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Consolidation project of an urban landslide: Case study on Tetarom Park Cluj-Napoca

Projet de consolidation pour un glissement de terrain urbain : Etude de cas pour Tetarom Park Cluj-Napoca

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ABSTRACT: Urban development of Cluj-Napoca had a sustained rhythm over the last decade leading to several building projects to be started on areas avoided in the past, due to landslide hazard. One such case was Tetarom Industrial Park, built on the northern side of the city. Vertical systematization required to build the industrial park, poor groundwater management and unpropped man made interventions triggered a new landslide, affecting both the investigated perimeter and neighboring areas. The landslide was of detrusive nature, of great depth, with the sliding plain at the contact between the reshuffled material above and the base rock (marls, sandstones). After a complex site and laboratory investigation, a retaining wall system was proposed based on multiple analyses, following the concept of “worst case scenario”. Beside this, a drainage system it was proposed on the upstream side of the consolidation system. The project was completed in May 2020 and is constantly monitored by inclinometer measurements in order to validate the efficiency of the consolidation system. This paper presents some aspects regarding this case study in order to put in light a proper judgment regarding landslide hazard associated risk for Transylvanian hills.

RÉSUMÉ : Le développement urbain de Cluj-Napoca a connu un rythme soutenu au cours de la dernière décennie, conduisant au plusieurs projets de construction sur des zones évitées dans le passé, en raison du risque de glissement de terrain. Le parc industriel Tetarom, construit au nord de la ville, en est un des exemples. La systématisation verticale nécessaire pour construire le parc industriel, la mauvaise gestion des eaux souterraines et les interventions humaines ont provoqué un nouveau glissement de terrain, affectant à la fois le périmètre étudié et les zones voisines. La déformation du talus était de nature detrusive, de grande profondeur, avec le plan de glissement au contact entre la couche mélangée au-dessus et la roche de base (marnes, grès). Après une investigation complexe sur site et en laboratoire, sur la base de multiples analyses, un système de mur de soutènement a été proposé, suivant le concept du “le pire scénario” et en plus un système de drainage en amont. Le projet a été finalisé en mai 2020 et est suivi par des mesures inclinométriques afin de valider l'efficacité du système de consolidation. Cet article présente quelques aspects concernant l’étude de cas pour de mettre en lumière un jugement approprié concernant le risque associé aux hasards de glissement de terrain pour les collines de Transylvanie.

KEYWORDS: urban landslide, pile retaining wall, geotechnical investigation, long term monitoring

1 INTRODUCTION

The natural disasters with one of the highest occurrences in Romania are the landslides (Olinic & Manea 2018), due to erosion, but also to human activity. The impact of landslides has both economic and social aspects, and they are the main reasons why construction projects have been avoided in those areas. The consolidation systems are expensive, but the cost of the land have become low compared to near sites, therefore construction projects started on or near slopes with high sliding risk (Popa & Ilies 2010, Muresan 2011, Ilies & Farcas 2016).

In the past decade Cluj-Napoca, have faced the problem of extensive real estate developments in the hilly area, at the border of the traditional urban settlement. The main reason that in the paste those areas were avoided, is the medium to very high slope instability. The specific geotechnical profile of Transylvanian hills, indicate this aspect (Molnar & Ilies 2011, Moldovan & Ilies 2015), the site of the project being on the northern side of Hoia Hill, along the Somesul Mic River. The Hoia Hill, as other hills surrounding the city are well known as sites with sliding risk, due to slope and geotechnical parameters and they mapped accordingly in local and national planning regulations (General Urban Plan Cluj-Napoca 2021).

The case study presented in this paper analyze the stability of the site and according to the obtained results, the consolidation solutions for the urban landslide. The Tetarom Industrial Park was built since 2008 on the northern side of the city. This area was known for instability phenomena, the first landslides on the slope can be tracked back to the Pleistocene era. Vertical systematization required to build the industrial park, poor groundwater management and unpropped man made interventions triggered a new landslide, affecting both the investigated perimeter and neighboring areas, as in Figure 1.

Figure 1. Project area on north side of Hoia Hill.

The landslide had a length of about 130 meters, with a width of 155m, as shown in Figure 2. The slope deformation was of detrusive nature, of great depth, with the sliding plain at the contact between the reshuffled material above and the base rock (marls, sandstones).

Figure 2. Landslide occurred in March 2016.

2 MATERIALS AND METHODS

2.1 *Geotechnical investigation*

The project site has a general slope ranging from 10^0 to15^0, with a south-north orientation, also including considerably steeper embankment zones with over 60º inclination. According to Romanian norm (NP 074 2014 and SR EN 1997-1 2006, SR EN 1997-2 2007) the general and local slope indicate a high potential of sliding. From the geological point of view the slope includes Neogene and Paleogene aged sediments. These deposits have a diluvia nature, and the landslides affecting the area have mobilized not only the top part of the sediments, but older formations also, thus forming a stack of considerable thickness, without an individualized stratiform structure.

The geotechnical investigation conducted on site included 11 deep drillings, with a maximal depth of 25 meters, and 13 heavy dynamic penetrations (DPH), with a maximal depth of 16 meters. The study revealed seven distinct layers of soil, each further divided in subcategories according to intrinsic varying of geotechnical parameters.

The top layer consists of clay filling mixed with gravel (layer 1a), found in the majority of drillings, only excluding two of them. This is followed by a brownish-greenish colored silty-sandy clay, a very active layer considering its swelling potential, which also includes volcanic tuff fragments (layer 2). The consistency index indicates a consistent-hard soil. Residual values considered for slope modelling after landslide start were: ϕ_{rez}=11,15º; also, for layer 1 was take into account the same value for residual parameter.

The next layer consisted of a medium thickened red-brown sand (layer 5), present only in some parts of the slope. Its mechanical characteristics were analyzed following the dynamic penetration test results.

The next layer was a sandy-clayey silt, with very active clay intercalations, followed by a continuous layer of green colored strong but fragmented volcanic tuff. Underneath lie two layers of medium to weakly cemented sandstones. The identified base layer which follows the entire length of the slope is a grey slightly clayey marl (layer 7). For this layer direct shear tests offered values of friction angle ϕ'=19÷33º and drained cohesion c'=38÷72 kPa.

In figure 6 is presented a geotechnical cross section with this complex stratigraphy and in table 1 are presented the geotechnical parameter used for modelling.

The site is characterized by a significant groundwater table, influenced by heterogeneous stratification, which allows the waters on the slope to seep into the permeable layers. In the drilled boreholes, the groundwater table was found between -3.00m (F1) and -9.50m (F8 and F9). In the F9 and F11 boreholes, a second groundwater table was intercepted, at -14.00m (F9) and -17.70m (F11), respectively.

The investigated site is characterized by intercalated stratification, with decimeter-metric alternations of layers with varying grains size and geotechnical characteristics. It was difficult to make a correlation between drillings, given that the soil layers have been altered by many landslides that have affected the site over time. The alternations of cohesive and non-cohesive layers and the presence of groundwater table, favor the appearance of landslides.

The landslide had a length of about 130 meters, with a width of 155 meters. General sliding depth was estimated at 14-15m from ground level, fact partially confirmed by initial inclinometer measurements on the wright side of the landslide. The landslide was of detrusive nature, of great depth, with the sliding plain at the contact between the reshuffled material above and the base rock (marls, sandstones), having a random shape of the breaking surface, but also with retrograde elements. The diverse stratification revealed in the studied profiles is the result of the remixing of sedimentary layers from the top as the landslide took place.

2.2 *The slope consolidation system*

The consolidation system of the slope consists of three retaining structures arranged as follows: SSB1 - upstream the existing industrial buildings, SSB2 - in the middle of the slope and SSB3 - downstream the forest (see fig.10 Slope consolidation system). In addition to the massive support structures the project is completed with measures to eliminate groundwater and surface water from the slope.

The SSB1 retaining structure, as seen in Figure 3, has 192 type 1 piles, with a diameter of Φ1200 and a length of 26,40m and 18 type 2 piles, with a diameter of Φ1200 and a length of 11,40m. Type 1 piles will be tied at the upper part by a raft foundation having a thickness of 1,55m, respectively type 2 piles by a cap beam having a thickness of 1,55 m. The piles will be embedded in a layer of light gray marl, very hard, respectively in a layer of whitish, gray sandstone. Also, in the area upstream the retaining structure it was designed a drain, with a rifled tube at the bottom. A ditch will be built over the drain, which will have the role of collecting rainwater from the slope.

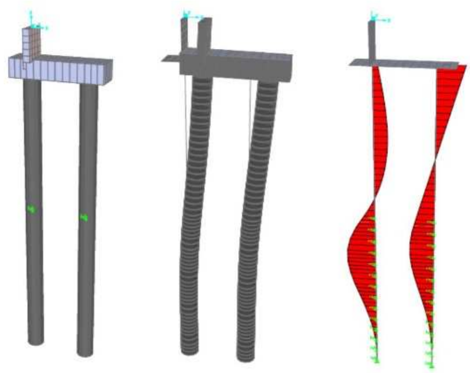

Figure 3. SSB1 Retaining structure -cross section.

The SSB2 retaining structure, as seen in Figure 4, has 181 type 3 piles, with a diameter of Φ1200 and a length of 26,40m and 17 type 4 piles, with a diameter of Φ1200 and a length of 11,40m; the SSB3 retaining structure, as seen in Figure 5, has 191 type 5 piles, with a diameter of Φ1200 and a length of 27,40m and 38 type 6 piles, with a diameter of Φ1200 and a length of 20,05m, with the same raft and cap beam on top and similar embedment.

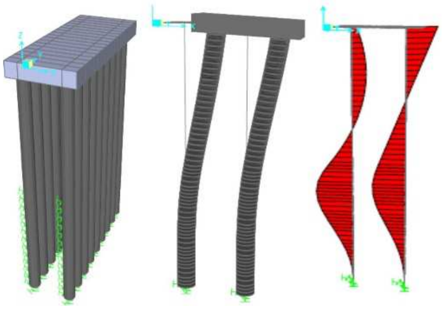

Figure 4. SSB2 Retaining structure -cross section.

For SSB 3 it was designed 96 permanent anchors with individual length of 30m, with 1,80m spacing. The anchorage length of the reinforcement (bulb) is 18,00m and the free length 12,00m; the diameter of the drilling is 150mm. The anchor is made of 7 strands of TBP 9 or similar. The bearing capacity of the anchor is 500kN, the nominal tensile force is 200kN (prestressing force).

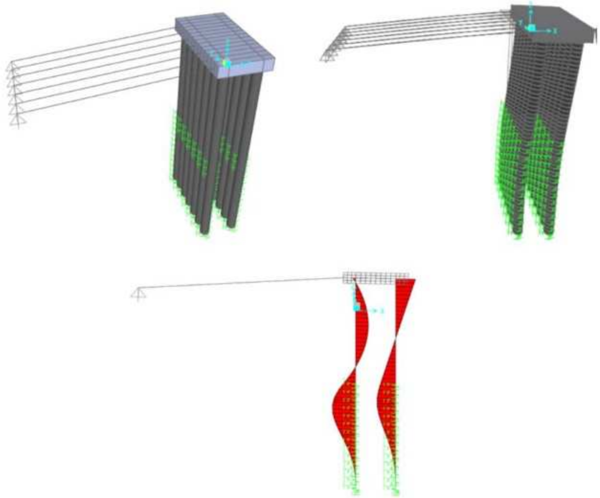

Figure 5. SSB2 Retaining structure -cross section.

The concrete class used for the retaining structure is C25/30, and BST500C and OB37 steel.

As drainage system there were designed 44 siphon drains with a depth of 12m, with 5,00m spacing. On the maximum slope line 5 drainage ditches will be made. They will have a width of 0,60m, a depth of 2,00m; at the bottom a rifled tube will be placed on a 10cm layer of sand. The ditch is covered in a geotextile and filled with drainage material. At the top it is not sealed, but the geotextile material is closed by overlapping and sewing with 1,5mm thick wire and covered with ballast. In the upstream part of the road near the forest, a ditch for collecting rainwater will be made.

The monitoring project includes 22 inclinometric tubes integrated in the piles, in a 110mm protective steel pipe. The annular space between the protection pipe and the inclinometric tubing will be filled with sand, having 0-3mm size, mixed with cement grout. The length of the inclinometers will be approx. 26m.

2.2 *Numerical modelling*

The event including landslide triggering and the consolidation of affected slope were numerically modelled in plain strain FEM framework to capture the mechanical behavior in detailed manner. Thereby, the geological section of the slope was modelled in plain stain analysis using triangular 15 nodes element, and the soil mechanical behavior was replicated by linear elastic-perfect plastic model (Mohr-Coulomb model) in addition to Tension Cut Off criterium (admitting null tension strength), in effective stresses. The complex stratigraphy of the sloped terrain is presented in Figure 6.

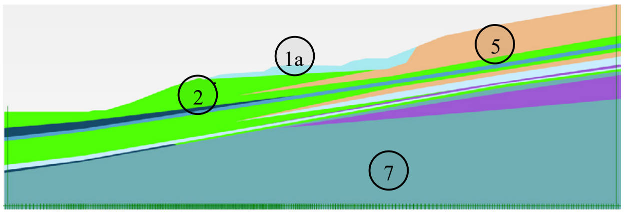

Figure 6. Soil stratigraphy of the analyzed section.

The geotechnical parameters, both mechanical and physical, used for MC model with Tension Cut Off criterium are given in table below.

Table 1. Geotechnical parameters values for main layers

Geotechnical parameter	*Layer 1a*	*Layer 2*	*Layer 5*	*Layer 7*
γ_{sat} (kN/m^3)	18.54	18.98	20.45	20.85
E (kN/m^2)	8511	9569	9950	26000
ν (-)	0.30	0.35	0.30	0.30
G (kN/m^2)	3273	3544	3827	10770
E_{oed} (kN/m^2)	11.46E3	15.3E3	13.39E3	37.69E3
c_{ref} (kN/m^2)	10	21.00	20.00	54.69
φ (°)	15	19.00	31.85	19.64
ψ (°)	0.00	0.00	3.00	0.00
φ_{rez} (°)	11.15	11.15	29.30	-

First, it was performed a slope stability analysis, considering an uprise of the ground water table (GWT). The arise of the GWT is denoted by an extreme climatical event, with high quantity of precipitation discharged in a short time. The factor of stability is determined using "Strength Reduction Method" integrated in FEM plain strain analysis framework. The obtained value of the factor of safety (FS) is 0,952, denoting that landslide is triggered and the exaggerated deformed shape is presented in Figure 7.

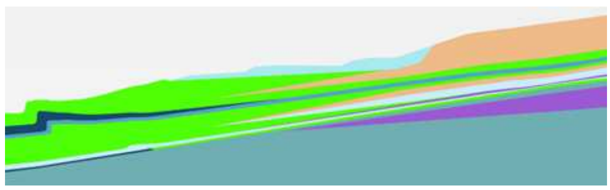

Figure 7. Deformed shape FS=0,952.

Besides that, the mechanical behavior of the landslide triggering can be described also by total displacement diagram as shown in Figure 8 and shearing strains distribution, shown in Figure 9.

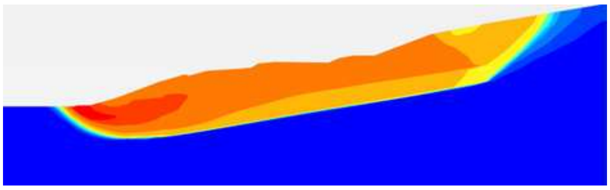

Figure 8. Total displacement diagram.

Figure 9. Shearing strains distribution in sloped terrain.

After the landslide occurrence, triggered by massive quantity of precipitation and uprising of GWT, it was necessary to adopt a consolidation solution to ensure the slope stability for short-term and long-term period. The consolidation system is designed to retain the sliding mass delimited by the sliding surface (strip), as presented in Figure 7. Besides this, at the level of sliding surface and shearing zones, all the soil remains in residual state (residual shearing strength parameters). Thereby, all the components of the consolidation system (SSB1, SSB2 and SSB3) are in interaction with a soil mass affected by shearing phenomena. The piles from the retaining structures (SSB1, SSB2 and SSB3) were modelled by embedded beam elements, and the reinforced concrete elements (rafts and walls) were modeled by solid elements with linear elastic model behavior. There was also adopted a drainage system of ground water and the surface of sloped terrains was rearranged, as seen in Figure 10.

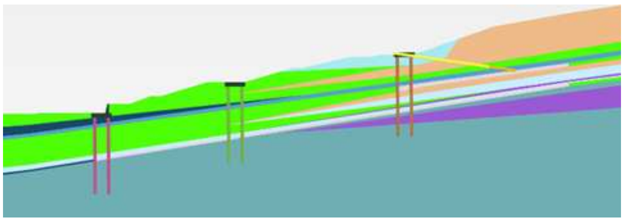

Figure 10. Slope consolidation system

The consolidated section was analyzed in nonlinear elasto-plastic incremental analysis. Thereby there were determined efforts in all structural members (piles and anchors) in order to realize the structural design (section design). It was also performed a slope stability analysis ("Strength Reduction Method") in order to check the stability. By this means, for the consolidated section of the slope, considering residual strength parameters of the soil and consolidation retaining structures, it was obtained FS=1.63. The obtained value of FS indicates that after the installation of consolidation system the slope recover its stability. Beside that, in time, the residual soil will "heal" and recover some quantum of original strength parameters and in that way during time the stability of the sloped terrain will increase. In Figure 11 it is presented an exaggerated deformed shape describing the failure mechanism of the consolidated slope having a safety factor value FS=1,63.

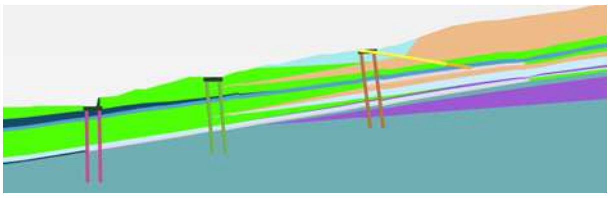

Figure 11. Deformed shape of the consolidated profile FS=1,63.

The consolidation system affects the failure mechanism of the slope in a drastic manner. In that way it is observed from Figure 12 and Figure 9 that shearing (sliding) surface is drastically modified by the consolidation system. Once that retaining systems are installed, it can be observed that there are mobilized multiple sliding surfaces (strips). Thus, the energy mobilized at one sliding strip (initially) is divided after consolidation in multiple sliding strips with lower level of loading. In this manner, all the retaining structural members composed by 3 independent retaining structures "reinforce" the slope and provide the stability.

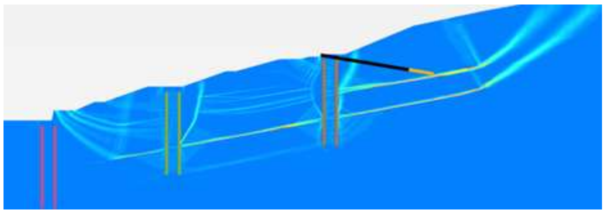

Figure 12. Shearing strains distribution in consolidated section (FS=1,63).

Another major aspect that should be noted is that by this consolidation system, the area from downstream the slope (existing industrial buildings) is fully protected. It can be observed also that after the installation of consolidation structures, there is no shearing zones, strips, or plains in the downstream zone of the SSB1 retaining structure, as seen in Figure 10.

2.3 *Results and discussion*

The sloped terrain develops a major landslide, as seen in Figure 2, with deep sliding plain / strip, as presented in Figure 4. This phenomenon results as a concurring factors like geology, geotechnical stratigraphy, history of the sloped terrain, mechanical behavior of soils, a major climatical event (high discharge of precipitation in a short time period), and a malicious human intervention (excavation at the base of slope).

A triggered landslide with such dimensions (length 130 meters, width 155 meters and 15 meters depth sliding surface) represent a real danger to the constructions built in the downslope of the landslide and for the terrain situated above.

It is necessary to consolidate the sloped terrain and to stop the evolution of the landslide. The loss of internal stress equilibrium of the slope and large deformation affect the soil mechanical parameters which are driven into residual state. In that way the consolidation system must be able to take over the sliding forces generated by huge sliding mass in residual state.

Thereby 3 support structures were positioned on the sloped terrain, as presented in Figure 5, in order to provide enough resisting forces to ensure global stability. Furthermore, it was adopted a groundwater drainage system and the slope surface was rearranged according to support structures positions.

The consolidated sloped analysis confirms that the stability is assured. A safety factor of 1,63 is obtained for the consolidated profile, see Figure 5. All the aspects regarding consolidated slope failure, presented in Figure 6 and Figure 7, indicates that global and local stability of the slope is ensured, and the buildings downslope are safe.

To model the phenomenon, there were performed numerical simulations using Plaxis 2D, in order to capture the exact causes of the landslide triggering mechanism and furthermore to be able to adopt an adequate consolidation system. FEM plain strain analysis framework represent a powerful and dangerous tool in geotechnical engineering field. The power of this method originates from its adaptability, versatility, and higher capacity of replication of soil mechanical behavior described by state-of-the-art constitutive models of soil behavior.

In this case to simulate soil mechanical behavior it was adopted a linear elastic perfect plastic constitutive model (denoted as Mohr-Coulomb model) in addition to Tension cut off criterium (no tension strength for the soil). The plain finite elements used in this case study, are 15 node triangular elements with 12 stress points incorporated per element. 15 node triangular elements are compatible with 5 node lineal elements (embedded beam row) and with 10 node virtual thickness interface elements, for plain contacts.

Obvious advantages of FEM plain strain analysis are that the stress and strain state is determined all over the analysis domain. In that way, by adopting an adequate mesh dimension of the analysis's domain, an adequate set of border conditions and an adequate and well calibrated mechanical constitutive model for the soil and structural members, by incremental iterative elasto-plastic analysis the real mechanical behavior is well replicate in a more or less tolerance range. The validity of this method is well sustained in the literature by benchmark models and experiment replications.

3 CONCLUSIONS

Landslide's phenomena are hazards which deeply affect the social life of community. Once triggered it is very difficult to manage this moving soil mass. Therefore, in case of works near stepped terrain it should be paid very much attention in all stages, from survey to design, construction, and service.

An important tool in geoengineering field is numerical modelling. The key element regarding mechanical behavior of the soil consists in an adequate calibration of the constitutive model based on laboratory and in situ investigations. Hence, there is a chain of trust between soil investigation, numerical modelling, and design.

4 ACKNOWLEDGEMENTS

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