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Research article

# Stability analysis in 4th Gol-e-Gohar Mine with residual parameters method and strain softening approach

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***Abstract***

Today, numerous surface and underground mining and construction projects can be found worldwide, built on a rock bed and surrounded by rock. Open pit mines are considered the primary sector for mineral production in the mining industry. The issue of slope stability is crucial for the economy and safety of open pit mines. Slope stability should be based on the determination of tectonic and lithological parameters and the determination of mine boundaries. It is illogical to allocate one slope for the entire walls of the mine, which are made of different materials and have different structural conditions. The purpose of slope stability analysis is to maintain a stable slope while mining activities continue. This research was conducted on slope stability analysis in the No. 4 Gol-e-Gohar mine in Sirjan. Geotechnical characteristics and necessary information for numerical modeling were obtained through mine visits, surveys, and tests on rock samples from exploratory boreholes. Based on two-dimensional modeling using PHASE 2D software, the CSFH behavior model (cohesion softening - friction hardening) for the northern wall will enhance the overall strength of the rock mass. However, this behavior model is not suitable for medium and weak-quality rocks. For rocks with softening behavior, the CSFS behavior model (cohesion softening - friction softening) provides more realistic results. Furthermore, to investigate the effect of schistosity plates, transisotropic behavior parameters were determined based on direct cutting tests on schistosity surfaces, and stability analysis was conducted. It was concluded that the orientation of the Turq surfaces in schist layers has a significant effect on the strength of these layers, leading to larger displacement values than other models.

## 1. INTRODUCTION

Today, many surface and underground mining and construction projects can be found all over the world, built on a rock bed and surrounded by rock. In the field of mining, open pit mines are considered the main sector for the production of mineral products worldwide. In open pit mines, the extraction walls in areas where the rock has high quality can have an almost vertical slope, but in areas where the rock quality has decreased, the wall slope should be lower to create stable conditions [1]. It is necessary to know the rock mass strength and rock mass deformation behavior for the design of rock slopes. The more information there is about the rock mass strength parameters (maximum and residual), the less expensive the design for such rock structures. Estimating the mechanical parameters of jointed rock masses is one of the most complex issues in rock mechanics science [2].

Many researchers have developed behavioral models that describe the strength and deformation behavior of the rock mass. Because many parameters affect the strength and deformation behavior of the jointed rock mass, it is practically impossible to develop a behavior model that accurately predicts the rock mass strength. The common methods used to measure these parameters are the plate loading test to measure the modulus of deformation and the in-situ block-cutting test to estimate the strength parameters of the rock mass. These tests can be performed when the exploratory tunnels are dug, although the cost of performing such tests is high [3].

Although the reverse analysis method, based on the measurements made at the project site to estimate the strength parameters and deformation of the rock mass along with the project's progress, can be helpful, its disadvantage is that it does not provide the necessary parameters for the design in the feasibility stage to the engineers [3]. Few attempts have been made to develop methods to utilize the properties of the rock mass for indirect estimation of its deformation and strength [3]. If the reduced parameters are not determined accurately, the proper design of a rock structure cannot be achieved. To properly design underground structures, both maximum and reduced strength parameters will be needed.

In this research, following geological surveys in the studied area, the characteristics of each rock unit were determined. Subsequently, different rock units were classified based on the Geological Strength Index (GSI) system. The properties of intact rock related to each of the rock units were determined based on the results of uniaxial and triaxial compression tests performed on rock samples obtained from exploratory boreholes. Using the maximum strength parameters, reduced values were determined according to the criteria of the constitutive behavioral model of Cohesion Softening – Friction Softening (CSFS) and the constitutive behavioral model of Cohesion Softening - Friction Hardening (CSFH) to analyze the stability of the most critical section of the north wall of mine No. 4. Finally, to investigate the effect of schistosity plates, parameters of the transversely isotropic behavior were determined based on the results obtained from direct shear tests on schistosity surfaces, and the analysis of the stability of the critical section was carried out considering the transversely isotropic behavior.

## 2. RESEARCH BACKGROUND

Until now, most of the research has focused on obtaining the maximum strength parameters, with limited efforts made to estimate the reduced strength parameters of the rock mass.

The geological strength index method developed by Hoek [4] is one of these methods. This method uses the characteristics of the in-situ rock and the jointing conditions to estimate the deformation and strength of the rock mass. The Geological Strength Index values are estimated based on the geological descriptions of the rock mass, making it a very good method when direct access to the rock mass is not available [4]. The Geological Strength Index method focuses on two factors: 1- rock structure and 2- surface condition of the rock block. This method includes complete mechanical parameters, such as Hook and Brown's strength parameters, Mohr-Coulomb's strength parameters, and the elastic modulus for design purposes [3].

When the stresses exceed the maximum strength of the rock, failure will occur. As the rock enters the plastic zone, its strength parameters will decrease from their maximum values, and these parameters are referred to as reduced parameters. In the initial version of the Hoek and Brown failure criterion, the maximum parameters are determined using the triaxial test. In the generalized version of the Hoek and Brown criterion, the maximum parameters are determined using $\sigma_{ci}$, geological strength index (GSI), rock parameter ($m_i$ which is the material constant for intact rock samples in the Hooke and Brown criterion) and also the disturbance factor D are determined. The geological strength index is used to establish a relationship between field observations and parameters of Hook and Brown's failure criteria. Therefore, it should be noted that the parameters GSI, $\sigma_{ci}$,, D and $m_i$, are used to determine the parameters of the generalized Hooke and Brown criteria, a, s and $m_b$.

Based on the studies conducted by Russo [5], when using the GSI classification to estimate the rock mass strength in the slope of the open pit mine, it has been concluded that assuming the rock mass is in the disturbed group, there is a good match between the estimated strength values and the estimated values from the regression analysis. Although this hypothesis needs to be validated by more data, it confirms that the reduction of $GSI^{peak}$ in order to estimate the reduced strength is a reasonable assumption.

Russo [5] suggested that the value of the reduced geological strength index ($GSI^{res}$) should be considered equal to 0.36 value of the maximum geological strength index ($GSI^{peak}$). Such a relationship may ignore strength values for very weak rock masses. On the other hand, for high-quality rock mass, the value of $GSI^{res}$ may be considered high.

Ribachi [6] suggested using Eqs. (1) to (3) to obtain the reduced strength of jointed rock mass. that in these equations:

$$m_b^{res} = 0.65\ m_b^{peak} \tag{1}$$

$$s^{res} = 0.04\ s^{peak} \tag{2}$$

$$\sigma_c^{res} = 0.2\ \sigma_c^{peak} \tag{3}$$

$m_b$: the strength parameter related to the rock type according to Hook and Brown (reduced value of $m_i$ parameter for intact rock), S: Hook and Brown's strength parameter related to rock mass properties, $\sigma_{ci}$: uniaxial compressive strength of healthy rock (MPa).

Eqs. (1) to (3) can be used for rock masses that have joints, a thin layer of filler material, or are slightly weathered. Therefore, the reduction of GSI from the maximum state to the residual state is expressed as Eq. (4) [6].

$$\mathrm{GSI}^{res} = 0.7\mathrm{GSI}^{peak} \tag{4}$$

Cai and Kaiser [3] have suggested that the value of $GSI^{peak}$ be determined based on two parameters of maximum block volume ($V_b^{peak}$) and maximum joint conditions ($J_c^{peak}$) and then by reducing these parameters to the reduced values, namely $V_b^{res}$ and $J_c^{res}$, calculated the geological strength index in the reduced state of $GSI^{res}$.

In summary, various efforts have been made to estimate the strength of jointed rock masses. It is logical to decrease the geological strength index from the maximum value to the reduced value, as the fracture of the rock mass is accompanied by the crushing and loss of the roughness of the grains.

### 2.1. Decreased Geological Strength Index

In the main rock mass classification systems, two of the most important parameters that control the quality of the rock mass and, of course, the strength and deformation of the rock mass are block volume and joint conditions. The volume of the block is affected by the spacing and continuity of the joints. The condition of the joint is controlled by the roughness of the joints, weathering, and filling materials. These are important factors that should be determined to estimate the reduced strength of the rock mass [3].

According to the principles of the Geological Strength Index system, rock mass strength is controlled by two factors: block size and joint conditions. The same principle will be valid for the mass of rock that is broken in a reduced state. In other words, the geological strength index in the reduced state of $GSI^{res}$ is a function of the reduced block volume $V_b^{res}$ and the reduced joint condition $J_c^{res}$ [3].

Therefore, $GSI^{res}$ can be calculated as a function of $V_b^{res}$ and $J_c^{res}$ according to Eq. (5) [3].

$$\mathrm{GSI}^{res} = \frac{26.5 + 8.79 \ln \mathrm{J_c}^{res} + 0.9 \ln \mathrm{V_b}^{res}}{1 + 0.0151 \ln \mathrm{J_c}^{res} - 0.0253 \ln \mathrm{V_b}^{res}} \tag{5}$$

The mechanical properties of uniaxial compressive strength of intact rock ($\sigma_{ci}$) and material constant for intact rock samples in the Hook and Brown criterion ($m_i$) will remain unchanged for the reduced state. What will change is the size of the block and the condition of the joint (especially the roughness). Therefore, Hook and Brown's failure criterion for the reduced strength of the jointed rock mass is written as Eq (6) [3].

$$\sigma_1 = \sigma_3 + \sigma_C \left(\mathrm{m_b}^{res} \frac{\sigma_3}{\sigma_C} + s^{res}\right)^{a^{res}} \tag{6}$$

In this regard, $m_b{}^{res}$, $s^{res}$ and $a^{res}$ are the strength parameters of Hooke and Brown criterion in reduced state for rock mass. It should be noted that these parameters are obtained based on $GSI^{res}$ and the same relationships introduced to obtain these parameters in the maximum state, so these equations used to obtain the maximum strength parameters will be valid and usable to obtain reduced strength parameters [3].

$$\mathrm{m_b}^{res} = \mathrm{m_i} \exp\left(\frac{\mathrm{GSI}^{res} - 100}{28}\right) \tag{7}$$

$$\mathrm{S}^{res} = \exp\left(\frac{\mathrm{GSI}^{res} - 100}{9}\right) \tag{8}$$

$$a^{res} = 0.5 + \frac{1}{6}\left(e^{\frac{-\mathrm{GSI}^{res}}{15}} - e^{\frac{-20}{3}}\right) \tag{9}$$

Another failure criterion that was discussed in this research was the Mohr-Coulomb criterion, which is also widely used in the field of rock engineering. The strength parameters of this criterion include the internal friction angle φ and adhesion C. In order to determine the strength parameters of the Mohr-Coulomb criterion, the relations proposed by Hook and Brown are used [7]. These relations are given as Eqs. (10) and (11).

$$\phi = \sin^{-1}\left[\frac{6\,a\,m_b\,(s + m_b\,\sigma_{3n})^{a+1}}{2(a+1)(2+a) + 6\,a\,m_b(s + m_b\,\sigma_{3n})^{a-1}}\right] \tag{10}$$

$$c = \frac{\sigma_{ci}\,[(1+2a)s + (1-a)m_b\,\sigma_{3n}](s + m_b\sigma_{3n})^{a-1}}{(1+a)(2+a)\sqrt{\frac{1 + (6\,a\,m_b(s + m_b\sigma_{3n})^{a-1})}{(1+a)(2+a)}}} \tag{11}$$

that in these Eqs.: $\sigma_{ci}$: Uniaxial compressive strength of intact rock (MPa), $a$: Strength parameter of Hooke and Brown criterion, $m_b$: the strength parameter of Hooke and Brown criterion, $\sigma_{3n}$: Minimum principal stress (MPa), ϕ: Internal friction angle (degrees), C: Cohesion (MPa).

The ratio of $\frac{GSI^{res}}{GSI^{peak}}$ depends on the value of $GSI^{peak}$ of the rock mass [3]. Studies conducted in several sites in Japan and several other countries to determine $GSI^{res}$ based on $GSI^{peak}$, have shown that the ratio of $\frac{GSI^{res}}{GSI^{peak}}$ is between 0.37 and 0.57 [3]. For very weak rocks, the value of $GSI^{res}$ is equal to $GSI^{peak}$ [3]. The quantitative method developed by Cai and Kaiser uses two parameters of block volume and joint conditions, and can only be applied when information from exploratory boreholes is available. Based on studies conducted on the maximum and reduced GSI values obtained using this method, Eq. (12) was presented to determine the value of $GSI^{res}$, which closely matched the performed experiments [3].

$$\mathrm{GSI^{res}} = \mathrm{GSI^{peak}}\ \mathrm{e}^{-0.0134\ \mathrm{GSI^{peak}}} \tag{12}$$

## 3. POST-FAILURE BEHAVIOR OF ROCKS

Hook and Brown [7] have suggested that the behavior after failure of the rock mass is different according to its quality and provided guidelines related to the reduced strength of the rock mass. These instructions are based on the types of rocks. For a very good and strong rock mass with a high Geological Strength Index (GSI>75), the rock mass will behave in an elastic-brittle manner, and the dilation angle (ψ) will be equal to $\frac{\varphi^{PEAK}}{4}$. For the mass of rocks with medium jointing (25<GSI <75), the strain-softening behavior is predicted, and the value of the dilation angle will be equal to $\frac{\varphi^{PEAK}}{8}$. For very weak rocks (GSI<25), complete elastic-plastic behavior without dilation is predicted. In other words, a reduction is not considered for the strength parameters, and the failure criterion is currently being reduced [7].

In the stress-strain diagram, the slope of the part after failure is called the drop modulus (M). When the drop modulus is equal to zero, the behavior after rock failure will be plastic, and when the drop modulus is equal to infinity, the rock behavior after failure will be brittle [8]. Strain-softening behavior can also include brittle behavior (drop modulus equal to infinity) and it can also include complete elastic-plastic behavior (drop modulus equal to zero). Therefore, it can be stated that brittle behavior and perfect plastic behavior are a special type of strain-softening behavior [8].

The strain-softening behavior is defined as a gradual decrease from the maximum failure criterion to the reduced failure criterion, which is controlled by the softening parameter η [8]. The softening parameter η, which is the maximum plastic shear strain, expresses the transition from the maximum state to the reduced state. The critical value of the softening parameter is shown by the parameter η'. This transition is defined in such a way that when the softening parameter is equal to zero, there is an elastic region, and the softening region will exist when 0 < η < η'. The reduced state occurs when η > η'. The value of the softening parameter controls the transition between the softening state and the reduced state (in other words, the value of the softening parameter will determine in which region of the stress-strain curve the rock is located) [8]. Fig. 1 shows the strain-softening behavior (weakening of strength) for the test of confined compressive strength.

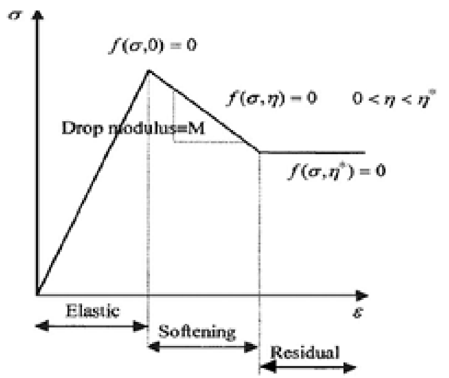


**Fig. 1. Stress-strain curve for strain-softening behavior [8].**

### 3.1. Determining The Drop Modulus For Rock Mass With Strain-Softening Behavior After Failure

Based on the laboratory results and field data, it has been observed that the deformation and behavior of rock masses are highly dependent on the quality of the rock mass and the limiting stress [8]. Alejano et al [8] proposed the estimation of rock mass drop modulus according to the quality of the maximum rock mass, which is determined based on $GSI^{peak}$. The elastic modulus (E) and the drop modulus (M) have been found to depend on the quality of the rock mass and the limiting stress, therefore, the value of the drop modulus function ($f_n$) will also depend on these parameters (Eq. 13) [8].

$$f_n(\sigma_3\ , GSI^{peak}) = 1 - \frac{E}{M} \tag{13}$$

The value of function $f_n(\sigma_3, GSI^{peak})$ can be determined by Eq. (14), for the strain-softening behavior model with constant drop modulus [8].

$$f_n(\sigma_3, GSI^{peak}) = 8 - 0.08\, GSI^{peak} \quad \text{for } 25 < GSI^{peak} < 75 \tag{14}$$

where in this equation: $f_n$: value of the loss modulus function, $GSI^{peak}$: maximum geological strength index.

The drop modulus will also be determined in the form of Eq. (15) [8].

$$M = \frac{E_{rm}}{0.08\, GSI^{peak} - 7} \quad \text{for } 25 < GSI^{peak} < 75 \tag{15}$$

where in this equation: M: drop modulus (MPa), $E_{rm}$: Rock mass deformation modulus (MPa), $GSI^{peak}$: maximum geological strength index.

If more complex strain-softening models are used, along with considering the dependence of the drop modulus on the confining stress, the drop modulus function will be determined according to Eq. (16) [8].

$$f_n(\sigma_3, GSI^{peak}) = \left[\left(\frac{225 - GSI^{peak}}{1000}\right).\sigma_3 + \left(\frac{55 - 0.6\, GSI^{peak}}{8}\right)\right] \tag{16}$$

and the drop modulus will be determined according to Eq. (17) [8].

$$M = \frac{1000\, E_{rm}}{GSI^{peak}.\sigma_3 + 75\, GSI^{peak} - 225\sigma_3 - 5875} \tag{17}$$

To obtain the deformation modulus of the rock mass ($E_{rm}$), it is possible to use Eq. (18) proposed by Hoek [9]. This equation considers more effective factors on the deformation, such as the elastic modulus of intact rock ($E_i$), disturbance factor (D), and the Geological Strength Index (GSI).

$$E_{rm} = E_i.\left(\frac{1 - \frac{D}{2}}{1 + e^{\left(\frac{75+25D-GSI^{peak}}{11}\right)}}\right) \tag{18}$$

## 3.2. Behavioral Model Of Cohesion Softening – Friction Hardening

In the behavioral model of CSFS, by reducing GSI from the maximum value to the reduced value, the strength parameters will be determined in the residual state. In fact, to determine the strength parameters, the properties of intact rock including Young's elastic modulus ($E_i$), uniaxial compressive strength of intact rock ($\sigma_{ci}$), and intact rock parameter ($m_i$) will not change. It was discussed that the value of $GSI^{peak}$ is reduced to $GSI^{res}$ and based on $GSI^{res}$, the residual strength parameters will be determined. The maximum strength parameters will also be determined based on the relationships developed for the Mohr-Coulomb criterion.

The reduction of cohesion strength and friction angle parameters in CSFS behavioral model is shown in Fig. 2.

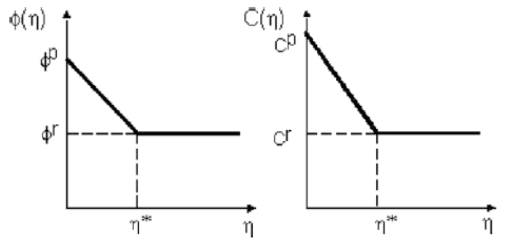


**Fig. 2. Reduction of cohesion strength and frictional strength in CSFS behavioral model [8].**

The method for determining the maximum and minimum strength parameters of the rock mass by considering the CSFS behavioral model is provided in Table 1.

**Table 1. Maximum and reduced strength parameters in the CSFS model**

| model | Cohesion | | Friction | |
|---|---|---|---|---|
| | Peak $GSI^{PEAK}$ | Residual $GSI^{res}$ | Peak $GSI^{PEAK}$ | Residual $GSI^{res}$ |
| CSFS | $C_{peak}$ | $C_{res}$ | $\phi_{Peak}$ | $\phi_{res}$ |

## 3.3. Behavioral Model Of Reducing Adhesion - Increasing The Angle Of Internal Friction

According to research conducted by Osterberg [10], frictional strength is caused by sliding between components. As particles move and friction intensifies, the cohesion strength between particles decreases. For very low plastic strains, friction is negligible. However, with an increase in strain, friction significantly increases while cohesion decreases [10].

The behavioral model proposed by Hajiabdolmajid et al. [11] is the model of decreasing cohesion and increasing internal friction (CSFH). According to this theory, when a rock mass experiences fracture, the cohesion between particles decreases and reaches zero. During the change of shape and movement of particles, friction increases. In the CSFH behavior model, the yield angle increases with the increase of plastic strain. In the first phase of rupture, the origin of the strength of a component is the cohesion strength of that component. With the development of microcracks and the increase of inelastic strains, the frictional strength increases progressively and takes the place of cohesion strength, and the main source of the strength of an object is its frictional strength [11].

Hajiabdolmajid et al. [11] proposed a simple process to reduce the cohesion strength by considering the softening parameter $\varepsilon_c^p \neq 0$, as well as the frictional strength at its maximum capacity without considering the hardening parameter $\varepsilon_f^p = 0$. The parameter $\varepsilon^P$ measures the path of plastic strain and is introduced as the effective plastic strain. According to the results of the experiments conducted by Hajiabdolmajid et al. [11], which led to the presentation of the CSFH behavioral model, it was suggested that the value of cohesion strength in the reduced state should be considered equal to 0.3 of its maximum value. Also, when the hardening parameter is considered equal to zero, it is suggested that the frictional strength in the reduced state should be considered equal to its initial value. The strength parameters based on the model proposed by Hajiabdolmajid et al. are shown in Fig. 3.

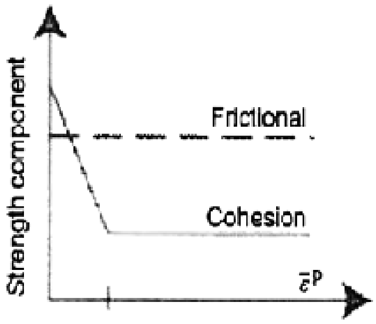


**Fig. 3. Strength parameters in the CSFH behavioral model, considering the full value for frictional strength [11].**

How to determine the maximum and minimum adhesion and friction strength parameters of the rock mass considering the CSFH behavior model is given in Table 2.

**Table 2. The maximum and reduced strength parameters in the CSFH model**

| model | Cohesion | | Friction | |
|---|---|---|---|---|
| | Peak $GSI^{PEAK}$ | Residual $GSI^{res}$ | Peak $GSI^{PEAK}$ | Residual $GSI^{res}$ |
| CSFH ($\varepsilon_f^p = 0$) | $C_{peak}$ | $0/3\ c_{peak}$ | $\phi_{Peak}$ | $\phi_{res}$ |

## 4. TRANSVERSELY ISOTROPIC BEHAVIORAL MODEL

The behavioral models that have been examined so far have been considered with the assumption that the rock is isotropic. However, due to the existence of some preferential orientations or layers, folds, etc., the behavior of many rocks is anisotropic. Anisotropy behavior is the dependence of rock properties on the direction of loading [12]. Transversely isotropic behavior is a special type of anisotropy that occurs due to layered planes and their extension [12]. Layering plates or schistosity play an important role in the mechanical behavior and fracture process of these types of rocks [12]. Due to the computational complexity and the difficulty of determining the required elastic constants for rocks with such behavior, a simple form of anisotropy called Transisotropic is used in designs [12]. Transversely isotropic behavior is a special type of anisotropy in which there is a specific axis so that the measured property appears the same when rotated around this axis, which means that it is transversely isotropic in this direction, if by leaving this axis, the characteristic to be measured will change [12]. The property of transversely isotropic makes many properties that appear similar in the transverse aspect, to be different in the vertical aspect. A transversely isotropic environment is an environment that has an isotropic plane (layering or schistosity planes) in which the properties are constant in different directions and also an isotropic axis that is perpendicular to the isotropic plane. Fig. 4 shows the transverse isotropic behavior model where x and y are the isotropic plane and z axis is perpendicular to the isotropic plane [12].

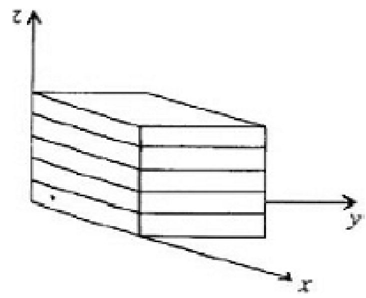


**Fig. 4. Showing the isotropic plane and its perpendicular axis in transversely isotropic behavior [12].**

The maximum strength of rocks with transversely isotropic behavior changes due to the presence of weak plates in different directions. Based on the results of the tests conducted on rock samples with schistosity by Naseri et al. [13], when the angle of the schistose plane is 30 degrees with respect to the loading axis, at the level of constant confining stress, the rock has the least amount of strength. On the other hand, with the increase of the confining stress level and the constant angle of the schistose plane with respect to the loading axis, the rock strength has increased.

### 4.1. The parameters of the transverse isotropic environment

The transverse environment is introduced by the following 5 constants [12]:

$E_1$: Deformation modulus in the isotropic plane

$E_2$: Deformation modulus perpendicular to the isotropic plane

$\vartheta_1$: Poisson's ratio in the isotropic plane

$\vartheta_2$: Poisson's ratio perpendicular to the isotropic plane

$G_2$: Shear modulus perpendicular to the isotropic plane

### 4.2. Determining the parameters of the transverse isotropic environment

In the case that the rock with transversely isotropic behavior includes a group of weak plates parallel to each other, the elastic constants of the transversely isotropic medium can be determined. If the isotropic elastic constants are E, $\vartheta$, and G, and the shear stiffness and normal stiffness of the weak plates are $K_s$ and $K_n$, respectively, and the distance between these weak surfaces is s, by using Eqs. (19) to (23), the elastic constants of the transversely isotropic behavior are determined [12].

$$E_1 = E \tag{19}$$

$$\frac{1}{E_2} = \frac{1}{E} + \frac{1}{(K_n)(S)} \tag{20}$$

$$\vartheta_1 = \vartheta \tag{21}$$

$$\vartheta_2 = \frac{E_2}{E}\ \vartheta \tag{22}$$

$$\frac{1}{G_2} = \frac{1}{G} + \frac{1}{(K_s)(S)} \tag{23}$$

that in these Eqs.:

E is the deformation modulus in MPa, G is the shear modulus in MPa, S is the distance of weak surfaces in mm, $K_s$ is the shear hardness in MPa/mm and $K_n$ is the normal hardness in MPa/mm.

## 5. CASE STUDY

Gol-e-Gohar iron ore complex is one of the major iron ore deposits in Iran, located 60 km southwest of Sirjan city in Kerman province. The complex includes 6 separate anomalies of iron ore. Gol-e-Gohar Mine No. 4 is one of the iron ore mines in this area, located about 3 kilometers from Mine No. 1. In terms of its structural position, Gol-e-Gohar mining area is situated on the eastern edge of the Sanandaj-Sirjan transform zone and on the edge of the large Khairabad depression, which marks the boundary between the Sanandaj-Sirjan zone and the Urmia-Dokhtar zone [14].

In general, the region represents a metamorphic complex containing schist, amphibolite, gneiss, marble, and other rocks specific to metamorphic regions <14>. The studied area of mine No. 4 is generally covered by alluvium of the present era and has limited outcrops of metamorphic rocks in the south and southwest, sedimentary rocks in the east, and a granite intrusive mass in the south of the mine [14]. Talc schist, quartz muscovite schist, garnet schist, amphibolite schist, and marble are among the sequences of the mentioned complex. The geological studies carried out in Gol-e-Gohar mine 4 showed that the exposed materials in the range primarily control the main parameters of the rock mass from the perspective of rock engineering. Anisotropy is the main characteristic of metamorphic rocks, including schist. Examining the qualitative index of the rock mass as well as the shear parameters shows the weakness of the materials of the Gol-e-Gohar mine No. 4. On the other hand, weathering and alteration, as destructive factors, have severely affected the geomechanical parameters of the rocks and reduced the strength of the rocks to a considerable extent. It is not possible to include this phenomenon in the analysis of wall stability, which has been investigated qualitatively from a geological point of view, except through numerical modeling and its quantification of the geomechanical parameters of rocks. Most of the rock units of the mine are made up of schist rocks. Anisotropy is the main characteristic of metamorphic rocks, including schists. Considering that the strength properties of these rocks are different and sometimes unclear in different directions, the importance of the effect of anisotropy on the strength has been essential. Next, in order to take into account, the reduced parameters of the rock units, the stability analysis of the critical section of the north wall of mine No. 4 was done using CSFS and CSFH behavioral models.

## 6. THE POSITION AND GEOMETRY OF THE SELECTED SECTION

The north wall of mine No. 4 has weaker strength than the other walls, which necessitates a numerical analysis of its stability. Several factors contribute to destabilizing the wall, including the presence of weathered schist layers and the mechanism of torque surfaces. In Gol-e-Gohar mine No. 4, the tectonized environment exhibits numerous discontinuities, behaving like soil where crushed rock is mixed with materials and the number of joints decreases with distance. The analysis is focused on the mine wall rather than the discontinuities. Considering the high walls (15 meters) and large-scale slopes in Gol-e-Gohar mine, the environment is treated as continuous for the analysis.

To establish the geometry of the selected section, the PHASE 2D software was utilized [15]. The chosen section for stability analysis comprises layers of alluvium, chlorite schist, and quartz schist in the upper levels, layers of gneiss and quartz schist in the middle steps, and layers of chlorite schist, quartz schist, and magnetite in the low step. The geometry of the desired section, along with its constituent layers and applied boundary conditions, is depicted in Fig. 5. The geological strength index (GSI) associated with each rock unit was determined based on the condition of different rock units in the northern wall and the measurements taken. Furthermore, the intact rock properties for each rock unit were established from tests conducted on rock samples obtained from exploratory boreholes.

Due to the presence of schist samples with schistosity planes, the values obtained for these samples were based on the results of the tests performed on the schistosity planes. Finally, the rock mass strength parameters were calculated using ROCKLAB software [16]. The characteristics and strength parameters of different rock units related to the analyzed section are provided in Table 3. To validate the constructed model and the data used, the section of the collapsed slope in the north wall of mine No. 4 was utilized, and the accuracy of the constructed model and data was confirmed.

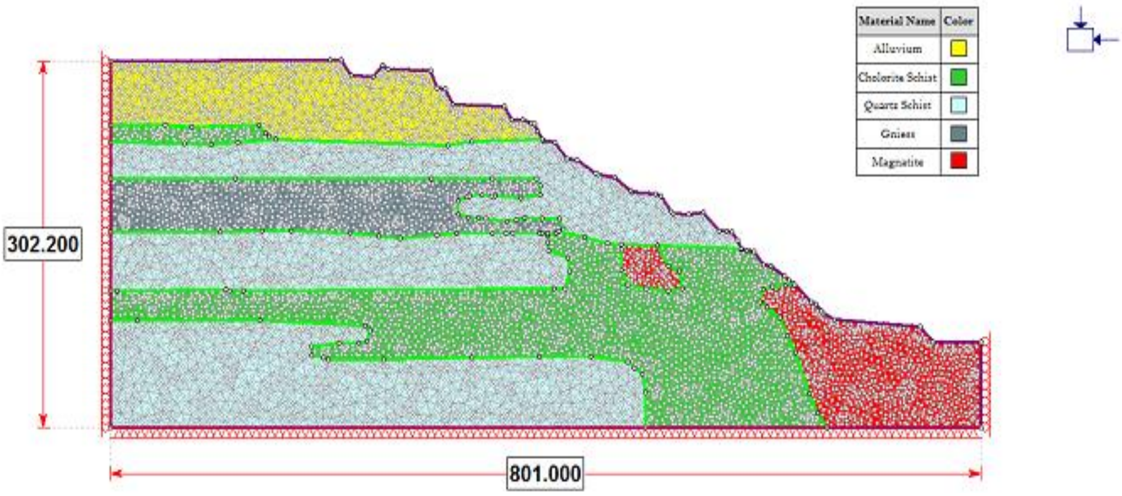


**Fig. 5. Geometry of selected section for stability analysis.**

**Table 3. Geomechanical properties and Strength parameters of selected cross-section rock units for stability analysis**

| Rock Unit | $\gamma(\frac{MN}{m^3})$ | *GSI* | $c\ (MPa)$ | $\varphi^{\circ}$ | $\vartheta$ | $E_{rm}$(Mpa) | $\sigma_t$(Mpa) |
|---|---|---|---|---|---|---|---|
| Alluvium | 0.0185 | 42 | 0.125 | 23.15 | 0.23 | 286.32 | 0.017 |
| Chlorite Schist | 0.027 | 37 | 0.166 | 26.96 | 0.25 | 381.77 | 0.0089 |
| Quartz Schist | 0.026 | 43 | 0.272 | 33.33 | 0.25 | 922.56 | 0.0147 |
| Gniess | 0.027 | 52 | 0.752 | 31.14 | 0.22 | 2617.96 | 0.0592 |
| Magnetite | 0.038 | 50 | 1.153 | 29.64 | 0.22 | 1234.01 | 0.008 |

## 7. ANALYZING THE STABILITY OF THE SELECTED SECTION BY CONSIDERING THE BEHAVIOR MODEL OF CSFS

In this article, the relationship between the maximum and minimum principal stress for rock samples in Gol-e-Gohar mine No. 4 is assumed to be linear. On the other hand, in the investigations that took place, the rock units in the north wall of mine No. 4 are mostly weak and have low strength. The aim of this research was to investigate the effect of cohesion strength and frictional strength parameters of the rock mass in the maximum and minimum state on the stability of the northern wall. For this reason, the Mohr-Coulomb failure criterion and the strength parameters of this failure criterion has been used in this research. In the CSFS behavioral model, by reducing GSI from the maximum value to the reduced value, the strength.

parameters will be determined in the reduced state. To determine the reduced strength parameters, the characteristics of intact rock including $E_i$, $m_i$, $\sigma_{ci}$ will not change and only the value of $GSI^{peak}$ will change to the reduced value

($GSI^{res}$), and based on $GSI^{res}$, the reduced strength parameters will be determined.

### 7.1. Determining The Reduced Strength Parameters Based On CSFS Behavioral Model

To determine the maximum strength parameters while considering the $GSI^{peak}$ values, the value of each rock unit was calculated using equation 12. The $GSI^{res}$ values for each rock unit are listed in Table 4. By applying the Mohr-Coulomb criterion in the plastic state in the PHASE 2D software, the parameters from Table 5 are assigned in both the maximum and reduced states in the Properties and Define Materials sections of the software.

**Table 4. $GSI^{peak}$ and $GSI^{res}$ values by different rock units for the selected section**

| Rock Unit | $GSI^{res}$ | $GSI^{peak}$ |
|---|---|---|
| Alluvium | 24 | 42 |
| Chlorite Schist | 22.53 | 37 |
| Gniess | 26 | 52 |
| Quartz Schist | 24.16 | 43 |
| Magnetite | 25.58 | 50 |

Next, the reduced strength parameters were determined based on $GSI^{res}$ values according to

Table 1. The reduced and maximum strength parameters considering CSFS behavioral model are given in Table 5.

**Table 5. Maximum and reduced strength parameters based on CSFS behavioral model**

| Rock Unit | $C^{peak}$ (Mpa) | $\varphi^{peak}$ | $E_{rm}^{peak}$ $(Mpa)$ | $\sigma_t^{peak}$ (Mpa) | $C^{res}$ (Mpa) | $\varphi^{res}$ | $E_{rm}^{res}$ (Mpa) | $\sigma_t^{res}$ (Mpa) | $\Psi = \frac{\varphi}{8}$ |
|---|---|---|---|---|---|---|---|---|---|
| Alluvium | 0.125 | 23.15 | 286.32 | 0.017 | 0.055 | 13.47 | 162.44 | 0.003 | 2.9 |
| Chlorite Schist | 0.166 | 26.96 | 381.77 | 0.0089 | 0.087 | 17.78 | 261.55 | 0.0022 | 3.37 |
| Gniess | 0.1752 | 31.14 | 261.96 | 0.0592 | 0.284 | 16.41 | 898.79 | 0.005 | 3.9 |
| Quartz Schist | 0.272 | 33.33 | 922.56 | 0.0147 | 0.129 | 21 | 500.38 | 0.0024 | 4.16 |
| Magnetite | 1.153 | 29.64 | 1234.01 | 0.008 | 0.495 | 16.54 | 472.36 | 0.001 | 3.7 |

### 7.2. The results of stability analysis considering CSFS behavioral model

From the obtained result in Fig. 6, it can be seen that the highest shear strain was created in the alluvial layers at the level 1752 to 1755 and the schist chlorite layer at the levels 1577 to 1605. The reliability coefficient determined by the shear strength reduction method was also determined to be 1.16, which shows that the intended slope did not collapse considering the CSFS behavior model, but there was a possibility of instability. In PHASE 2D software, when the stresses applied to a component are higher than the maximum strength of that component, that component has reached the yield point and entered the phase after its failure, and reduced strength parameters will be used.

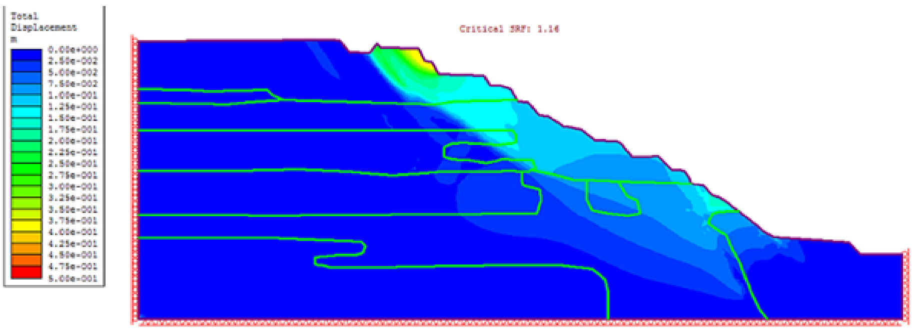


**Fig. 6. A view of the total displacement created in the slope considering CSFS behavior.**

The highest amount of displacement was in the upper steps of the slope in the alluvium and quartz schist layers, and also in the lower steps near the heel of the slope in the chlorite schist layer. The amount of displacement in the alluvium layer was about 40 cm, in the quartz schist layer about 15 cm, and in the chlorite schist layer was 17 cm. Most of the upper levels of the slope, including the alluvial layers, quartz schist, gneiss, as well as the chlorite schist layer in the lower level of the slope, have yielded (red color) and have entered the post-failure phase. The reduced parameter has been used, while other parts have not reached their yield point (blue color) (Fig. 7).

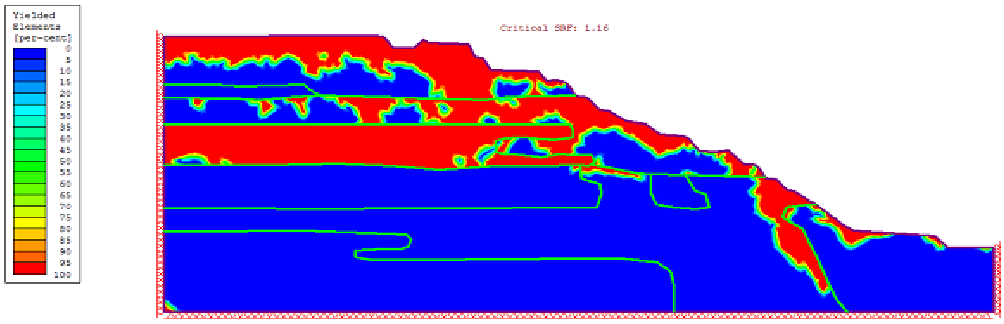


**Fig. 7. A view of the yielded parts in stability analyzing considering the CSFS behavior model (yield percentage).**

## 8. ANALYZING THE STABILITY OF THE SELECTED SECTION BY CONSIDERING THE CSFH BEHAVIORAL MODEL

Hajiabdolmajid et al [11] proposed a simple process for reducing the cohesion strength by considering the softening parameter $\varepsilon_c^p \neq 0$, and also reducing the frictional strength to its maximum capacity without considering the stiffness parameter $\varepsilon_f^p = 0$. They suggested that the value of cohesion strength in the reduced state should be 0.3 of its maximum value. Additionally, when the hardening parameter is considered to be zero, the frictional strength in the reduced state should be equal to its initial value [11].

### 8.1. Determining The Reduced Strength Parameters Based On The CSFH Behavioral Model

To determine the strength parameters of the CSFH behavioral model, we followed the theory of Hajiabdolmajid et al. The reduced cohesion strength was considered to be 0.3 of its maximum value. Additionally, when the hardening parameter is zero, the frictional strength in the reduced state is equal to its initial value. Other rock mass strength parameters, such as tensile strength and rock mass deformation modulus, were determined based on $GSI^{res}$ values. The maximum and reduced cohesion and friction strength parameters, as per the CSFH behavior model, are presented in Table 6. By applying the Mohr-Coulomb criterion in the plastic state in the PHASE 2D software, the parameters from Table 6 were assigned in both the maximum and reduced states in the Properties and Define Materials sections of the software.

### 8.2. The Results Of The Stability Analysis Considering The CSFH Behavioral Model

From the obtained results according to Fig. 8, it can be seen that the highest shear strain occurred in the alluvial layers at levels 1725 to 1740 and in the schist chlorite at levels 1577 to 1605. The reliability coefficient was determined to be 21/1, which indicates that the desired slope has not failed, considering the CSFH behavioral model.

The highest amount of displacement occurred in the upper steps of the slope in the alluvium and quartz schist layer, as well as in the lower steps near the heel of the slope in the chlorite schist layer. The displacement in the alluvial layer measured about 34 cm in step 1754, while in the quartz schist layer it was approximately 8 cm in step 1695, and in the chlorite schist layer it was 13 cm in step 1592.

The majority of the upper levels of the slope, including alluvial layers, quartz schist, almost all parts of the gneiss layer, as well as parts of the chlorite schist layer in the lower level of the slope, have yielded (red color). The stresses have exceeded their strength and entered the post-failure phase, and reduced parameters have been used. In magnetite, at levels 1577 to 1548, some parts have yielded, considering that these parts formed the heel of the slope. After sliding in the chlorite schist layer above it, a concentration of tension is created, causing surrender. Due to the high strength of the magnetite layer, no significant displacement has occurred. The displacement of the entire slope and the yielded parts is shown in figs. 8 and 9.

**Table 6. Maximum and reduced strength parameters based on the CSFH behavioral model.**

| Rock Unit | $C^{peak}$ $(Mpa)$ | $\varphi^{peak}$ | $E_{rm}^{peak}$ $(Mpa)$ | $\sigma_t^{peak}$ $(Mpa)$ | $C^{res}$ $(Mpa)$ | $\varphi^{res}$ | $E_{rm}^{res}$ $(Mpa)$ | $\sigma_t^{res}$ $(Mpa)$ | $\Psi = \frac{\varphi}{8}$ |
|---|---|---|---|---|---|---|---|---|---|
| Alluvium | 0.125 | 23.15 | 286.32 | 0.017 | 0.0375 | 23.15 | 162.44 | 0.003 | 2.9 |
| Chlorite Schist | 0.166 | 26.96 | 381.77 | 0.0089 | 0.0498 | 26.96 | 261.55 | 0.0022 | 3.37 |
| Gniess | 0.752 | 31.14 | 2617.96 | 0.0592 | 0.225 | 31.14 | 898.79 | 0.005 | 3.9 |
| Quartz Schist | 0.272 | 33.33 | 922.56 | 0.0147 | 0.0816 | 33.33 | 500.38 | 0.0024 | 4.16 |
| Magnetite | 1.153 | 29.64 | 1234.01 | 0.008 | 0.3459 | 29.64 | 472.36 | 0.001 | 3.7 |

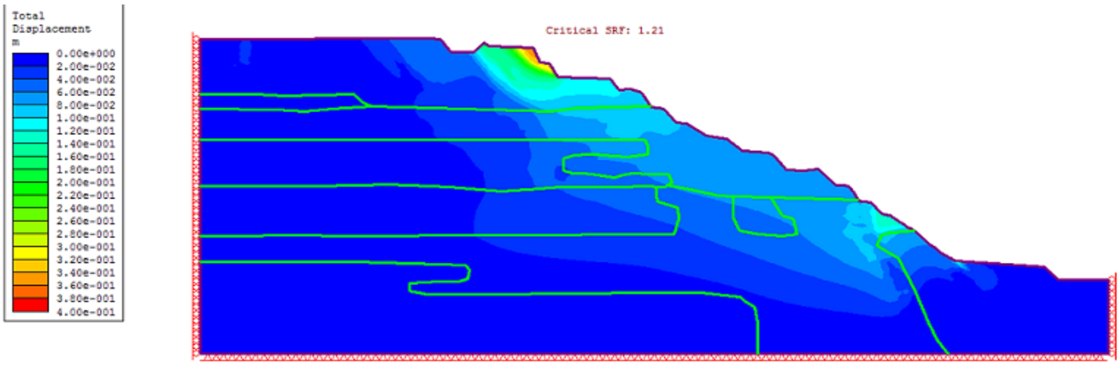


**Fig. 8. A view of the total displacement created in the slope considering the CSFH behavioral Model.**

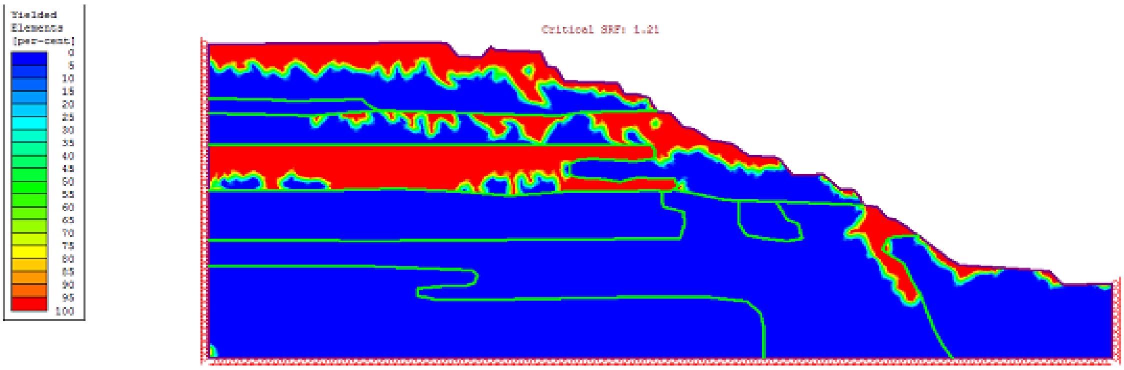


**Fig. 9. A view of the yielded parts considering the CSFH behavioral model (yield percentage).**

## 9. ANALYZING THE STABILITY OF THE CHOSEN SECTION BY CONSIDERING TRANSVERSELY ISOTROPIC BEHAVIOR

As the geological studies in Gol-e-Gohar mine No. 4 have shown, the major part of the mine is composed of metamorphic rocks and schists. Anisotropy is the main characteristic of metamorphic rocks, including schist, as their strength properties vary in different directions, making the anisotropy effect essential. In a transversely isotropic behavior, the strength of the rock changes by altering the direction of the schistose plane relative to the loading axis.

### 9.1. Determining The Parameters Of The Transverse Isotropic Environment

In order to determine the parameters of transversely isotropic behavior, it is necessary to determine the shear and normal strength on the schistosity plates. The results of direct shear tests performed on schist rock samples and the elastic constants determined according to equations 19 to 23 are given in Table 7. According to experts in the field of rock mechanics, the normal stiffness is often between 3 and 6 times the shear stiffness. However, it should be noted that the normal and shear stiffness are measured based on laboratory tests on different surfaces, such as joints, schistosity levels, etc., therefore the values can be different. Based on the results of laboratory tests obtained from Iran's Mines Development and Renovation Organization in Gol-e-Gohar Mining Complex in 1401, this value is considered equal to 10 <17>. For other rock units including alluvium, gneiss, and magnetite, transversely isotropic behavior was not considered due to the lack of schistosity. The maximum strength parameters were also based on the parameters used in the isotropic analysis mode. In the PHASE 2D software, the Transversely Isotropic analysis mode is selected in the elastic section under the Properties and Define Materials sections. In this mode, it is possible to define the elastic parameters $E_1$, $E_2$, $\vartheta_1$, $\vartheta_2$ and $G_2$.

### 9.2. The Results Of Stability Analysis Considering Transverse Isotropic Behavior

From the results shown in Fig. 10, it is evident that the total displacement of the slope, considering the transversely isotropic behavior, was 70 cm, indicating instability. The most significant displacement occurred in the quartz schist layer at levels 1607 to 1637, with a displacement of 66 cm, and in the chlorite schist layer at levels 1594 to 1605. Additionally, there was a displacement of about 52 cm in the alluvial layer at levels 1697 to 1724.

**Table 7. Values of elastic constants determined for schist units**

| Rock Unit | Shear Stiffness $(\frac{Mpa}{mm})$ | S (mm) | $E_1$ *(Mpa)* | $\vartheta_1$ | $G_1$ *(Mpa)* | $E_2$ *(Mpa)* | $\vartheta_2$ | $G_2$ *(Mpa)* | α (Degree) | Normal Stiffness $(\frac{Mpa}{mm})$ |
|---|---|---|---|---|---|---|---|---|---|---|
| Quartz Schist | 3.88 | 350 | 922.56 | 0.25 | 369.024 | 863.87 | 0.23 | 290.17 | 60 | 38.8 |
| Chlorite Schist | 4.9 | 310 | 381.77 | 0.25 | 152.708 | 372.41 | 0.24 | 138.77 | 60 | 49 |

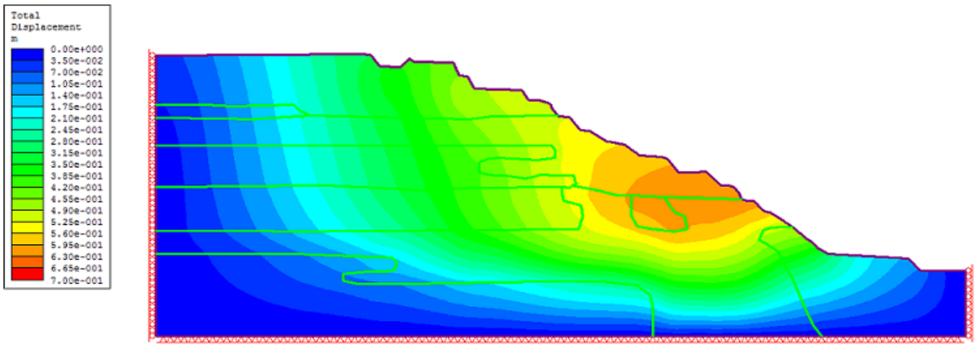


**Fig. 10. A view of the total displacement in the slope considering the transversely isotropic behavior.**

## 10. COMPARING THE RESULTS OF BEHAVIORAL MODELS

The comparison of the minimum principal stress values and their effect on the behavior of the rock mass in the north wall of mine No. 4, as well as the comparison of the displacement values based on the analyses carried out, are given below.

### 10.1. Comparison Of Minimum Principal Stress

The minimum principal stress value based on the results of PHASE 2D software was 8.4 MPa in the CSFS behavior model (Fig. 11) and 7.6 MPa in the CSFH behavior model (Fig. 12).

With the increase of stress $\sigma_3$, the behavior will tend towards plastic behavior. Therefore, for the elements that enter the post-failure phase, the slope of the stress-strain diagram will be gentler, and the drop modulus, which is the slope of the part after the peak of the stress-strain diagram, will be less. In the case of the CSFH behavior model, the value of stress $\sigma_3$ is less, and the behavior tends to be brittle. This has caused the slope of the stress-strain diagram in the post-failure part for the elements that enter the post-failure phase to be sharpened compared to the CSFS mode, and the drop modulus for the CSFH behavioral model should increase.

Table 8 shows the minimum principal stress values based on the results obtained from stability analysis with CSFS and CSFH models for parts of the slope that have reached yielding and experienced the post-failure phase. It should be noted that the points for CSFS and CSFH behavioral models were selected corresponding to each other, and these selected points are considered as representative points to examine the differences between these two behavioral models.

**Table 8. Minimum principal stress values in the yielded parts of the slope**

| Rock Unit | Model : CSFS $\sigma_3$(Mpa) | Model:CSFH $\sigma_3$(Mpa) |
|---|---|---|
| Alluvium | 0.68 | 0.55 |
| Chlorite Schist | 4.06 | 3.73 |
| Quartz Schist | 1.28 | 0.95 |
| Gniess | 2.6 | 2.34 |
| Magnetite | 1.22 | 1.18 |

Due to the dependence of the drop modulus on the minimum principal stress ($\sigma_3$), the drop modulus function was used to determine the drop modulus. It has been observed that the modulus of deformation (E) and the drop modulus (M) depend on the quality of the rock mass and the confining stress, so the value of the $f_n$ function will also depend on these parameters. Equation 17 was used to determine the drop modulus, and the resulting values are listed in Table 9.

**Table 9. The drop modulus values determined based on the results of CSFS and CSFH behavioral models**

| Rock Unit | Model : CSFS M(Mpa) | Model:CSFH M(Mpa) |
|---|---|---|
| Alluvium | -100.48 | -101.32 |
| Chlorite Schist | -98.72 | -100.45 |
| Quartz Schist | -320 | -326.81 |
| Gniess | -1079.66 | -1100 |
| Magnetite | -527.693 | -530 |

### 10. 2. Comparison Of Drop Modulus Values

As evident from the results, the drop modulus values determined for the CSFH behavioral model were larger than those for the CSFS model at the corresponding points. In the case of the CSFH behavioral model, the rock's behavior tends to be brittle, whereas in the CSFS behavioral model, the rock's behavior tends to be ductile. This issue is illustrated in Fig. 13.

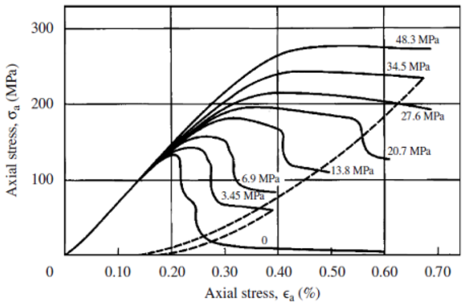


**Fig. 13. The stress-strain curve at different confining stress levels [3].**

### 10. 3. Comparing displacement values

As concluded so far, in the CSFS model, the behavior of the material tends to be ductile, while in the CSFH model, the behavior tends to be brittle. The materials in the CSFS behavioral model have shown more flexibility, causing more strains to occur compared to the CSFH model. This is also evident in the comparison of displacement values. The maximum displacement determined based on the CSFS behavioral model was 50 cm in the alluvium layer at levels 1725 to 1755, including two steps. In contrast, based on the CSFH model, the maximum displacement recorded in these levels was 40 cm. Displacement values obtained by considering the transversely isotropic model were greater than in other models, with 52 cm of displacement occurring in the alluvium layer. The largest displacement, 70 cm, was observed in the layers of quartz schist and chlorite schist when considering transversely isotropic behavior, while in other models, the largest displacement was in the upper levels and in the alluvium layer. The difference in displacement amounts and locations is related to the characteristics of schist rocks.

In the analysis based on the CSFS and CSFH models, it was not possible to consider the condition of the schistose plates in the schist units for the cross-section. As a result, the strength parameters in these models are not taken into account, causing the displacement values and the location of the maximum displacement to differ from the results of the analysis with the transversely isotropic behavior.

## 11. COMPARISON OF THE FALL THAT HAPPENED IN THE MINE AREA WITH THE RELEVANT MODEL

To validate the constructed section for stability analysis in this research, the collapsed slope of the northern wall of mine No. 4 was utilized. The stability analysis of the slope section is depicted in Fig. 14, showing a factor of safety of 0.79, which was entirely justified based on the observations made in mine No. 4. The primary cause of the collapse on the north wall was attributed to the presence of talc schist layer and low-strength schist units. The analysis results of the constructed model indicate that instability and sliding occurred on the talc schist layer.

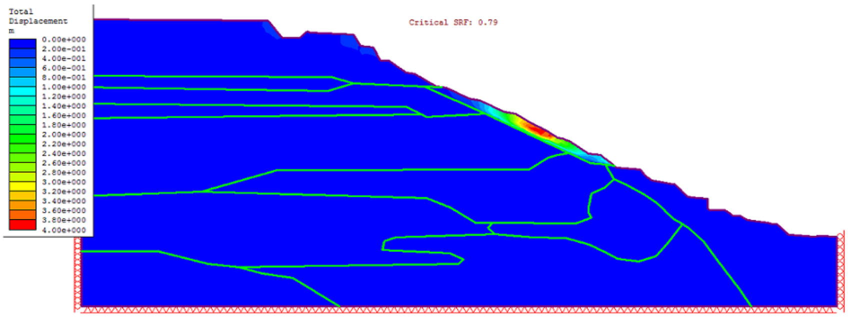


**Fig. 14. A view of the condition of the collapsed slope section in the north wall of the mine by the method of reducing shear strength.**

**Fig. 15. A view of the failure that happened on the north wall of mine No. 4.**

## 12. CONCLUSION

Applying CSFH and CSFS behavior models to all rock masses of any quality will not be accurate. The CFH behavior model leads to brittle behavior and ignores plastic strains. The CSFH model is suitable for high-quality rock masses with brittle behavior, as they reach their reduced state with a steep slope after failure and show less strain. Compared to the CSFS model, it provides a better understanding of the brittle behavior of high-quality rock masses. For rocks of medium or weak quality, the CSFH model increases the overall strength of the rock mass. The factor of safety obtained for the CSFH model is larger than that obtained for the CSFS model.

In general, for weak and medium rock masses that exhibit plastic behavior and softening, the CSFH behavior model will cause an overall increase in strength. Consequently, the determination and selection of the desired parameters may be associated with errors. Investigations have revealed that the rock units in mine No. 4 are weak to medium in strength, resulting in widespread collapses in the north wall of the mine. Therefore, using the CSFH behavior model for these rock units can lead to an increase in the strength parameters and, consequently, an increase in the reliability coefficient of the slope, which can have irreparable consequences. The transversely isotropic behavior model has proven to be very suitable for the northern wall of mine No. 4, where most of the rock units consist of schist, as it takes into account the state of the schistosity plates in the analysis.

Therefore, considering the transversely isotropic behavior along with the yield criterion in the real modeling of slope behavior plays a significant role. In the end, it should be mentioned that the results obtained from the analyses have been validated by the collapsed slope in the north wall of mine No. 4 and also in the form of reverse analysis by the radar data available in the Gol-e-Gohar mining area, and the obtained results are reliable.

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**ATTACHMENT**

**- How To Determine The Strength Parameters For The Alluvial Layer In CSFS Behavioral Model**

In this research, ROCLAB software was used to determine the strength parameters. The input parameters to the software included parameters $\sigma_{ci}$، $m_i$، $E_i$، γ، $GSI^{peak}$. The value of these parameters for the alluvium layer is given in Table 1.

**Table 1. Geological strength index and intact rock parameters for the alluvium unit**

| Parameter | Value |
|---|---|
| $\sigma_{ci}$ | 26 Mpa |
| $m_i$ | 6 |
| $E_i$ | 6550 Mpa |
| $\gamma$ | 0.0185 $\frac{MN}{m^3}$ |
| $GSI^{peak}$ | 42 |

Quantitative conversion of parameters of Hooke and Brown's criterion to parameters of Mohr-Columb criterion is done by fitting an average linear relationship to non-linear criterion of Hooke and Brown for a range of minimum principal stress ($\sigma_3$). The relations introduced in reference 18 are used for the connection between the parameters of the Hooke and Brown criterion and the Mohr-Columb criterion. The Mohr-Columb fracture criterion fitted to the alluvium unit with the specifications mentioned in Table 1 in order to determine the parameters of the Mohr-Columb fracture criterion is shown in Fig. 1. Based on this, the software will provide the parameters of friction angle, cohesion, modulus of deformation of rock mass and tensile strength in the maximum mode. In order to determine the reduced parameters based on the CSFS behavioral model, first the value of $GSI^{res}$ is determined based on equation 12 and by placing this value instead of $GSI^{PEAK}$ and without changing the parameters related to intact rock, the parameters of the friction angle, cohesion, modulus of deformation of rock mass and tensile strength will be obtained in the reduced state. These values for the alluvium layer are given in Table 2.

**- How To Determine The Strength Parameters For The Alluvial Layer In The Behavioral Model**

In the CSFH behavioral model, the parameters of the friction angle, cohesion, rock mass deformation modulus, and tensile strength in the maximum state are similar to the CSFS behavioral model. They will be determined based on the solution mentioned in the previous section and using ROCLAB software. According to the principles of the CSFH behavioral model discussed earlier, the value of the friction angle in the maximum and reduced state will be equal, and no reduction will be considered for this parameter. For the cohesion parameter, the reduced value equal to 0.3 of its maximum value will be considered. For the alluvium layer, we will have:

$$c_{res} = 0/3 \times 0/125 = 0/0375 \quad (1)$$

In order to determine the tensile strength parameters and deformation modulus in the CSFH behavior model for the reduced state, the value of $GSI^{res}$ is determined based on Eq. (12) and will replace the value of $GSI^{PEAK}$. The parameters related to intact rock will not change. These values for the alluvium layer are given in Table 3.

**Table 2. Maximum and reduced strength parameters based on CSFS behavioral model**

| Rock Unit | $C^{peak}$ (Mpa) | $\varphi^{peak}$ | $E_{rm}^{peak}$ (Mpa) | $\sigma_t^{peak}$ (Mpa) | $C^{res}$ (Mpa) | $\varphi^{res}$ | $E_{rm}^{res}$ (Mpa) | $\sigma_t^{res}$ (Mpa) |
|---|---|---|---|---|---|---|---|---|
| Alluvium | 0.125 | 23.15 | 286.32 | 0.017 | 0.055 | 13.47 | 162.44 | 0.003 |

**Table 3. Maximum and reduced strength parameters based on the CSFH behavioral model**

| Rock Unit | $C^{peak}$ (Mpa) | $\varphi^{peak}$ | $E_{rm}^{peak}$ (Mpa) | $\sigma_t^{peak}$ (Mpa) | $C^{res}$ (Mpa) | $\varphi^{res}$ | $E_{rm}^{res}$ (Mpa) | $\sigma_t^{res}$ (Mpa) |
|---|---|---|---|---|---|---|---|---|
| Alluvium | 0.125 | 23.15 | 286.32 | 0.017 | 0.0375 | 23.15 | 162.44 | 0.003 |

### -Determining The Parameters Of The Transversely Isotropic Behavior For The Quartz Schist Layer

Eqs. (19) to (23) have been used to determine the parameters of the transversely isotropic environment. The calculation results for the quartz schist layer are provided below.

$$E_1 = 922/56 \text{ (Mpa)} \tag{2}$$

$$E_2 = 863/87 \text{ (Mpa)} \tag{3}$$

$$\vartheta_1 = 0/25 \tag{4}$$

$$\vartheta_2 = 0/23 \tag{5}$$

$$G_2 = 290/17 \text{ (Mpa)} \tag{6}$$

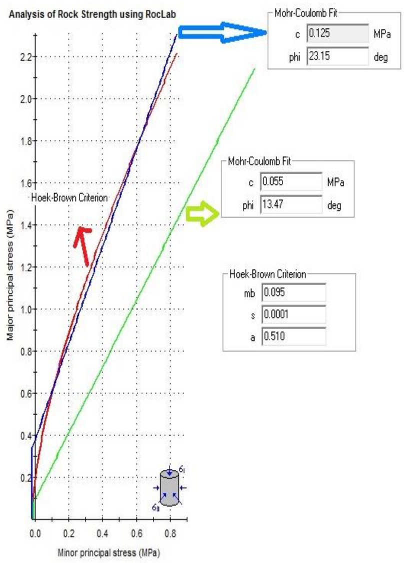


**Fig. 1. Fitting the Mohr-Columb linear failure criterion (blue and green line) to the Hooke and Brown nonlinear failure criterion (red line) for the alluvium unit.**