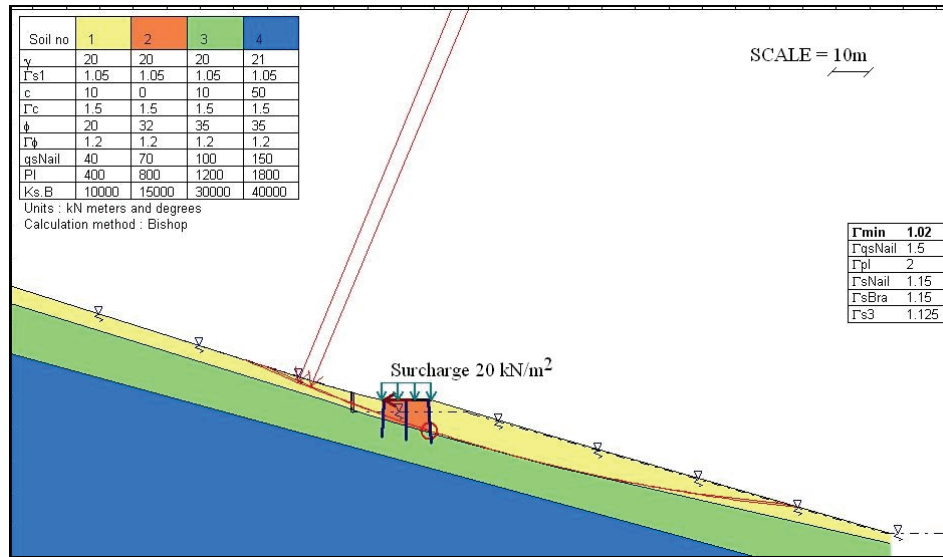


Figure 7. Solution Model under Saturated Conditions

The pile length was set as to ensure on site at least 3m embedment into the bedrock layer. According to the initial soil investigation an average pile length of 12m was expected. Assuming a road surcharge of 20 kN/m², a global safety factor $\Gamma_{min}=1.02$ was achieved as shown in Figure 7, with the following support system: 3 rows of 600 mm diameter piles, two of them being battered, with a spacing of 2.5m c/c between piles, all connected at the top by a 400 mm thick slab-on-grade. Also a drainage trench was to be installed towards the uphill side with the role of releasing excess pore water pressures near the roadway and around the support system.

Structural Integrity

The system was subjected to vertical as well as lateral loading. A structural 2D frame representing a typical section of the support system was considered. This frame was analyzed by the Finite Element method using Arch/Effel structural software. Assuming small strains, the soil and rock material were modeled as elastic springs acting laterally and vertically and attached to discrete elements on the piles. By considering the geometry, the surcharge load, and the lateral earth pressure on the piles, a soil structure interaction behavior was obtained. The output given in Figure 8, shows that under the most critical conditions, the maximum displacement of the structure is in the horizontal direction at the top, and is expected to reach 32mm. Such displacement remains within tolerable limits for piles that are 12m long. As supplemental information, the expected moment diagram for the structural system is presented in Figure 9.

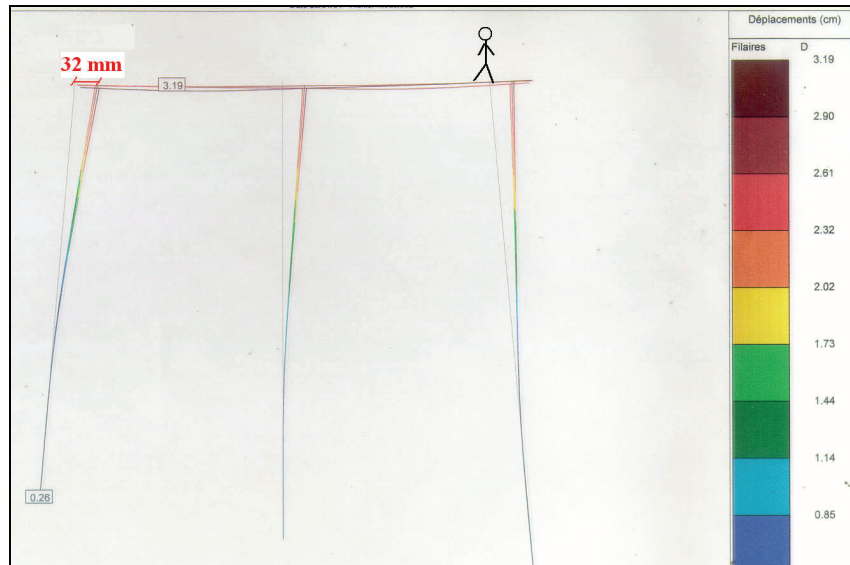


Figure 8. Expected Maximum Displacements

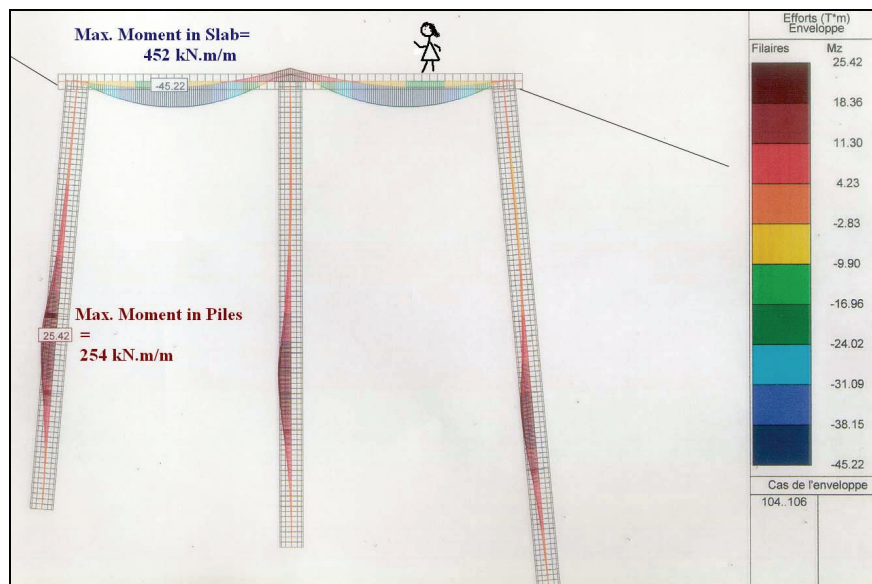


Figure 9. Expected Moment Diagram under Critical Conditions

CONSTRUCTION PHASE

The construction phase started with the installation of the drainage trench by the main contractor. This was important since, as mentioned earlier, the perched water table was near the surface around borehole S5 on the south side of the road.

The piles were installed using a Soilmec R10 Drill rig shown in Figure 10. The pile installation went smoothly with little construction difficulties except for the discovery of very deep bedrock within the landslide zone. This was unexpected and led to the belief that actually the Bireh landslide is nothing but a volcano crater filled with erosion deposits with weak shear strength characteristics, ready to slip when saturated. A graphic representation with a plan view and section view of the 3 rows of piles is given in Figure 11. The different soil layers

encountered during drilling are also indicated in Figure 11, where the varying bedrock depth delimiting the landslide is apparent.

It should be noted that four underground water sources were discovered during the pile drilling operations as shown in Figure 11. These water sources were found at a depth of about 5m below the road level. This discovery proved that the 5m deep drainage trench on the uphill side played its role.

After the pile installation, the pile heads were cut-off and the grade preparation put in place. This phase of the construction is shown in Figure 12.

A slab-on-grade was then poured to connect the pile heads and create a monolithic structural frame. The slab was a reinforced concrete slab 400mm thick. Its surface was marked for future monitoring of the displacements.



Figure 10. Pile Installation

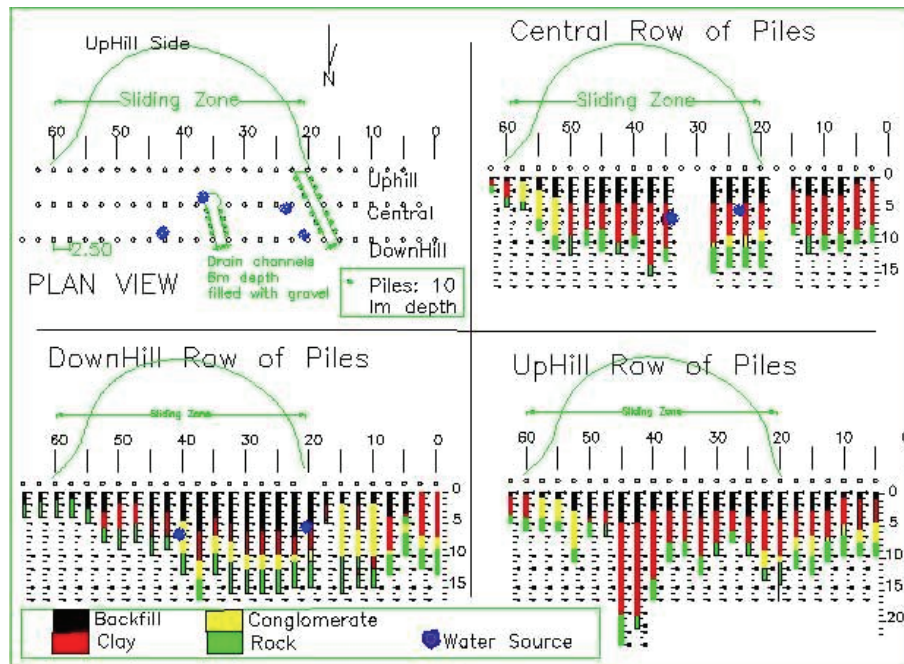


Figure 11. As-Built Drawings of Pile Support System



Figure 12. Final Layout of the Piling System

SYSTEM PERFORMANCE

The works were completed by October 2005, and the support system was kept under close supervision by monitoring the top displacements of the slab at regular intervals. Throughout the winter of 2005-2006 and up to the date of writing this paper, October 2006, the maximum displacement measured was in the order of 3 mm found at the southwest corner.

This measured displacement represents almost 10% of the predicted displacement; however, this doesn't mean that the system was over designed. The design was made to withstand the most critical conditions which have not been reached this past winter. The 3D effect of the

structural element, bridging horizontally above the landslide zone as shown in Figure 11, is believed to have provided extra rigidity to the landslide support system for the Bireh road. This extra rigidity may have provided an extra hidden safety in the solution. The performance of the support system will be monitored for the next 5 years. Already this year the slope has been soaked with the first rains of its second winter season 2006-2007... without a problem!

CONCLUSIONS

An old recurring landslide was stopped by implementing a well engineered solution comprising four phases: Site exploration, design using back-analysis, construction and monitoring. Such solution proved to be successful on this particular project. This paper summarizes all four phases of the solution and gives new insight to what could be the reason behind landslide occurrences and how to stop them.

It is recommended in the future to model the landslide using finite element models with elasto-plasticity, where the soil and the structure could be combined. Also, the instrumentation of the piles is recommended for similar problems since it allows evaluating the performance of the system with greater details. Unfortunately, a user-friendly three dimensional finite element software that can simulate the heterogeneity of the soil layers and the pile 3D geometry, is not readily available yet. Such software would have been an interesting tool for studying this problem and explaining the discrepancy between measured and predicted displacements.

Acknowledgments: The author would like to acknowledge many individuals who made the evolvement of this paper possible. First the client, Mr. Antoine Makhoul, and his son, Roger, who had previously been my student and now is a engineer. The input and cooperation of engineer Richard Nehmeh, the project manager representing the Makhoul Enterprise is greatly appreciated. The finite element modeling done by engineer Charles Khoury, and the slope stability modeling prepared by Int'l IGM engineer Adib Osta were essential to reach the successful design conclusions for this project. Representing Int'l IGM on site was engineer Pierre Nahas, who kept a close watch on the well being of the project and prepared the as-built documents. Finally, my wife Hala, who is always by my side providing the necessary stamina to go further. They are all an integral part of making this project successful.

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