

“Anchored Sheet Pile Wall” as an Alternative Solution for Landslide-Prone Road Slope in Georgia – A Case Study

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Abstract. The construction of high-class highways in mountainous terrain is frequently associated with the occurrence of landslide-prone slopes. To ensure slope stability, terraced and anchored systems are increasingly employed alongside conventional retaining structures. However, such approaches typically require substantial modification of the natural terrain and may result in significant environmental disturbance over extensive areas. Under certain geotechnical conditions, an alternative to gravity retaining walls and terraced-anchored systems is the so-called Anchored Sheet Pile Wall (ASPW). This study presents a design calculation for an ASPW applied to a representative section of the Rikoti Pass highway. The results demonstrate that, for the conditions considered, the ASPW provides the required slope stability while minimizing alterations to the natural terrain compared to terraced-anchored solutions. In addition, its application significantly reduces the volume of retaining structures and earthworks, thereby contributing to improved construction safety. © 2026 Bull. Georg. Natl. Acad. Sci.

Keywords: roadside slope reinforcement, anchored sheet pile wall, finite element method

Introduction

A modern, high-quality highway typically requires greater width, a larger turning radius, and a flatter longitudinal profile. Achieving all of this on mountainous terrain often requires significant modifications to the natural topography, increasing cuts and gradients, and consequently, economic and environmental challenges. This is even more pressing given the greater the specific slope angles and other engineering and geological elements. Consequently, it becomes necessary to rationally design each individual road section, selecting the optimal option from among tunnels, overpasses, expensive geotechnical structures, and various possible options for each. One of the complex and challenging aspects of this comprehensive task is reliably calculating the stability of the optimal road cut and the resulting landslide slope, as well as ensuring its stability.

The development of methods for calculating the stability of landslide slopes has a history spanning more than a century. Due to the large number of difficult-to-predict factors, the difficulty of fully accounting for them, and the accuracy and feasibility of their calculations, numerous analytical and numerical methods and computer programs for slope stability analysis have been developed to date. A critical analysis of these

methods is presented in specialized literature, among which it is described in sufficient detail in GEO-SLOPE International Ltd. 2012, where it is noted:

“Methods for analyzing the stability of geotechnical limit equilibrium have limitations, which arise primarily because these methods do not consider the compatibility of deformations and displacements. This has two serious consequences: first, it is impossible to account for local variations in safety factors, and second, the calculated stress distribution is often unrealistic. The absence of a stress-strain relationship in traditional limit equilibrium analysis methods represents a fundamental missing element of physics.”

Despite these and other inherent limitations, the limit equilibrium methods of modern rock mass stability analysis, which the authors themselves call “Simplified Methods”, still remain a useful tool for practical use, especially for modeling landslide bodies whose sliding surface may have the shape of a circular arc. In general, the improvement of methods for geotechnical calculation of slope stability and making optimal decisions from a technical and economic point of view to meet their operational requirements remains an urgent scientific and technical problem.

Recent methods, which are considered by specialists as a “new era in slope stability analysis”, are a combination of limit equilibrium (LE), shear stress reduction (SSR) and finite element methods (FEM). Among such methods, in some cases, the anchored pile wall may be of particular interest, which, unlike pure piles, does not operate in a cantilevered mode and can often be more effective and competitive than the traditional and widespread method of terracing and anchoring the entire landslide-prone surface. In this regard, the computer program “Anchored Sheet Pile Wall” developed by the Canadian company Rocscience is of particular interest (Rocscience Inc., 2001; 2022).

Innovative technologies of anchored pile walls have been used in recent years, for example in China, on a number of large-scale landslide-prone slopes (Chen et al., 2016). A schematic longitudinal section of one of them shows a typical slope profile that was reinforced with pile walls, 20 m high and 5 m wide, and 3000 kN prestressed anchors. Monitoring after the completion of the works showed that the slopes are stable and safe. It is worth noting that the peculiarity of the scheme was significantly determined by the stability of the slope, as well as ensuring effective drainage of groundwater. This determined both the design and dimensions of the pile walls. In some places, the depth of the piles reached 64 m, and they were not only drilled from above in the traditional way, but also by mining methods, from the bottom up. According to the authors, this new technology provides invaluable experience for strengthening high slopes prone to landslides.

An example of the use of pile walls in Georgia is the reinforcement of an active creep-plastic landslide-prone slope in the vicinity of Sheshelidze Street in Gldani, Tbilisi, which was designed and implemented in 2020 by Caucasus Road Project LLC (CRP) with the participation of the Bavarian Geological Engineering Bureau (BBB) of Germany and the Tsulukidze Mining Institute. The slope was reinforced with 1200 mm diameter reinforced concrete pile caps at depths of 15, 18, 25 and 30 meters and 25 m long anchored piles (Fig. 1).

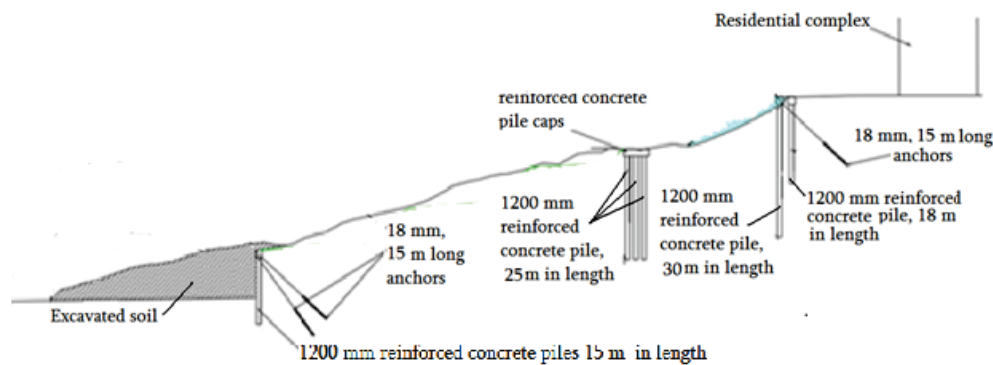


Fig. 1. Schematic design for strengthening the Gldani creep-plastic landslide-prone slope with restrained and anchored piles.

A visual front view of the lower wall with cuts is given in Fig. 2. Here too, monitoring conducted over the five years after the completion of the works showed that the slope is stable, its previous movement is no longer observed, and the condition of the multi-story residential buildings on top of the slope is safe.

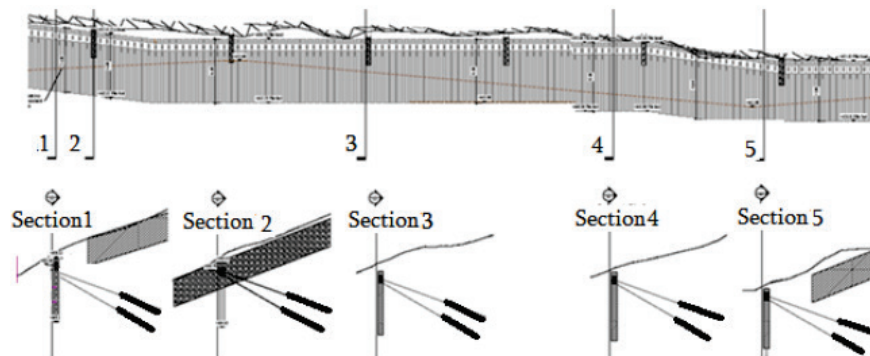


Fig. 2. Imaginary view of the lower anchored pile wall of the Gldani landslide-prone slope with cuts.

The use of this method in practice is not very common yet, although the above examples show that the method of reinforcement with an Anchored Sheet Pile Wall can be effective, especially in a number of cases of reinforcement of landslide-prone slopes along highways, and in urban conditions, in the walls of construction pits and other outcrops.

For example, Fig. 3 shows some of the numerous roadside slopes of the Rikoti pass. Figure 4a shows a cross-section in its original state, along with the lithological boreholes and component rock characteristics for one of these slopes, located at the 149-km mark. The estimated minimum safety factor for this slope, calculated (BT Consult, 2023) using computer program Slide 6 and the Spencer method (Fig. 4b), was $SF=1.616$.



Fig. 3. Some examples of cut slopes caused by road widening at the Rikoti crossing.

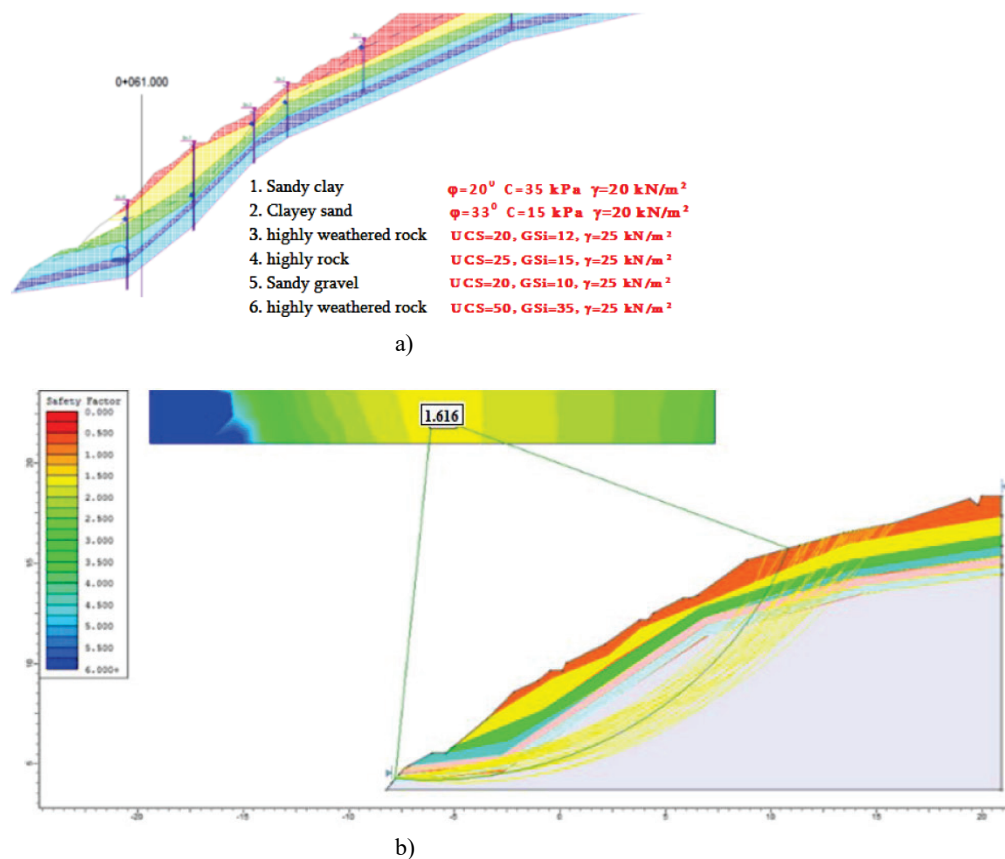


Fig. 4. a) 1-1 section of the existing relief with lithological wells and rock characteristics; b) Minimum Safety Factor before slope change.

The cross-section diagram of this part of the slope in the initial state, with the lithological wells marked on it and the characteristics of the constituent rocks, is given in Fig. 4.a. The estimated minimum safety factor of this slope, calculated using Rocscience Slide 6 and the Spencer method (Fig. 4.b), should have been SF=1.616.

The traditional terracing-anchoring rule was adopted as the optimal geotechnical model. The new roadbed required cutting a 30-meter-high ground at its edge. To restore the stability of the cut slope, the design based on the terracing-anchoring rule provided for lowering the existing surface over a large distance to redistribute the height of the cut (Fig. 5).

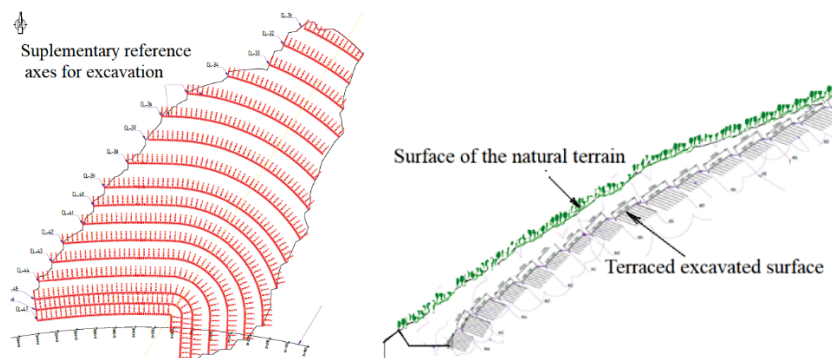


Fig. 5. Terracing – Anchoring plan-section on Rikoti Road (BT Consult, 2023).

The works included clearing 6.5 hectares of scrub, excavation of 1.08 million cubic meters of surface rock, installation of over 5,000 square meters of steel wire mesh, revegetation, and construction of a surface water drainage network. In addition, 11,500 anchors (32 mm in diameter) were installed, with a total length of 163,700 meters (1,033 tons), along with other ancillary works.

In case of need to cut the base of such a natural slope, an alternative and competitive variant of the terrace-anchor scheme can be considered a wall of anchor piles, the scheme of which for three-stage implementation, say, a two-stage variant, would have the form shown in Fig. 6.

At the first stage, a certain number of upper piles and anchors provided for by the working design are arranged on top of the orientation pit. At the second stage, the upper part of the pit is cut, the lower piles and anchors are installed. At the third stage, the ground is cut to the bottom of the pit, i.e. to the design level of the road (Fig. 6).

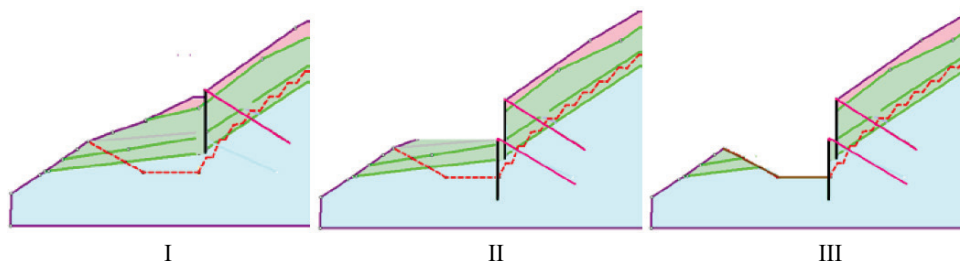


Fig. 6. Three-stage implementation scheme of the two-stage version of the anchored sheet pile wall.

For a two-stage wall constructed in this manner, where 20-meter piles form 16- and 17-meter-high walls reinforced with 30-meter anchors, a two-dimensional finite element analysis model (Fig. 7a) was developed using the Rocscience Phase 2 software to evaluate the stress-strain state of the slope rock mass and retaining elements. A polychromatic picture of stress distribution in the rock mass and displacements during the third stage, as well as the internal forces acting in the piles and anchors, are presented in Fig. 7b. The analysis allows the following conclusions to be drawn.

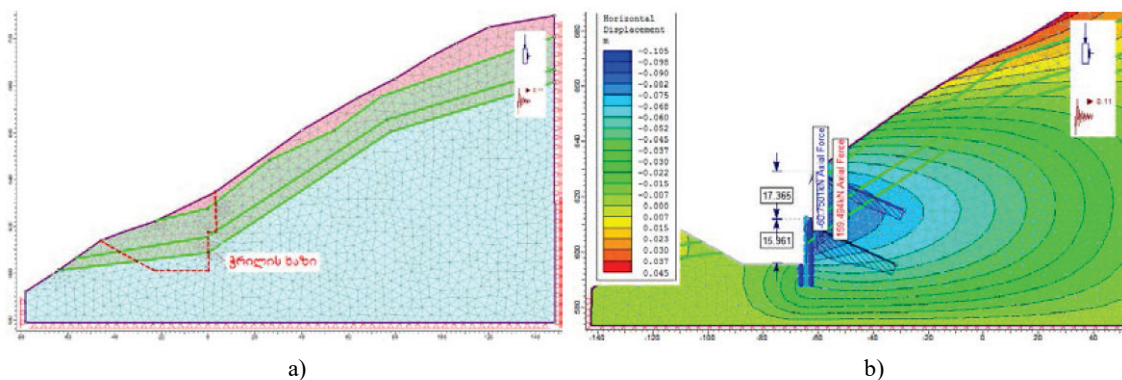


Fig. 7. a) Two-dimensional analysis model of the Rocscience Phase 2 finite element method and b) image of rock mass displacements.

For the conditions of the calculation data of the slope shown in Fig. 4.a, the mass displacements of the embankment wall points caused by the roadside excavation are given in Fig. 8, from which it can be seen that the maximum displacement is 5 cm and occurs at 18-21 m from the base of the wall. For a two-stage wall constructed in this manner, where 20-meter piles form 16- and 17-meter-high walls reinforced with

30-meter anchors, a two-dimensional finite element analysis model (Fig. 7.a) was developed using the Rocscience Phase 2 software to evaluate the stress–strain state of the slope rock mass and retaining elements. A polychromatic picture of stress distribution in the rock mass and displacements during the third stage, as well as the internal forces acting in the piles and anchors, are presented in Fig.7.b. The analysis allows the following conclusions to be drawn.

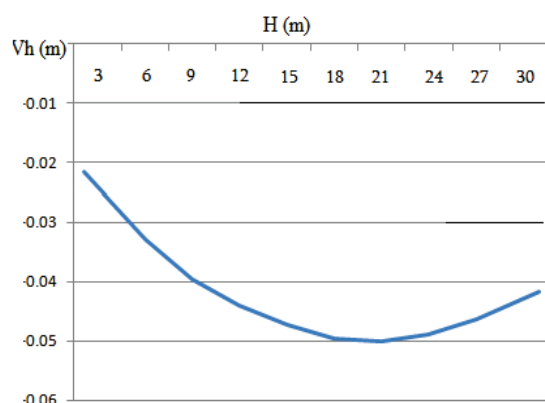


Fig. 8. Horizontal displacements of wall points.

As for the tension forces in the anchors, according to Fig. 9, the axial forces $P = 64$ kN and $P = 168.5$ kN act, respectively, in the upper and lower anchors per meter of wall width. Using this program, the required cross-sections of the anchors can be selected by verification calculation. If for preliminary calculation, steel passive end-mounted anchors with a design resistance $R_a = 500$ MPa and a diameter $d = 32$ mm are installed at a spacing of 1 m on top of the upper and lower walls, the variant of their design dimensions and resistance and strength reserves for gravitational and seismic loads are given in Table 1.

Table 1. Calculation options for anchors and safety factors

Anchors row	d (cm)	Des. Resist. (kPa)	Section area (m ²)	Tensile force (kN)	Stress (kPa)	Safety Factor
Upper	3.2	5.00E+05	8.04E-04	64	79580	6.28
Lower	3.2	5.00E+05	8.04E-04	168	208897	2.39

To reduce the excess reserve to the normative value, it is possible to reduce the diameters of the anchors or increase the distances between them and repeat the verification calculation. The maximum bending moments per unit wall width (Fig. 9) for the upper piles are 248.3 kNm, while in the lower piles, the support is equal to 411.4 kNm, and in the beam -292.3 kNm.

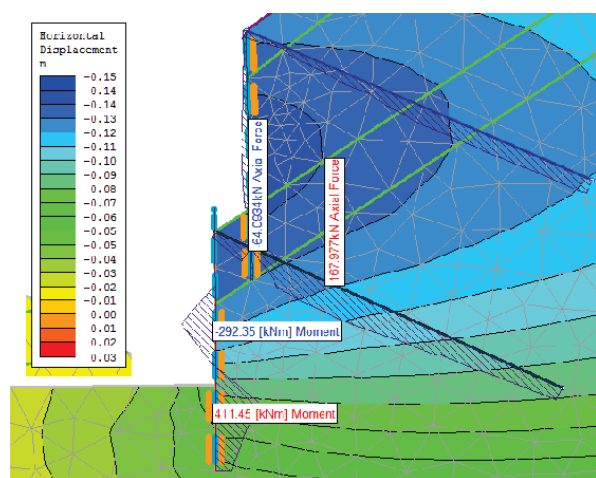


Fig. 9. Diagrams and maximum values of anchor tensile forces and piles bending moments.

The design dimensions and resistance of anchors to gravity and seismic loads, as well as their safety factors, are presented in Table 1, while the maximum internal forces and displacements acting in the beams are presented in Table 2.

The structural characteristics of the beams are determined by a so-called verification calculation. The design dimensions are taken from practical experience, their bearing capacity for eccentric compression caused by the above-mentioned forces is verified, and the necessary and sufficient dimensions are determined using an iterative method.

For example, suppose a reinforced concrete beam wall (Fig. 10) with an initial thickness of $h = 0.5$ m is reinforced on both sides with steel bars with a diameter of $d_a = 32$ mm and a design resistance of $R_a = 500$ MPa. Four bars are inserted on each side of a 1 m long wall section with a cross-sectional area of $F_a = F_a' = 4 * 3.1415 * 1.62 = 32.2$ cm². Concrete with a design strength of $R_b = 30$ MPa and a maximum deformation of $\varepsilon = 0.003$. The calculation results (Fig. 9) are presented in Table 2.

Table 2. Maximum internal forces and displacements acting in beams

Row of piles	Max. force (kN)	Max. bending moment (kNm)	Max. displacement (m)
Upper	64.1	248.3	
Lower	166.9	411.45 (-292.35)	0.05

The eccentricity of the action of such internal forces on the upper and lower piles wall will be:

$$E_{\text{upper}} = M/N = 248.3/64.1 = 3.87 \text{ m};$$

$$E_{\text{lower}} = 411.4 (-292.3)/166.9 = 2.46 (-1.75) \text{ m},$$

That is, the compression of the piles in both rows occurs with greater eccentricity (Fig. 10).

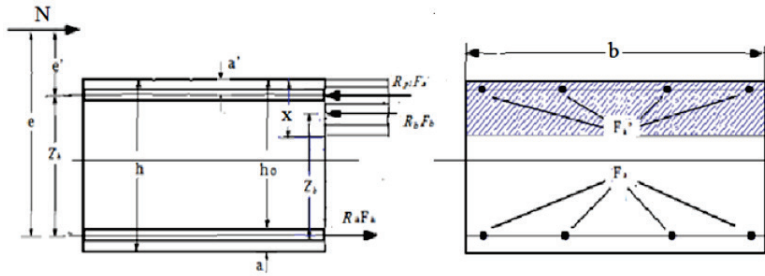


Fig. 10. Calculation diagram for a piles wall with large eccentricity.

In such a case, the calculation scheme for the limit state of a double-reinforced concrete wall will have the form shown in Fig. 10, and the thickness and area of the compressed part of the concrete indicated on it will be per unit length of the wall:

$$x = 0.8 * h / 2 = 0.2 \text{ m}; F_b = 0.2 \text{ m}^2.$$

The strength of a wall with the dimensions and reinforcement taken can be checked for eccentric compression under the following conditions: resistance of the reinforced concrete section of the wall to compression

$$N_{\text{res.}} = R_b F_b + R_{pl} F_a' - R_b F_a = 30000 * (0.2 - 0.00322) + 500000 * 0.00322 = 1574 \text{ kN}. \quad (1)$$

This exceeds the effective compressive forces:

In the upper piles - $1574/64.1 = 24.5$ times, and in the lower piles - $1574/167 = 10.5$ times.

The limit of resistance of the cross-section of the reinforcement rods during wall bending:

$$M_{\text{lim}} = R_a F_a Z = 500000 * 0.00322 * (0.5 - 0.06) = 707.7 \text{ kNm}. \quad (2)$$

This exceeds the maximum bending moments given in Table 2: in the upper slope $707.7 / 248.3 = 2.85$ times, and in the lower slope - $707.7 / 411.45 = 1.72$ times.

These reserves may be acceptable from a structural point of view for walls of the given dimensions, despite the fact that the wall has a very large reserve for eccentric compression. To reduce the excessive reserve to the normative, it is also possible to reduce the wall dimensions and reinforcement and repeat the verification calculation.

After such a preliminary calculation, it is necessary to assess the degree of overall stability of the slope. This first of all requires finding the critical sliding surface, which is a significant problem for anchored wall schemes, despite the existence of a large group of modern special programs for assessing the stability of landslide-prone slopes (Slide 1-6, SLOPE/W, GEO-5, etc.). In such cases, to solve specific problems, it is necessary to test and specify the positions of the test sliding surfaces (BT Consult, 2023). For this, we use the calculation scheme shown in Fig. 7 of the Rocscience Phase 2 program, since the Slide 6 program is not provided for similar schemes.

Stability is assessed, for example, at 14 points of 60 degree inclined and circular arc planes, on which the XX, YY and XY stresses are determined by the Rocscience Phase 2 program. These stresses are shown in Fig. 11 and are included in Excel Tables 3 and 4.

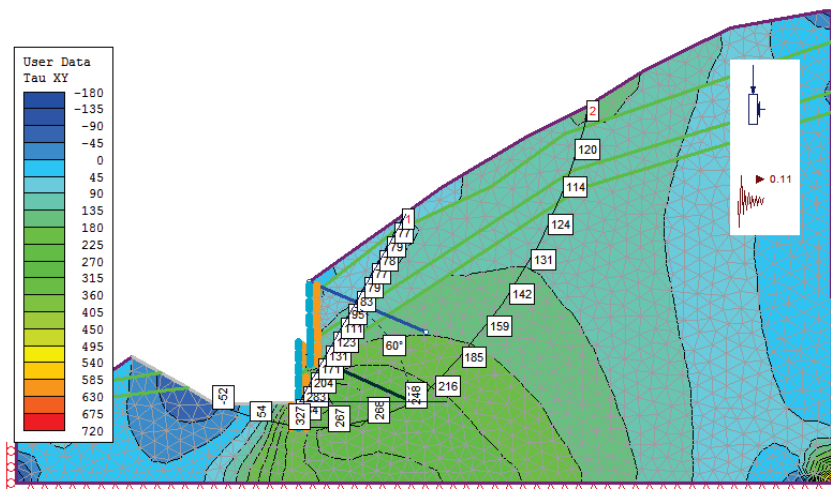


Fig. 11. Determination of stresses on 45 and 60 degree inclined planes of rock mass.

Table 3. Calculation of the Safety Factor on the 60° inclined plane of a reinforced slope

#	XX	YY	XY	θ^0	θ rad	N kPa	T kPa	C kPa	φ^0	φ rad	[T] kPa	SF
1	485	1163	237	60	1.05	1368	-412	15	35	0.61	972.96	2.36
2	176	993	302	60	1.05	1254	-505	15	35	0.61	893.13	1.77
3	110	895	218	60	1.05	1084	-449	15	35	0.61	773.74	1.72
-	-	-	-	-	-	-	-	-	-	-	-	-
13	101	203	75	60	1.05	268	-82	15	35	0.61	202.80	2.49
14	105	88	75	60	1.05	153	-30	15	35	0.61	122.02	4.05
						$\Sigma T =$	-3501			$\Sigma [T] =$	6798	
											SF =	1.96

Table 4. Calculation of the Safety Factor on the circular arc plane of the slope

N	XX	YY	XY	θ^0	θ rad	N kPa	T kPa	C kPa	φ^0	φ rad	[T] kPa	SF
1	485	1163	-52	-19	-0.33	321	168	15	35	0.61	240	-1.43
2	176	993	54	0	0.00	0	54	15	35	0.61	15	-0.28
3	110	895	327	14	0.24	0	104	15	35	0.61	15	-0.14
-	-	-	-	-	-	-	-	-	-	-	-	=
13	101	203	114	70	1.22	304	-120	15	35	0.61	228	1.90
14	105	88	120	74	1.29	145	-98	15	35	0.61	117	1.20
						$\Sigma T =$	-1798			$\Sigma [T] =$	3955	
											SF =	2.20

Using their values and formulas (3) and (4) of stress theory, the holding, normal N , and moving, tangential T stresses acting at the points of the mentioned planes are calculated.

$$N = 0.5(XX + YY) + 0.5(XX - YY)\cos 2\theta + XY \sin 2\theta, \quad (3)$$

$$T = 0.5(YY - XX) \sin 2\theta + XY \cos 2\theta. \quad (4)$$

Using formulas (3) and (4) and the Coulomb-Mohr boundary equilibrium condition (5), we evaluate the stability at the same points under normal and extreme conditions.

$$T \leq [T] = C + (N - u) \tan \varphi \quad (5)$$

In this formula, $[T]$ are the ultimate holding contact stresses, calculated using Excel spreadsheets, C and φ are the coefficients of rock cohesion and internal friction, u is the so-called pore pressure coefficient.

The Safety factor thus determined for the alternative method of anchored pile wall of the considered slope on the planes inclined by 60 degrees and on the circular arc is equal to 1.96 and 2.2, respectively, which exceeds the 1.25 provided for by the Eurocode and the values of the global and internal Safety Factor specified in the project implemented by the terracing-anchoring method. In addition, the alternative option of slope excavation requires minimal damage to the natural slope, a significant reduction in the volume of structures and earthworks, and increased safety.

Conclusion

To ensure the stability of landslide-prone slopes during the construction of high-class highways in mountainous terrain, the so-called *Anchored Sheet Pile Wall (ASPW)* may be used as an alternative to conventional retaining structures and terraced-anchored systems. In many cases, its application results in:

- minimal disturbance to the existing terrain and natural conditions;
- a significant reduction in the volume of retaining structures and earthworks;
- improved construction safety.

The proposed method for the design calculation of the *Anchored Sheet Pile Wall (ASPW)* is based on the Phase2 program developed by Rocscience. This approach enables the evaluation of the stress-strain state at each point within a staged rock mass, the identification of local variations in safety factors, and the determination of the load-bearing capacity of the retaining structure. Thus, the method is free from the critical remarks (GEO-SLOPE International Ltd., 2012), expressed above regarding modern, limit equilibrium analysis methods.

გამოყენებითი მექანიკა

დაანკერებული ხიმინჯოვანი კედელი, როგორც გზისპირა მეწყერსაშიში ფერდობის ალტერნატიული გამაგრება საქართველოში (შემთხვევის შესწავლა)

ლევან ჯაფარიძე

აკადემიის წევრი, გ.წულუკიძის სახ. სამთო ინსტიტუტი, თბილისი, საქართველო

მთაგორიანი ტერიტორიების ფერდობებზე მაღალი კლასის გზატკეცილის მშენებლობისას ხშირად წარმოიშობა მეწყერსაშიში ფერდობები. მათი მდგრადობის უზრუნველსაყოფად ტრადიციულ საყრდენ კედლებთან ერთად უფრო მეტად გამოიყენება ტერასულ-ანკერირებული სქემები. ეს კი ხშირად მოითხოვს არსებული რელიეფის მნიშვნელოვან შეცვლას და ბუნებრივი პირობების დაზიანებას საკმაოდ დიდ ფართობებზე. გრავიტაციული საყრდენი კედლებისა და ტერასულ-ანკერირებული ფერდობების ალტერნატიული ვარიანტი განსაზღვრულ პირობებში შეიძლება იყოს ე.წ. დაანკერებული ხიმინჯოვანი კედელი, რომლის გაანგარიშების მაგალითი რიკოტის უღელტეხილის გზის კონკრეტული უბნისათვის მოცემულია წარმოდგენილ სამუშაოში. ნაჩვენებია, რომ განხილული უბნის მსგავს პირობებში ფერდობის მეწყერსაშიშროების მიმართ უსაფრთხოების უზრუნველსაყოფად ტერასულ-ანკერირებულ მეთოდთან შედარებით დაანკერებული ხიმინჯოვანი კედელი მოითხოვს ბუნებრივი რელიეფის გაცილებით ნაკლებ დაზიანებას, განაპირობებს სამაგრი კონსტრუქციებისა და მიწის სამუშაოთა მოცულობების მნიშვნელოვან შემცირებასა და შრომის უსაფრთხოების გაზრდას.

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