Comparative assessment of slope stability in weathered schists using Q-slope and LEM

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Abstract. The slope stability in weathered schists formations has always been of great importance for studies in rock mechanics. The majority of the methods used in the geotechnical practise for estimating the slope stability are based on the traditional limit equilibrium methods, analysing the possibilities of circular and non-circular failures. On the other side, very few empirical methods exist for assessment of the slope stability. Q-slope is a relatively new methodology for the assessment of slope stability in terrains built from rock masses. This method was developed over the last decade by Barton and Bar (2015, 2017), with modification of the original Q-system and it is based on the results of more than 500 case studies all over the world in reinforcement-free slopes. This paper presents results of the slope stability assessment with the limit equilibrium and Q-slope method at four characteristic cut sections with unusual general heights (ranging from 30 m to 92 m), along the route of the A2 motorway, section Kicevo-Ohrid in the Republic of N. Macedonia, which is under construction. The subject cuts presented in this paper were excavated in the period 2014 – 2018 without the application of special measures for protection. The research area is built mainly from phyllite, sericite schists, quartz-sericite schists and graphitic schists, which are folded, tectonically disturbed and affected by the process of degradation. According to the results of the analyses, it can be emphasised that both limit equilibrium method and Q-slope are applicable at terrains built from weathered and tectonically disturbed schists because the stability conditions on site were confirmed with these two methodologically different approaches. As a general conclusion, we can summarize that the best way to assess the slope stability condition is when we combine different tools and methodologies, especially when we face highly variable rock mass conditions.

1. Introduction
Slope stability in deep cuts built from highly weathered anisotropic rocks is a very popular theme for discussion by geological and geotechnical engineers all over the world. The main reason for this is the vulnerability and susceptibility of the deep cut sections in terms of developing potentially unstable zones over time, due to foliation and joint sets with unfavourable orientation.

Assessment of the slope stability in such rock mass media is a very complex task, where failures although can occur globally affecting few berms or the entire cut, most often have local character (plane failure, wedging, local toppling). On the other side, very often the slope angles are constant from the top to the toe of the cut, where not only the lithological structures can differ, but also the degree of alteration, tectonic disturbance etc.
The Limit Equilibrium Method (LEM) is used extensively for slope stability analysis, primarily because of its simplicity. In this study, depending on the kinematic conditions, Slide, Swedge and RocPlane from the software package Rocscience, were used for assessment of the slope stability.

Very few empirical methods exist for the assessment of rock slope stability. Slope mass rating (SMR) developed by Romana, 1985, 1995 and global slope performance index developed by Sullivan, 2013 can be used to evaluate the competence and performance of a particular excavated rock slope and to predict measures for support and reinforcement. These methods are based on several factors influencing the strength and deformability of the rock masses (joint orientation and density, intact strength, geological structure, groundwater). Although these two empirical rock-engineering methods propose stabilization and supportive measures if necessary, they do not indicate the long-term stable slope angles without reinforcement. Moreover, these slopes are most common.

The Q-slope method is a relatively new methodology for the assessment of slope stability in terrains built from rock masses. This method was developed over the last decade by Barton and Bar, 2015, 2017 with modification of the original Q-system. The main advantage of this methodology is that it can help engineers to estimate the steepest long-term stable slopes, which are reinforcement-free.

The results with Q-slope are compared with the results obtained with LEM to have data for appropriate comparison and engineering judgement.

2. A brief overview of the methods used for slope stability assessment

2.1. Limit equilibrium method
Since 1930, slope stability analyses were usually done with LEM. Although several other methods have been developed in the past few decades, the limit equilibrium approach remains to be the most used method in geotechnical practice, primarily because of its simplicity. LEM is based on a single value, which arises from the balance of the resisting and driving forces and determines either the factor of safety or the probability of failure. The drawback of this approach is that the factor of safety is supposed to be constant along the assumed slip surface. This methodological approach is well known, so this paper won’t go through the details of it.

2.2. Q-slope method
The purpose of the Q-slope method is to allow engineering geologists and rock engineers to assess the stability of excavated rock slopes in the field and make potential adjustments to slope angles as rock mass conditions become visible during construction (Barton and Bar, 2015). The Q-slope method is intended for use in reinforcement-free rock slopes and it is based on the results from more than 500 case studies all over the world, which is a reliable database for engineering judgments of slope stability in other projects and further development.

Q-slope is estimated from an empirical equation, which is very similar to the original Q-system and utilizes the same six parameters as shown in equation (1). The first four parameters (RQD, Jn, Jr, and Ja) in the equation remain unchanged from the original Q-system (Barton et al., 1974, Barton and Bar, 2015). Only the important changes (adding’s) of the original Q-system are shortly explained in addition.

\[
Q_{\text{slope}} = \frac{RQD \cdot (J_r)}{J_n \cdot (J_a) \cdot O \cdot SRF_{\text{slope}}} \]

Where,
- \(RQD\)  Rock quality designation index
- \(J_n\) Joint set number
- \(J_r\) Joint roughness number
- \(J_a\) Joint alteration number
- \(O\) Discontinuity orientation factor
- \(J_{vice}\) Environmental and geological condition number
- \(SRF_{\text{slope}}\) Strength reduction factor
The ratings for the discontinuity orientation factor (O-factor) are shown in table 1. Set A refers to the most unfavourable discontinuity set, while Set B refers to the secondary discontinuity set. Set B is applied only in case of forming potentially unstable wedges.

Table 1. Discontinuity orientation factor (O-factor).

| Description                          | Set A | Set B |
|--------------------------------------|-------|-------|
| Very favourably oriented             | 2.0   | 1.5   |
| Quite favourable                     | 1.0   | 1.0   |
| Unfavourable                         | 0.75  | 0.9   |
| Very unfavourable                    | 0.50  | 0.8   |
| Causing failure if unsupported       | 0.25  | 0.5   |

The environmental and geological condition number $J_{wice}$ presented in table 2, in relation to $J_w$ of the original Q-system, takes into account the climatic and environmental influence (intense erosive rainfall and ice-wedging), where the influence of drainage and slope supportive measures are also included. It should be noted that when drainage measures are installed, $J_{wice}$ should be multiplied with 1.5, when slope reinforcement measures are installed $J_{wice}$ should be multiplied with 1.3 and when both measures are installed $J_{wice}$ should be multiplied with 1.95 (1.5×1.3).

Table 2. Environmental and geological condition number.

| $J_{wice}$                          | Desert environment | Wet environment | Tropical storms | Ice wedging |
|-------------------------------------|--------------------|-----------------|-----------------|-------------|
| Stable structure; competent rock    | 1.0                | 0.7             | 0.5             | 0.9         |
| Stable structure; incompetent rock  | 0.7                | 0.6             | 0.3             | 0.5         |
| Unstable structure; competent rock  | 0.8                | 0.5             | 0.1             | 0.3         |
| Unstable structure; incompetent rock| 0.5                | 0.3             | 0.05            | 0.2         |

The strength reduction factor $SRF_{slope}$ is evaluated by using the least favourable case i.e. maximum of $SRF_a$, $SRF_b$, and $SRF_c$. Table 3 describes $SRF_a$ for the physical condition of the slope surface (present or expected) due to susceptibility to weathering and erosion, table 4 describes $SRF_b$ for adverse stress-strength ranges in the slope, where $\sigma_c$ is unconfined compressive strength (UCS) and $\sigma_1$ is maximum principal stress and table 5 describes $SRF_c$ for major discontinuities such as faults, weaknesses zones and joint swarms which may also contain clay filling that adversely affects slope stability (lowers the shear strength), where RQD$_{100}$ refers to 1 m perpendicular sample of discontinuity and RQD$_{300}$ refers to 3 m perpendicular sample of discontinuity.

Table 3. $SRF_a$ – physical condition of slope surface.

| Description                                                                 | Set B |
|-----------------------------------------------------------------------------|-------|
| A  Slight loosening due to surface location, disturbance from blasting or excavation | 2.5   |
| B  Loose blocks, signs of tension cracks and joint shearing, susceptibility to weathering, severe disturbance from blasting | 5     |
| C  Same as “B”, but strong susceptibility to weathering                      | 10    |
| D  The slope is in an advanced stage of erosion and loosening due to periodic erosion by water and/or ice-wedging effects | 15    |
| E  Residual slope with significant transport of material downslope            | 20    |
Table 4. SRFb – stress and strength range in slope.

| Description                                      | $\sigma_c/\sigma_{1a}$ | SRFb |
|--------------------------------------------------|-------------------------|------|
| F Moderate stress-strength range                 | 50 – 200                | 2.5  |
| G High stress-strength range                     | 10 – 50                 | 5    |
| H Localized intact rock failure                  | 5 – 10                  | 10   |
| J Crushing or plastic yield                      | 2.5 – 5                 | 15   |
| K Plastic flow of strain softened material       | 1 – 2.5                 | 20   |

Table 5. SRFc – major discontinuities.

| Description                                      | Favourable | Unfavourable | Very unfavourable | Causing failure if unsupported |
|--------------------------------------------------|------------|--------------|-------------------|-------------------------------|
| L Major discontinuity with little or no clay     | 1          | 2            | 4                 | 8                            |
| M Major discontinuity with $RQD_{100} = 0^a$ due to clay and crushed rock | 2          | 4            | 8                 | 16                           |
| N Major discontinuity with $RQD_{300} = 0^b$ due to clay and crushed rock | 4          | 8            | 12                | 24                           |

After evaluation of the $Q_{slope}$ value, we can estimate the steepest long-term stable slopes, which are reinforcement-free. Namely, Barton and Bar (2015) derived a simple formula for the steepest slope angle ($\beta$) not requiring reinforcement or support:

$$\beta = 20 \log_{10} Q_{slope} + 65^\circ \quad (2)$$

It should be noted, that although equation (2) does not estimate a specific factor of safety, it represents a long-term stable slope angle without reinforcement measures. The equation represents the average data for stable slope angles greater than 35° and less than 85°.

3. Geological and geotechnical characteristics of the research area

The research area covers the A2 motorway from Kicevo to Ohrid, which is under construction. This is a capital facility for the Republic of N. Macedonia, where many deep cut sections are designed with significant heights varying from 30 m to more than 90 m. The location is built mainly from phyllite, sericite, quartz-sericite and graphitic schists and a small part from chlorite-epidote and carbonate-sericite schists. The subject anisotropic rocks are folded, tectonically disturbed and affected by the process of degradation. The rocks often transform from one type to another both in a horizontal and vertical direction. Foliation dip angles are varying in the range from 10° – 50° dipping to the hill, which is favourable in terms of stability.

The subject rocks are very prone to local deformations, especially in aspects of the long-term impact of atmospheric influences and seismic influence, as well as the existence of joint sets with unfavourable orientation.

In various stages of research, within the complete construction activities, a significant database has been synthesized and systematized, consisted of more than 1370 rocks samples incorporated in 162 results from laboratory tests and more than 100 field investigations. Graphic interpretation of the main strength parameter of intact rock samples, depending on the type of rock is shown in figure 1. The average values of the uniaxial compressive strength (UCS) are based on a statistical variation of the test results. According to the results (figure 1), it is clear that the strength varies up to 5 times depending on the direction.

Besides the strength of intact parts, discontinuity conditions have a great influence on the mechanical behaviour, especially on the type of failure. RQD parameter varies in the wide range from 25 – 75%.
Figure 1. Graphic interpretation of the value of the uniaxial compressive strength (UCS), expressed through point load test (PLT), for measurements performed perpendicular and parallel to foliation.

The comparative assessment of the slope stability was conducted on 4 characteristic cut slopes. The cuts are selected having in mind that some local (wedge) failures occurred on two cuts while the remaining cuts are stable, as shown in figure 2. The failures occurred few weeks after the excavation.

Figure 2. View of the analysed cuts: a) 11+740 – 12+140; b) 12+340 – 12+600; c) 15+690 – 16+114; d) 17+900 – 18+200.
The first two cuts at chainage km 11+740 – 12+140 and km 12+340 – 12+600 were excavated with 10 m high and 4.0 m wide berms and inclination 5:1 (78.7°). The general heights of the cuts are 47 m and 30 m respectively. The cut at chainage km 15+690 – 16+114 is excavated with 6 m high and 4 m wide berms and inclination of 1.5:1 (56.3°) on the lowest four berms and 1:1 (45°) on the remaining berms. The general height of the cut is 92 m. The last cut at chainage km 17+900 – 18+200 is excavated with 6 m high and 3.5 m wide berms and inclination of 1.5:1 (56.3°), with a general height of 60 m.

4. Analyses and results

In the design phase, LEM was used as the main design method, combined with the finite element method in critical locations. For an illustration, output results from global stability analyses in Slide, for one of the cuts is shown in figure 3. It should be noted, that global stability, as is the case with most of the rock slopes, is not authoritative. Therefore, depending on the kinematic conditions, the local stability along discontinuities was also analysed for the subject cut slopes.

The Q-slope evaluation of the slopes, depending on the cut section, is shown in table 6. Besides the ratings for all the necessary input parameters, the last two rows of the table present the Q-slope value and the steepest slope angle not requiring reinforcement.

| Parameter | a) 11+740 – 12+140 | b) 12+340 – 12+600 | c) 15+690 – 16+114 | d) 15+690 – 16+114 | e) 17+900 – 18+200 |
|-----------|---------------------|---------------------|---------------------|---------------------|---------------------|
| RQD [%]   | 60 – 65             | 45 – 50             | 35 – 40             | 55 – 60             | 40 – 45             |
| $J_a$     | 9                   | 6                   | 9                   | 9                   | 6                   |
| $J_t$     | 1.5 (A), 2.0 (B)    | 1.5 (A), 2.0 (B)    | 2.0                 | 2.0                 | 2.0                 |
| $J_s$     | 2.0 (A), 2.0 (B)    | 3.0 (A), 2.0 (B)    | 4.0                 | 3.0                 | 3.0                 |
| O-factor  | 0.75 (A), 0.9 (B)   | 0.75 (A), 0.9 (B)   | 0.5                 | 0.75                | 0.75                |
| $J_{wcie}$| 0.7                 | 0.7                 | 0.7                 | 0.7                 | 0.7                 |
| SRFslope  | 5 (SRFa)            | 5 (SRFa, SRFb)      | 5 (SRFa)            | 5 (SRFa)            | 5 (SRFa)            |
| Q-slope   | 0.492               | 0.335               | 0.146               | 0.447               | 0.496               |
| $\beta$ [°] | 58.8            | 55.5                | 48.3                | 58.0                | 58.9                |
According to the results presented in table 6, it is clear that for the first two cuts where failures occurred, the Q-slope method suggests smaller slope angles. Namely, the slopes of the first cut should be excavated with an inclination less than 58.8° and the second cut should be excavated with an inclination less than 55.5°. For the remaining two cuts, the steepest long-term stable slopes which are reinforcement-free, are slightly steeper than excavated slope angles (≤3.3°), which means that the slope angles for these cuts are appropriate to the terrain conditions.

A sublimate of all results obtained with both LEM and Q-slope analyses is shown in figure 4.

![Figure 4. Q-slope stability chart (Barton and Bar, 2015) for the subject cut slopes and their corresponding factors of safety obtained with LEM in Slide and Swedge.](image)

The coloured dots in figure 4 represent the Q-slope value. The factors of safety in the software package Slide refer to the global stability and the factors of safety obtained with the software package Swedge refer to the stability of surface wedges, depending on the cut section. According to the results, it is clear that the cuts with failures (green and blue dot, figure 4) are quite deep in the unstable zone of the Q-slope stability chart. In addition, for these two cuts, the values of the factor of safety against forming surface wedges are smaller than 1.0, which confirms the failures. The global factor of safety is not indicating a failure (which in rock slopes is not common) but it is not satisfactory also. We can conclude that these two cuts must be stabilized, either with flattening the slopes i.e. additional excavation or with reinforcement measures.

The remaining cuts (black, red and purple dot, figure 4) are in the stable zone of the Q-stability chart, but very close to the area which indicates uncertain stability. This means that the slope angles are appropriate to the terrain conditions i.e. the long-term stability is satisfied and at the same time, the cut slopes are not overdesigned. The values of the factors of safety also indicate stable slopes, although their values are debatable.
5. Concluding remarks
In a frame of the article, results for different possible failure mechanisms in complex geological media were presented, using the traditional limit equilibrium method and the relatively new empirical approach Q-slope.

Q-slope is a relatively fast and intuitive empirical method for the stability assessment of excavated reinforced-free slopes. The main advantage of Q-slope over the other empirical methods is that it helps estimate the long-term stable slope angles without reinforcement.

It should be noted, that for the subject cut slopes assessed with the Q-slope method and presented in this paper, the strength reduction factor SRF\textsubscript{b} that accents the stresses and the strength range in the slope was authoritative for estimation of the Q-slope. This is because SRF\textsubscript{b} is more crucial for highly weathered rocks that have low strength and it increases as slope height increases because the stresses in the slope are also increasing.

According to the results of the analyses, it can be emphasised that both LEM and Q-slope are applicable at terrains built from weathered and tectonically disturbed schists because the stability conditions on site were confirmed with these two methodologically different approaches.

The final judgement regarding the stability of rock slopes should be made after we can inspect the excavated surfaces because it is very difficult to register all the discontinuities and their condition before the construction phase starts.

As a general conclusion, we can summarize that the best way to assess the stability condition of cut slopes is when we combine different tools and methodologies, especially when we face highly variable rock mass conditions.

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