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Proposing a Viable Stabilization Method for Slope in a Weak Rock Mass Environment using Numerical Modelling: a Case Study from Cut Slopes

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Abstract

The design of a stable slope in a rock mass environment is a quite complicated job due to the anisotropic behaviour of the rock mass. In this research work, the cut slopes at the Swat motorway in the weakest schist rock is numerically analyzed by the shear strength reduction (SSR) approach using the Finite Element-based 2D RS2 software. The slope is divided into two cases according to the nature of the rock. Each case of the cut slope is analyzed by two stabilization methods: 1) changing the characteristics of the slope 2) support system installation based on the Mohr-Coulomb (MCC) and Generalized Hoek and Brown (GHB) failure criteria in order to propose the most appropriate method for slope stabilization. The results obtained reveal that the Critical Strength Reduction Factor (CSRF) before applying the stabilization methods is 1.34 (MCC) and 1.04 (GHB) for Case-I and 1.21 (MCC) and 0.53 (GHB) for Case-II. CSRF for Case-I after changing the characteristics of the slope is observed to be 2.43 (MCC) and 2.33 (GHB), while for Case-II is 1.82 (MCC) and 1.26 (GHB), respectively. CSRF for Case-I after the support installation criteria is 1.59 (MCC) and 1.07 (GHB), while for Case-II is 1.65 (MCC) and 0.5 (GHB), respectively. Based on the comparative analysis, it is revealed that changing the characteristics of the slope method shows prominent results in both cases; therefore, this method can be effectively used in order to stabilize the slope in the weakest rock mass environment.

1. Introduction

A slope, either natural or human-made, is a significant concern to the civil, mining, and rock engineering professionals in designing the engineering projects like a road in a hilly area, tunnel slopes, and dams due to the variations and heterogeneity in the rock and rock mass properties [1]. Furthermore, the instability of the slope may lead to the failure and collapse of the engineering structures. Therefore, it is essential to carry out the stability analysis of the slope in order to overcome the future consequences and propose an appropriate method for the slope stability, especially in a weak rock mass environment.

Many techniques have been developed by different researchers worldwide for evaluation of the slope stability. These include the conventional (Kinematics, Limit Equilibrium Method (LEM)), empirical (Slope Mass Rating (SMR) and Q-slope), soft computing, and numerical (continuum, discontinue, and hybrid). The primary purpose of these techniques is to determine the factor of safety (FOS) of a slope at failure. FOS represents the stability of a slope, which can be defined as "the ratio between the shear strength and the shear stress." The conventional methods are commonly used only for the FOS determination. However, the conventional methods require comprehensive slope stability analysis calculations, which is time-consuming and cost-effective, i.e. LEM divides the rock mass into several slices and assumes different inter-slice normal and shear forces for determination of FOS [2]. These methods do not

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give information about the displacement or movements over a failure plan [2]. The empirical methods, i.e. SMR and Q-Slope, are effectively used in the stability of the slope, and give a proper mode of failure, whereas these methods require comprehensive field experience and do not give any information about the total displacement during slope failure and support performance. Compared to the conventional and empirical methods, the soft computing techniques have gained more attention from the researchers to apply it in slope stability [3-4]. In this regard, Daftaribesheli et al. [5] have quantified the ambiguities presented during the rock mass characterization, while applying the SMR classification for slope stability analysis using the fuzzy set theory. They presented an algorithm called the Mamdani fuzzy algorithm. Goshtasbi et al. [6] have suggested some modifications in the slope of an open-pit of the Jajarm bauxite mine wall based on the genetic algorithm. It can be seen that soft computing is applied successfully in analyzing the slope stability; however, no study in this field has yet been reported to give information about the total displacement during the slope failure and support performance. Therefore, many researchers are now focusing on the numerical methods to capture the real image of rock mass environment for stability analysis of slope due to: 1) less time-consumption, 2) giving detailed information about the mode of failure, 3) giving FOS and total displacement, 4) giving details about the performance of support, and 5) capturing the rock stress-strain behaviour and neglecting the assumptions required in the LEM analysis. There are numerous numerical methods available for slope stability analysis, as listed by [7]. (Jing, 2003); among those, the Finite Element Method (FEM) is commonly used for the stability analysis of slopes, tunnels, and underground excavations [8-11].

Using FEM as the numerical approach, the rock mass is considered and modelled as a continuum and effectively developed as an appropriate constitutive model based on the Shear Strength Reduction Approach (SSRA) for the design and stability analysis of slopes. In SSRA, the slope failure occurs due to continuous reduction (by some factors) of the shear strengths until the slope failure occurs. Furthermore, slope failure happens when the equilibrium cannot be maintained, like the limit equilibrium method. At this stage, the FEM-based developed model does not converge the data to give a solution. The critical factor at which failure occurs is called the Factor of Safety (FOS) [2]. The SSRA analysis gives an insight picture about the failure mechanisms, formation, and proposed path for failure. Further, the SSRA analysis can model the behaviour of support elements effectively; the interactions triggered by the relative stiffness of slope materials also give complete information about the performance of support elements, and can even show the deformations process at failure. In short, SSRA is the best approach to be used effectively for the slope stability analysis.

Various research works have been carried out successfully on the finite element analysis for slope stability using SSRA [2,12]. In this regard, Ataei and Bodaghabadi [13] have researched the Chador-Malu iron ore mine in order to identify the slope stability problems using the numerical and LEM methods. They concluded that some instability problems might occur by increasing the slope height. Chakraborti et al. [14] have investigated different approaches for the assessment of FOS based on SSRA using the non-linear Hoek and Brown criteria. They proposed that the global approach could be considered as a first approximation. A comparison of both methods reveals that the global approach in comparison to the local approach can lead to a deviation of up to 15% in both directions. Gupta et al. [15] have studied the failed slope at Surabhi Resort Landslide using SSRA. They suggested that proper monitoring could protect further rock failure. Hammah et al. [16] have investigated generalized Hoek and Brown failure criteria for slope stability analysis using SSRA. They found that this failure criterion was suitable to be used for analyzing slope stability. You et al. [17] have analyzed slope in dry and saturated conditions using SSRA based on the MCC and GHB failure criteria. They proposed that MCC could give an overestimation of the shear reduction parameters compared to the GHB failure criterion. It can be seen from the literature that SSRA is used only for the slope stability analysis using the MCC and GHB failure criteria. However, SSRA has not yet been applied to stabilize a slope or propose a suitable slope stabilization method in weak rock mass environments. Therefore, it is imperative to propose an appropriate slope stabilization method, especially for weak rock mass environments, to stop the rock failure in a better way. In this research work, the numerical modelling based on FEM using SSRA for slope stability analysis of cut slopes at Swat Motorway was carried out using the Mohr-Coulomb and non-linear Generalized Hoek and Brown failure criteria, and proposed a viable method for slope stabilization in a weak rock mass environment. The findings of this research work will provide a better understanding to field professionals about applying a numerical approach in a better way for slope stabilization and a safe execution of the engineering projects.

2. General and geological description of project area

The Swat motorway is known as M16 or Swat Expressway, located in Khyber Pakhtunkhwa, Pakistan; it connects the M-1 motorway to Chakdara. It is 81-Km long, starting from Karnal Sher Khan Interchange to Chakdara. The Swat Motorway project area is located in the Peshawar basin. Internally, the Peshawar basin has Quaternary sediments including fluvial gravels, sand, and lacustrine deposits. However, the outer fringes of the Peshawar basin are predominantly agglomerate derived from adjacent encircling mountains such as Malakand-Lower Swat Ranges in the north, Attock Cherat-Dara Adam Khel Ranges in the south, and the Khyber Ranges to the west [18]. The Sediments of the Peshawar basin have been impounded by the uplifting of the Attock-Cherat, whose range and movement on Main Boundary Thrust (MBT) are located at its southern fringes. These rocks range in the age from late Precambrian to early Mesozoic, as shown in Figure 1.

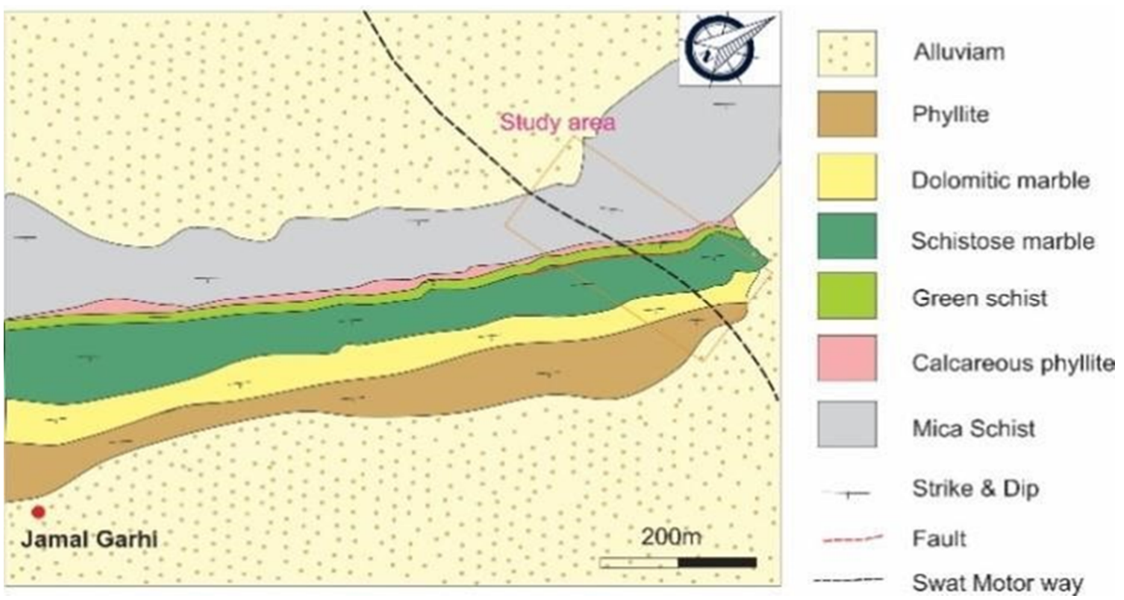


Figure 1. Rock range in the age from late Precambrian to early Mesozoic.

2.1. Research background about swat motorway

Swat Motorway is one of the highways that passes through such weak and steep zones. There are specific zones on the motorway where the chances of land sliding are more. One of these locations is near 37 Km, where excavation has been done through the rock height reaching 71 m, and is mainly located in the weak rock mass environment. Various sloped fails around this chainage can be seen in Figure 2. This slope segment is very crucial, which may cause serious risks for transportation on the motorway. It is very imperative to evaluate the slopes at Swat Motorway at section 37 Km and propose an appropriate slope stabilization method.

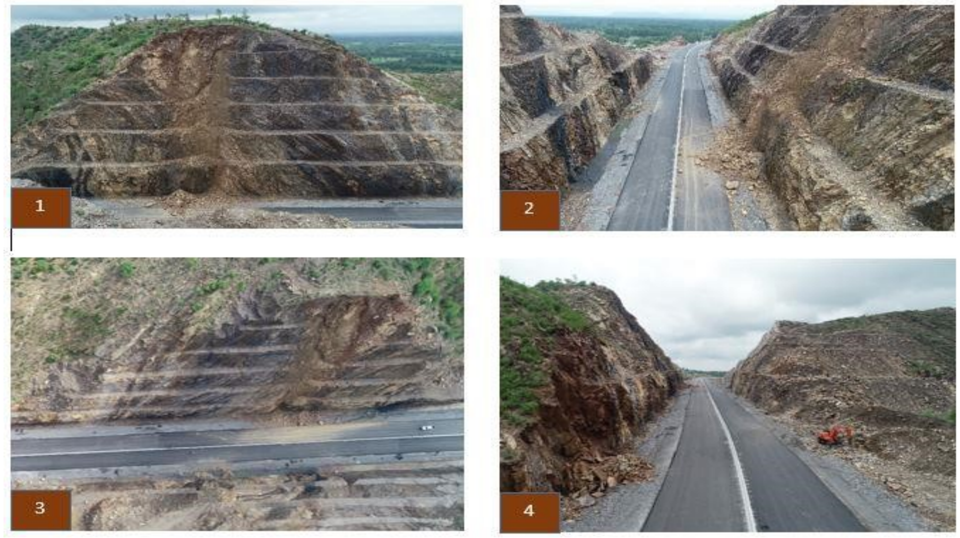


Figure 2. Failure of slope at 37 Km chainage of Swat motorway.

3. Field investigations

Considering the potential hazards in terms of failure, which are associated with cut slope at the Swat Motorway stability at chainage 37 Km, a comprehensive investigation of this incompetent rock mass was carried out. At 37 Km of the Swat Motorway, it was observed that the cut slopes were more susceptible to failure due to a weak schist rock. For the effective numerical modelling, the cut slopes were divided into two cases, i.e. Case-I and Case-II, for quickly understanding and comparative analysis, as shown in Figure 3. Case-I is located in a schist rock that can be considered partially competent compared to Case-II. Case-II is located in the weakest schist rock mass environment, and the slope is failed at the existing conditions of slope geometry.

Figure 3. Case-I and Case- II of cut slope.

4. Numerical modeling

Numerical modelling is commonly used as a tool in civil and mining engineering in order to facilitate the site engineers to assess rock mass behaviour and its effects on the engineering structures. Compared to the conventional methods, numerical modelling gives a sound sympathetic to solve complex engineering problems related to the slope, tunnel shape, size, mine layout, and design of these engineering structures [19]. Numerical modelling is considered as an exciting tool for research and innovations. The numerical methods are divided into a continuum, discontinued and hybrid [20] to analyze the engineering structures and their effects in the rock mass environment. Compared to the discontinue and hybrids methods, the continuum methods are very flexible and simple for engineering purposes [7]. Further, the finite element methods among the continuum numerical methods are commonly used for slopes, tunnels, and underground excavations [21]. In FEM, the rock mass is modelled as a continuum, and develops an appropriate constitutive model based on SSRA for slope the design and stability analysis. FEM is one of the essential numerical methods extensively used for the design and stability analysis of slope as compared to the other numerical methods based on its unique characteristics of handling the complex rock mass environment, analyzing the interrelation properties of a problem in segments, and then combined all segments to form a general solution of the problem [15]. Further, FEM is also used to resolve a complex engineering problem utilizing Plane Strain Two-Dimension (2D) Analysis, Axisymmetric 2D Analysis, and Three-Dimension (3D) Analysis. (Hudson & Feng, 2010) In this work, the 2D elasto-plastic software, i.e. RS2 developed by rocscience, is used. *RS2* is an explicit finite element method widely used for slope stability analysis and design of stable slopes. The finite element-based simulated models of slopes at the tunnel entrance and exit portal were developed using SSRA applying the finite element analysis in order to analyze the slope stability. The simulated models were developed by the following steps: 1) external boundary function 2) gravity field stresses using actual ground surface 3) finite element mesh having a uniform mesh of 6 nodes tringles element type and approximately 1500 number of mesh elements 4) discretized the mesh slope 5) established the boundary conditions and left free the slope surface, and 6) rock and rock mass properties. The slope stability analysis is carried out using SSRA based on the linear Mohr-Coulomb and non-linear Generalized Hoek and Brown failure criteria for FOS.

4.1. Shear strength reduction (SSR)

The SSR approach is unique due to its flexibility and dealing with complex rock mass environments. The slope failure occurs in SSRA due to the continuous reduction (by some factors) of the shear strengths until the slope failure occurs. Furthermore, the slope failure happens when the equilibrium cannot be maintained as the limit equilibrium method. At this stage, the FEM-based developed model does not converge the data to give a solution. The critical factor at which failure occurs is called the Factor of Safety (FOS) [2]. SSRA has several advantages over the conventional methods for slope stability analysis (LEM, Kinematics). The conventional methods only analyze FOS, and fail to analyze the support elements' total displacement and shear strain behaviour. The conventional methods are suitable for analyzing the simple slope stability, while fails to analyze the complex rock mass environments. As compared to the conventional methods, SSRA can give a complete analysis about the failure mechanisms and its mode in a better way; SSRA can effectively model the support element behaviour and interactions instigated due to the relative stiffness of slope materials, and can easily give detailed information about the deformations at the failure point. The SSR method has proved its applicability effectively where the limit-equilibrium methods give or produce ambiguous and misleading results or where LEM cannot be applied, for example, the analysis of such a slope stability in which the excavations including caverns and tunnels are constructed. The SSRA method can smoothly analyze the mentioned problem in a better way. Hence, SSRA is the best approach for stability analysis of simple and complex slopes in a better way than the conventional slope stability analysis methods.

5. Failure criteria for analysis of slope stability

5.1. Mohr-Coulomb failure criterion

The Mohr-Coulomb failure criterion is also known as the linear failure criterion, and considers that the failure envelope for rock behaves linearly. This failure criterion is widely applied in the conventional methods and software for slope stability analysis [22]. This failure criterion uses two parameters, i.e. internal friction angle (φ) and cohesion (c), to represent the shear strength of an intact rock. The shear strength of a rock is determined by Equation 1.

$$\tau = c + \sigma n \tan\varphi \tag{1}$$

According to the research works studying rocks, it is revealed that the failure envelope of a rock is not linear [22]. Therefore, it is essential to use the non-linear failure criterion to capture the actual image of the rock and rock mass at failure. For this purpose, the non-linear failure criterion, i.e. Generalized Hoek and Brown failure criterion, is also used in the slope stability analysis.

5.2. Generalized hoek and brown failure criterion

The generalized Hoek and Brown failure criterion was first introduced by Hoek (1994), and further modified by Hoek [23]. The simple theme of this failure criterion is to estimate the strength properties of a rock mass. The uniqueness of this failure criterion over the Mohr-Coulomb failure criterion considered strength envelops/failure envelops as the not linear envelope for both soils. This failure criterion is expressed as:

$$\sigma 1 = \sigma 3 + \sigma \mathrm{ci}\ (\mathrm{mb}\frac{\sigma 3}{\sigma \mathrm{ci}} + \mathrm{s})^{\mathrm{a}} \qquad (2)$$

where $\sigma 1$ and $\sigma 3$ are the major and minor principal stresses, respectively, σci is the uniaxial compressive strength, and mb, s, and a are the rock mass material constants. The rock mass material constants are estimated using Equations 4, 5, and 6, respectively.

$$mb = mi\, \exp[(\frac{GSI - 100}{28 - 14D})] \qquad (3)$$

where mi is the intact rock constant, GSI is the geological strength index, and D is the disturbance factor due to the nature of the blast.

$$s = \exp\left[\left(\frac{GSI - 100}{9 - 3D}\right)\right] \qquad (4)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \qquad (5)$$

6. Results and discussion

6.1. Geo-mechanical properties

Numerical modelling is an essential approach for the stability analysis, mainly used at the initial stage for a proper design and stability analysis of a slope. This approach requires rock properties about the site for the design and execution of the given project. The geo-mechanical properties of the representative rock samples collected from the cut slope face of Case-I and Case-II were assessed in the laboratory according to ISRM. The generalized Hoek and Brown parameters were selected using the roclab software developed by rocscience. The value of GSI was determined based on the geological characteristics and joint pattern at the slope. The results obtained are presented in Tables 1 and 2 for Case-I and Case-II based on the generalized Hoek and Brown and Mohr-Coulomb failure criteria, respectively.

Table 1. Input data of generalized Hoek and Brown criteriion for case I and case II.

Input parameters	Case I	Case II
Intact uniaxial comp. strength (MPa)	35	25
GSI	40	20
Mi	10	10
Disturbance factor	0.7	0.7
Intact modulus (MPa)	23625	16875
mb	0.37005	0.213
s	0.00017	0.000039
a	0.51137	0.522
Young modulus (MPa)	1375.7	606.688
Poisson's ration	0.3	0.3
Unit weight (MN/m^3)	0.027	0.027

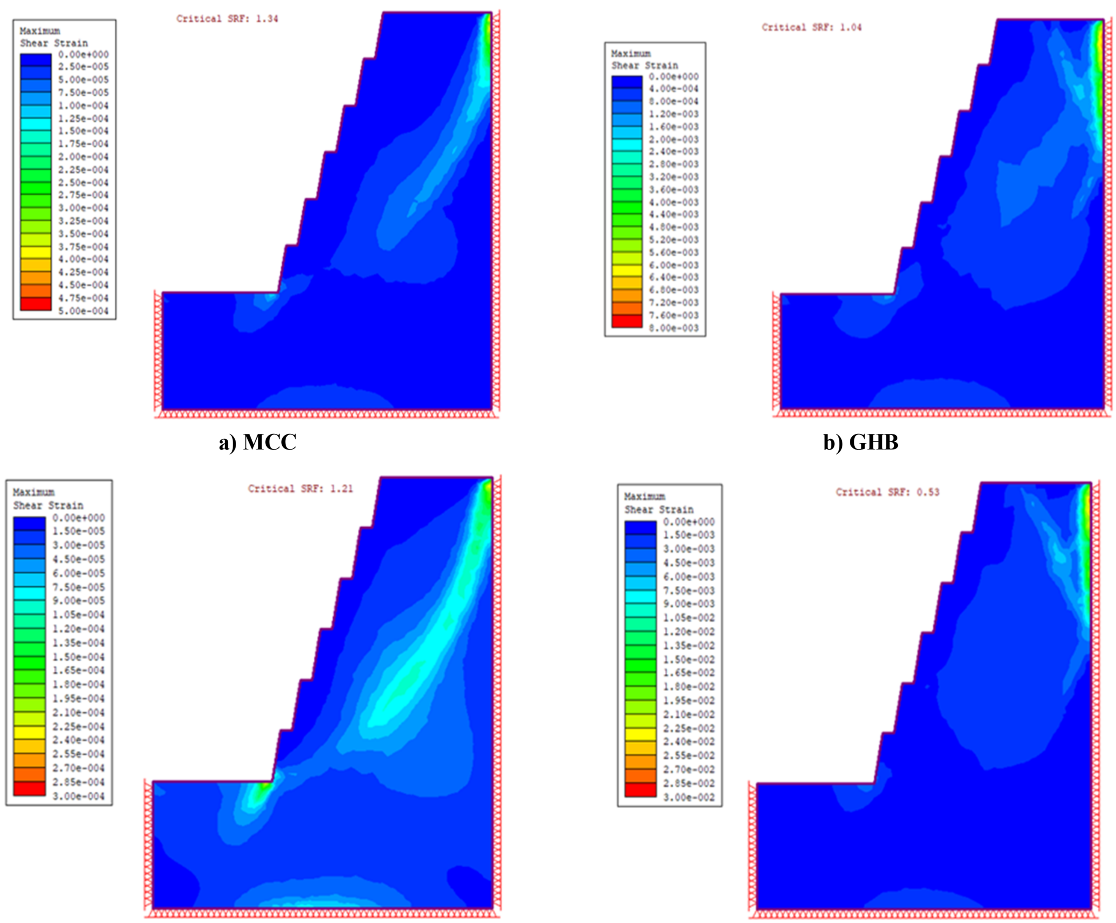
Table 2. Input data of Mohr-Coulomb failure criterion for case I and case II.

Input parameters	Case I	Case II
Young modulus (MPa)	23625	1685
Cohesion (MPa)	0.26	0.51
UTS (MPa)	3.5	2.5
Angle of internal friction (phi)	33.96^0	15.1^0

As presented in Tables 1 and 2, the rock mass in Case-I and Case-II are very weak rocks. Compared to Case-I, the rock strength properties of Case-II are very weak.

6.2. Numerical analysis of slope in its existence condition

The stability of cut slopes at the mentioned chainage was comprehensively analyzed using the SSR approach based on the generalized Hoek and Brown and Mohr-Coulomb failure criteria in the *RS2* software. Two slope stabilization techniques, i.e. changes in the characteristics of slope and support system installation, were applied in order to stabilize the cut slope. The simulated models of the cut slopes of Case-I and Case-II having a bench height of 12 ft, a width of 3 ft, and a bench face angle of almost 70^0 were developed in the software. The results obtained revealed the Critical Strength Reduction Factor (CSRF) before applying the stabilization techniques based on the MCC and GHB failure criteria; CSRF for Case-I was obtained to be 1.34 and 1.04, as shown in Figures 4a and 4b, while for Case-II, it was 1.21 and 0.53, as shown in Figures 4c and 4d, respectively.



a) MCC b) GHB

c) MCC d) GHB

Figure 4. a) Represents Case-I CSRF based on MCC, b) Represents Case-I CSRF based on GHB, c) Represents Case-II CSRF based on MCC, d) Represents Case-II CSRF based on GHB.

According to a safety viewpoint, CSRF in both cases is less than 1.5, which means that the slope is not stable. Therefore, it is essential to stabilize the slope by support installation or by changing the characteristics of the slope.

6.3. Numerical analysis after support installation

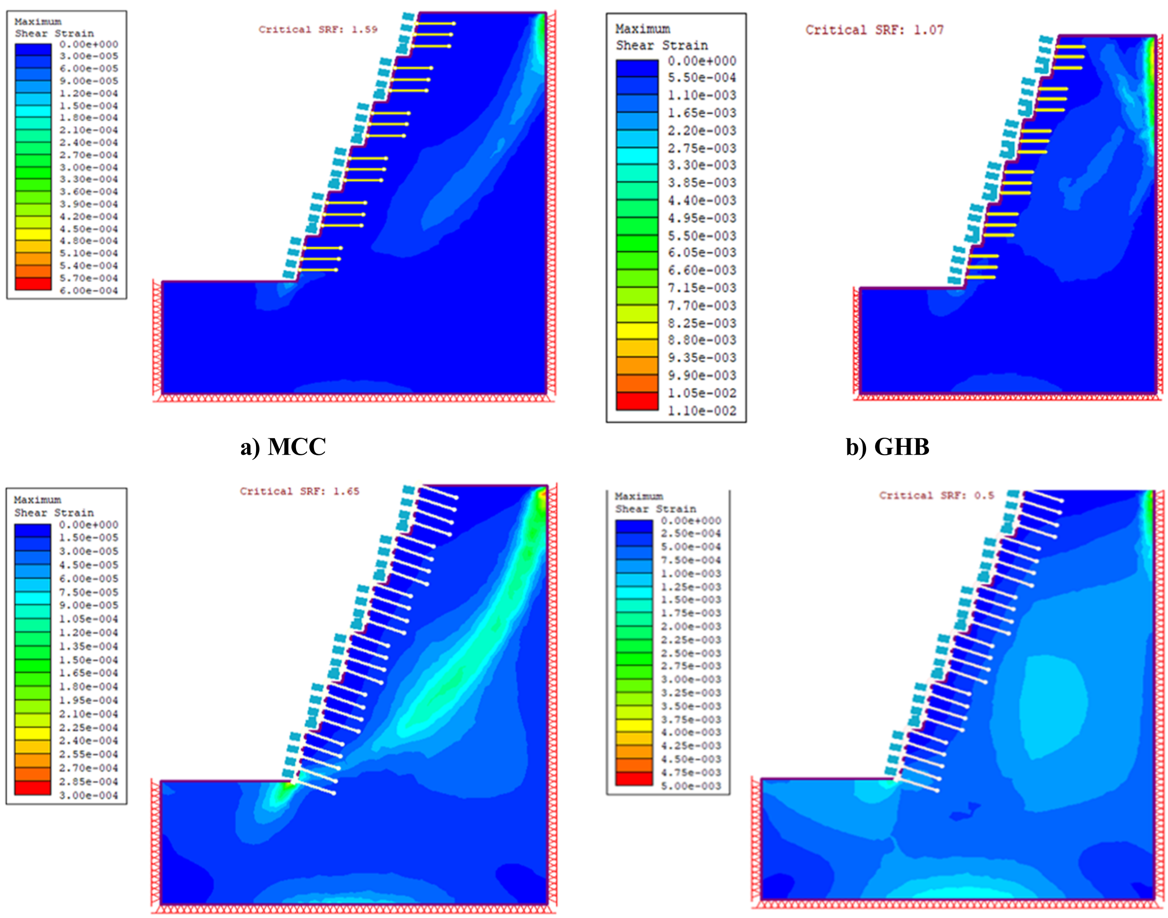
With an anchored rock bolt having a length of 4 ft, a spacing of 3 ft with shotcrete thickness of 20 mm was used to stabilize the cut slope. The other details of the support system are given in Table 3.

Table 3. Details of support system.

Bolt type	End anchored
Bolt diameter (mm)	19
Tensile capacity (MN)	0.1
Bolt modulus, E (MPa)	200000
Concrete thickness (m)	0.02
Tensile strength (MPa)	400
Formulation	Bernoulli

The CSRF for Case-I after support installation based on the MCC and GHB failure criteria was observed to be 1.59 and 1.07, respectively, shown in Figures 5a and 5b, while for Case-II it was1.65 and 0.5, respectively, as shown in Figures 5c and 5d.

It was observed that CSRF was improved in 5a, 5b, and 5c, while not improved in 5d. The maximum shear strain was observed in Case-II compared to Case-I; the zone of the shear strain was located at the top right corner of the slopes in both cases. A predominant increase in CSRF was observed, while stability analysis of cut slope based on MCC compared to the GHB failure criterion.



a) MCC b) GHB

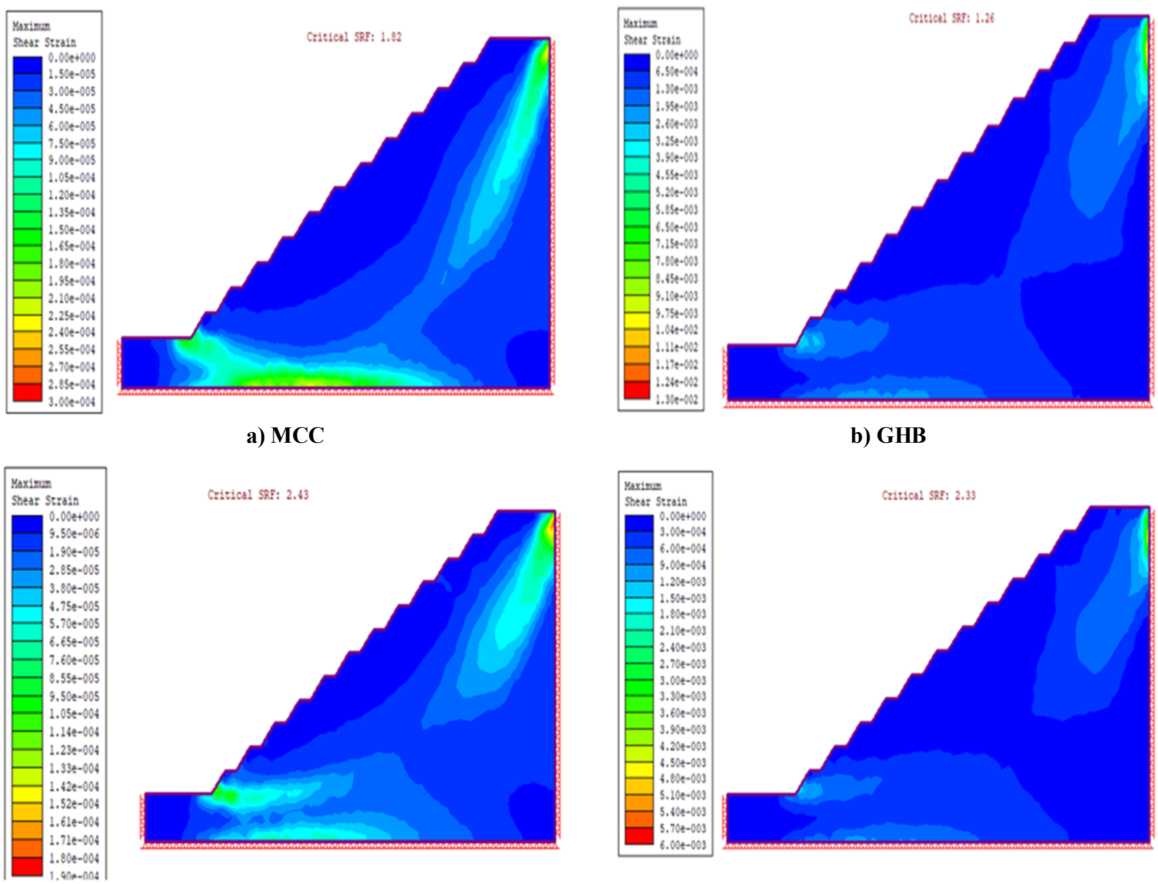
c) MCC d) GHB

Figure 5. a) Represents Case-I CSRF based on MCC, b) Represents Case-I CSRF based on GHB, c) Represents Case-II CSRF based on MCC, d) Represents Case-II CSRF based on GHB.

6.4. Changes in characteristics of slope

Even applying the support system discussed in the previous section did not give a satisfactory outcome, although the slope was stable based on MCC, while not stable based on the GHB failure criterion. Therefore, changes in the characteristics of the slope method was applied. For this purpose, the bench height was reduced from 12 ft to 6 ft, the bench width increased from 3 ft to 4 ft, and the bench slope face angle of each bench was reduced to 50^0.

CSRF for Case-I by changing the slope characteristics based on the MCC and GHB failure criteria was observed to be 2.43 and 2.33, as shown in Figures 6a and 6b, respectively, while for Case-II, it was 1.82 and 1.24, as shown in Figures 6c and 6d, respectively.



a) MCC b) GHB

c) MCC d) GHB

Figure 6. a) Represents Case-I CSRF based on MCC, b) Represents Case-I CSRF based on GHB, c) Represents Case-II CSRF based on MCC, d) Represents Case-II CSRF based on GHB.

6.5. Significance of proposed method for slope stabilization

CSRF was improved in both cases of the cut slope. However, CSRF was predominantly increased after applying the changes in the characteristics of the slope compared to the support installation method for slope stabilization. more, the maximum shear strain was decreased in the changes in the characteristics of the slope method compared to the support installation. The rock mass environment was weak; therefore, it did not require any blasting to change to the characteristics of the slope. Compared to the slope stabilization using the support system, the changes in slope characteristics method were cost-effective, and might prevent progressive failure and displacement. Therefore, it can be proposed that in the weak rock mass environment, the changes in the characteristics of the cut slope method can be effectively applied as a slope stabilization technique.

7. Conclusions

The following conclusions were drawn from the work:

i. The Case-I and Case-II slope models were developed and analyzed using SSRA in the *RS2* software based on the Mohr-Coulomb and Hook and Brown failure criteria. The numerical evaluation result showed that CSRF before applying the stabilization methods was 1.34 (MCC) and 1.04

(GHB) for Case-I and 1.21 (MCC) and 0.53 (GHB) for Case-II.

ii. CSRF for Case-I after support installation criteria was 1.59 (MCC) and 1.07 (GHB), while for Case-II it was 1.65 (MCC) and 0.5 (GHB), respectively.

iii. CSRF for Case-I after changing the characteristics of slope was observed to be 2.43 (MCC) and 2.33 (GHB), while for Case-II it was 1.82 (MCC) and 1.26 (GHB), respectively.

iv. It was revealed from the numerical analysis based on the Mohr-Coulomb and Hook and Brown failure criteria that the changes in the characteristics of the slope method were much better than the support system installation method for the stabilization of slope. Therefore, the changes in the characteristics of the slope method was proposed to be used for an effective slope stabilization in a weak rock mass environment.

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پیشنهاد یک روش پایدارسازی برای شیب در یک محیط توده سنگ ضعیف با استفاده از مدل‌سازی عددی: مطالعه موردی

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چکیده:

طراحی یک شیب پایدار در توده سنگ‌ها به دلیل رفتار ناهمسانگرد توده سنگ، کار کاملاً پیچیده ای است. در این کار تحقیقاتی، شیب‌های برش در بزرگراه سوات در ضعیف‌ترین سنگ شیست با رویکرد کاهش مقاومت برشی (SSR) با استفاده از نرم‌افزار ۲ بعدی RS2 مبتنی بر المان محدود تحلیل عددی شده است. شیب با توجه به ماهیت سنگ به دو حالت تقسیم شد. هر بخش از شیب با دو روش پایداری: ۱) تغییر ویژگی های شیب ۲) نصب سیستم نگهداری بر اساس معیارهای شکست موهر-کلومب (MCC) و معیار شکست تعمیم یافته Hoek و Brown (GHB) به منظور پیشنهاد مناسب ترین روش برای پایداری شیب تجزیه و تحلیل شده است. نتایج به‌دست‌آمده نشان می‌دهد که ضریب کاهش مقاومت بحرانی (CSRF) قبل از اعمال روش‌های پایداری و سیستم نگهداری به ترتیب ۱/۳۴ (MCC) و ۱/۰۴ (GHB) برای مورد مطالعاتی اول و ۱/۲۱ (MCC) و ۰/۵۳ (GHB) برای مورد مطالعاتی دوم بوده است. CSRF برای مورد مطالعاتی اول پس از تغییر ویژگی های شیب به ترتیب ۲/۴۳ (MCC) و ۲/۳۳ (GHB) مشاهده می شود، در حالی که برای مورد مطالعاتی دوم به ترتیب ۱/۸۲ (MCC) و ۱/۲۶ (GHB) است. CSRF برای مورد مطالعاتی اول پس از نصب نگهدارنده‌ها ۱/۵۹ (MCC) و ۱/۰۷ (GHB) است، در حالی که برای مورد مطالعاتی دوم به ترتیب ۱/۶۵ (MCC) و ۰/۵ (GHB) است. بر اساس تجزیه و تحلیل مقایسه‌ای، مشخص شد که تغییر ویژگی‌های روش شیب نتایج قابل توجهی را در هر دو مورد نشان می دهد. بنابراین می‌توان از این روش برای تثبیت شیب در ضعیف ترین محیط توده سنگی به طور موثر استفاده کرد.

کلمات کلیدی: پایداری شیب، کاهش مقاومت برشی، معیار شکست هوک و براون تعمیم یافته، معیارهای شکست موهر کولب.