The Influence of Horizontal Stress on the Failure Mechanism and Slope Stability in Chador-Malu Iron Open Pit Mine

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Abstract

Owing to the development of advanced facilities and mining machinery, depth of open pit mines is ever increasing. Due to this the risk of large-scale slope instability has augmented and hazards control has become complicated. Choosing appropriate slope of open pit mine, both from the stability and financial point of view, is by far the most vital part of design procedures. Moreover, various robust numerical methods have been utilized by engineers to investigate the behaviour of rock mass and to study the stability conditions of slopes.

In this paper, the slope instability of Chador-Malu iron open pit was assessed. This mine is located in central Iran, 120km northeast of Yazd city. The current depth of mine is 50m and designed depth of pit after 30 years is 225m. In this research, several numerical models for 30 years pit have been investigated and using these models the influence of horizontal stress ratio on the failure mechanism have been studied and stability situations were compared. The results show that the tensile and shear failure would expand and the stability of the slope would decrease due to increase in horizontal stress ratio from 0.3 to 2.

1 Introduction

The stability of rock slopes especially stability of mine walls is always of great concern in the field of rock mechanics and mining engineering due to the fact that the excavation of rock slopes may cause instabilities and would interrupt production of the mine. The large-scale slope design of mine requires several parameters and consideration of various details which are complicated and troublesome to achieve.

In open pit mining, the optimum slope design is usually one that maximizes overall slope angles and minimizes the amount of waste stripping. At the same time, it must effectively manage the risk of overall slope instability, and provide for safe and efficient movement of personnel, equipment and materials during mining operations (Wyllie & Mah 2004).

There are various methods to analyze and design slopes. These methods, which have specific advantages and disadvantages inherent in their respective methodologies, include empirical method, probabilistic technique, limit equilibrium and numerical methods. Stead et al. (2001) reviewed limit equilibrium and numerical analysis techniques in detail with respect to their application to rock slope analyses (Eberhardt et al. 2004).

The role of stresses has been traditionally ignored in slope analyses. One particular advantage of numerical models is their ability to include premining initial stress state in stability analyses and to evaluate their importance (Lorig & Varona 2000).

In this study the limit equilibrium models and 2-D finite difference method have been utilized to investigate the stability of final slope of Chador-Malu iron open pit mine after 30 years of operations. The purpose of the analyses were to gain an insight into the effects of horizontal stress state on the deformation mechanism, stress redistribution of the slope and eventually to compare the hazards of future stability problems during the mine life.
2 Chador-Malu iron open pit mine

Chador-Malu mine is situated 120km northeast of Yazd city in Iran. Figure 1 shows the plan view of final pit. According to the initial design, the final pit has a heart like shape with a width of 960m and depth of approximately 225m. Employing drilling information and simple models, the overall slope angle of 54 degrees and bench slope angle of 70 degrees with a 15m bench height were recommended (KaniKavan 2006). Due to complex geology, ground conditions and different geotechnical properties of rock masses, and presence of fault, pit walls might become unstable, therefore its stability needs to be evaluated considering the various geological conditions and the geotechnical properties.

Figure 1. Final pit design of Chador-Malu mine.

3 Geological and geotechnical characterization

Assessment of rock slope failure mechanisms requires an understanding of structural geology, groundwater and climate, rock mass strength and deformability, in situ stress conditions and seismicity (Rose & Hungr 2007). The area of Chador-Malu mine has complex conditions caused by tectonics activity and complicated geology. Five geotechnical domains have been defined based on the geology, lithology, geotechnical boreholes and rock mass conditions. Figure 2 shows various geological units and five geotechnical domains of the pit.

Figure 2. Geological units and geotechnical domains of final pit area.
In order to evaluate the slope stability, one section has been chosen from the southwest sector of the mine. This section of the mine is positioned in the third geotechnical domain. This section is expected to attain the final designed depth sooner than other sections. The third domain is highly fractured. The southwest wall consists of diorites with intercalated undifferentiated Metasomatites and Albitites underlain by more competent unaltered Metasomatites and iron ore located at the toe of the slope (SRK & KaniKavan 2006). Summary of the rock mass properties for the third domain are tabulated in Table 1.

Table 1. Rock mass properties of chosen section

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>C(KPa)</th>
<th>Φ(Deg)</th>
<th>g (KN/m3)</th>
<th>E(MPa)</th>
<th>t(MPa)</th>
<th>ν</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granite</td>
<td>203</td>
<td>56</td>
<td>24</td>
<td>2607</td>
<td>0.019</td>
<td>0.22</td>
</tr>
<tr>
<td>Albitite Metasomatite</td>
<td>120</td>
<td>46</td>
<td>26</td>
<td>2385</td>
<td>0.006</td>
<td>0.25</td>
</tr>
<tr>
<td>Metasomatite</td>
<td>109</td>
<td>43</td>
<td>27</td>
<td>2252</td>
<td>0.006</td>
<td>0.24</td>
</tr>
<tr>
<td>Diorite</td>
<td>148</td>
<td>49</td>
<td>27</td>
<td>2418</td>
<td>0.009</td>
<td>0.25</td>
</tr>
<tr>
<td>Poor Ore</td>
<td>120</td>
<td>46</td>
<td>26</td>
<td>2546</td>
<td>0.006</td>
<td>0.25</td>
</tr>
</tbody>
</table>

*C= cohesive strength, φ= friction angle, g = density, E= modulus of elasticity, t= tensile strength, ν = poison’s ratio.

4 Limit equilibrium method

The limit equilibrium method (LEM) is widely used in the stability analysis of rock slopes (e.g. Duncan 1996, Wang et al. 2004). In the LEM, a sliding surface is assumed to be formed along the weakest layer of shear resistance which may be obtained through a searching routine (Low et al. 1998). The factor of safety, which is used to evaluate the stability of a slope, is defined as the ratio of the resisting force to the sliding force along the sliding surface. A detailed review of equilibrium methods of slope stability analysis is presented by Duncan (1996). Figure 3 shows the geometry of chosen section, different materials and also slope stability analysis of final designed pit slope utilizing the SLIDE software. The safety factor of limit equilibrium model is 1.11.

Figure 3. Calculated safety factor of final pit in the first section.
5 Numerical analysis method

Numerical analysis methods are useful techniques to assess and judge the stability of slopes. In these methods, one can consider deformations, stress redistribution and therefore appraise failure mechanisms and instability hazards or the behavior of slopes. One particular advantage of stress analysis programs such as numerical models is their ability to include pre-mining initial stress states in stability analyses and to evaluate their importance (Wyllie & Mah 2004).

Eberhardt et al. (2004) suggested that most unstable rock slopes undergo some degree of progressive shear plane development, deformation and extensive internal disruption of the slope mass. As a consequence, the factors governing initiation and eventual failure are complex and not easily included in simple static analyses. Both the continuum and discontinuum models can capture certain aspects of progressive shear plane development.

The horizontal stress ratio is a vital parameter for numerical modeling of rock slopes. Stead et al. (2001) demonstrated that the initial in situ stress ratio is an important input parameter that is commonly overlooked, and Chowdhury (1977) suggested that the initial stress state controls the direction in which the failure surface propagates (Eberhardt et al. 2004). Therefore several models were constructed with different horizontal stress ratios (K=0.3, 0.7, 1, 1.3, 1.7, 2) to assess its influence on the failure mechanism of Chador-Malu open pit mine.

Fast Lagrangian Analysis of Continua (FLAC) was employed in order to study the effects of horizontal stress ratio on the slope behaviour. FLAC is a two-dimensional explicit finite difference program for engineering mechanics computation (Itasca 2000). Peter Cundall developed FLAC in 1986. Figure 4 shows the slope stability analysis of final designed pit slope utilizing the factor of safety option by FLAC. The calculated safety factor of finite difference model is 0.99 considering K=0.3.

![Figure 4. Safety factor of 0.99 of final pit employing finite difference code.](image)

Call et al. (2000) suggested that high in situ horizontal stresses is one of the main geological characteristics of large scale regressive slope movements. When designing overall pit slopes, high horizontal stresses related to regional tectonics should be assessed. As mining progresses, high horizontal stresses accentuate the deviatoric stress in the rock mass near the toe of the slope. If the rock mass is not strong enough to withstand the deviatoric stress, it will yield. Higher horizontal stresses will cause more rock- yielding, leading to larger displacements.

Since the in situ stress tests have not been carried out for the mine, and the effects are largely unknown, in current study, different in situ horizontal stress ratios were chosen, the failure mechanism of the slope were studied and the slope instability hazards were evaluated. Several parameters including stress redistribution, yielded elements, plastic zones, maximum shear strain contours, horizontal and vertical displacements have been considered to appraise the effects of horizontal stress on the slope stability.
The first step is to study the growth of plastic zones and yielded elements in both shear and tension so as to recommend the failure surface. Figure 5 shows the developed yielded elements in the slope for the horizontal stress ratios of 0.3 and 2. The differences between these two figures show how the yielded elements extend due to the increase of horizontal stress. Most parts of slope go under plastic state in the second situation.

In Figure 6 the maximum shear strain contour has been compared for both horizontal stress ratios. For K=0.3, the maximum shear strain occurred in the fifth bench of the slope with the maximum of approximately $1 \times 10^{-2}$ and for K=2, the maximum shear strains calculated were approximately $2 \times 10^{-2}$ in the top five benches and approximately $4 \times 10^{-2}$ in the toe of the slope.

![Figure 5. Plasticity indicator and yielded elements in shear and tension for a) K=0.3 and b) K=2.](image)

The locations of the failure surface in the numerical analysis models were different from that in the limit equilibrium model. In FLAC models the failure surface for both cases (K=0.3, 2) pass through the toe of fifth bench but the one for Slide model (which do not consider the in situ stresses) passes through the toe of the sixth bench. The same result has been reported by Sjoberg (2000). He reported that in his analyses, the location of the failure surface in the FLAC models differed substantially from that in the limit equilibrium analyses. In his research, it has been mentioned that the situation is partly due to grid discretization effect and partly an effect of high confining stress at the slope toe, which “pushes” the failure surface slightly above the toe. Figure 7 shows the time history of vertical displacement in several points of the slope for K=0.3 and 2.

The vertical displacement for K=0.3 of the first bench (green line) increases in the preliminary steps then decreases dramatically. Vertical displacement in the fifth bench increases noticeably at first and would remain approximately the same in the next steps. For K=2 it is apparent that not only the vertical displacement values are higher than the first case but also, the vertical displacements change in different fashion.

In order to study the stress redistribution of the slope, the vertical stress, shear stress, major and minor principal stresses have been evaluated. Figure 8 shows the vertical stress contours redistribution in both situations. One can compare the vertical stress redistribution considering the increase in the horizontal stress ratio from 0.3 to 2.
In the first case the major principal stress reaches to 5MPa while the minor principal stress is 1.5MPa. Redistributed stresses near the excavated slope drop to zero. The direction of the major principal stress becomes almost parallel to the excavated slope surface, while the direction of the minor principal stress is perpendicular to the cut surface. For $k=2$ the major principal stress reaches to 13MPa while the minor principal stress is 4MPa.

Figure 9 demonstrates the redistributed shear stress after excavation for both cases ($K=0.3$, 2). The maximum shear stress was calculated as 0.8MPa for $K=0.3$ and the concentration distributed along the benches of the slope but for $K=2$ it has been calculated as 4.6MPa and the concentration distributed in the toe of the slope.

Tensile and shear failure mechanisms of the slope have been assessed by increasing the horizontal stress ratio. Figure 10 show elements with zero tension and the maximum shear strain contours. After excavation the tension value reduces to zero in some parts, suggesting a tensile failure.
For $K=0.3$, the shear failure covers five benches and the maximum shear strain is calculated as $2 \times 10^{-2}$. The tensile failures occurred behind the first bench and also at toe of the slope (Fig. 10). For $K=0.7$, tensile failure has been decreased in the toe but it has been extended in the upslope. Shear strain contours have been expanded in the toe but the maximum value was reduced to $1.25 \times 10^{-2}$ (Fig. 10b). By increasing the horizontal stress ratios, the tensile failure continues up the slope and behind the first bench and shear strain contours would expand as well. The maximum value of shear strain is calculated as $1.75 \times 10^{-2}$ for $K=1$ and $2 \times 10^{-2}$ for $K=1.3$. This value increased to $3.5 \times 10^{-2}$ and $5 \times 10^{-2}$ for the next cases ($K=1.7, 2$).

![Figure 10a](image1.png)  
**Figure 10a.** Maximum shear strain contours and elements with zero tension of slope for $K=0.3$.

![Figure 10b](image2.png)  
**Figure 10b.** Maximum shear strain contours and elements with zero tension of slope for $K=0.7$.

![Figure 10c](image3.png)  
**Figure 10c.** Maximum shear strain contours and elements with zero tension of slope for $K=1$. 
6 Failure initiation and propagation

In limit equilibrium techniques, the whole slope will be considered as a rigid body which will move together but in numerical models failure of slopes could be studied in detail. Furthermore, the limit equilibrium solution only identifies the onset of failure, whereas the FLAC solution includes the effect of stress redistribution and progressive failure after movement has been initiated (Wyllie & Mah 2004). Sjoberg (2000) employed the numerical model to study the failure development of high slope. It has been reported that failure occurred in several phases and significant displacements occurred before the failure surfaces could develop fully.
Zavodni (2000) mentioned that all natural and human-made rock slopes deform with time in response to excavation. Moreover, Stead et al. (2006) pointed out that the traditional engineering approach is to analyze either the failure initiation mechanism or the transport/deposition stage. They suggest, however, that if true risk is to be ascertained then the deformation characteristics prior to failure and the post failure movement must be linked (Stroth 2006).

In this research, onset and development of the failure mechanism in Chador-Malu mine have been studied in details. Considering the numerical results, the displacements of various points in conjunction with the failure mechanisms of slope have been chosen to explore the slope stability circumstances. Solving the FLAC models, various cycles were reviewed to understand the behaviour of slope due to excavation.

All slopes are expected to experience a period of initial response as a result of elastic rebound, relaxation and/or dilation of the rock mass due to the changes in stress induced by the excavation (Zavodni 2000). This initial deformation is expected to occur without the development of a defined failure surface or failure mechanism. Martin (1993) reports on the initial response measurements of three open pit mines, which showed that total displacement, varied from 150mm in a strong massive rock mass at Palabora in South Africa to more than 500mm in highly fractured and altered rock at the Goldstrike Mine in Nevada. The initial deformation may not significantly affect rock-mass strength or the stability of the slope (Zavodni 2000).

Regarding the numerical model, when the slope first excavated to a final level and after only 1000 cycles, yielding started to take place and tension became zero in some benches (it is not simulating a true time dependent behaviour but to assess the stability situations with cycle increase). The magnitude and direction of total displacement for K=0.3 in some points of slope have shown in Fig. 11. Maximum shear strains calculated as $2 \times 10^{-2}$ after 20000 cycles and a band of yielded elements in shear has formed. Shear strain accumulation starts at the fifth bench and developed upward and reached the crest, a failure surface has formed, together with increase in total displacements. In addition tension value behind the crest of slope became zero. By continuing the cycles, maximum shear strain increases to $1.2 \times 10^{-1}$ and develops fully. Furthermore tension value of the second and third benches partly became zero. In the first steps, the maximum total displacements occurred in the toe of the slope but by increasing the cycles and owing to the growth of the failure surface the place of maximum total displacements changed from the toe to the crest of the slope (Fig. 11 a, b and c). The directions of total displacements have changed. By increasing cycle, FLAC models continued to deform more but this is not an issue in this article. The difference between various stresses situations are main matter in this research.

The same procedure was repeated for K=2 so as to realize the behaviour of the slope when the horizontal stress state is high. After 1000 cycles the final excavated slope showed 0.2m heave in the toe of the slope. Yielded elements have formed and shear strain accumulation started at the toe. In addition, tension became zero in all the benches. The amounts of zero tension elements in this state are enormously higher than the first case. After 20000 cycles, maximum shear strains calculated as $5 \times 10^{-2}$ and a band of yielded elements in shear has formed in the upslope. Shear strain accumulation (failure surface) started at the fifth bench and developed upward and reached the crest. Zero tension covered behind the crest of the slope. Maximum total displacements in the fifth bench are higher than displacements of the toe. By continuing the cycles, it can be seen that the maximum total displacements will reach to 0.55m in the fifth bench and 0.5m in the crest but they have same direction in different points. Maximum shear strain increased to $9 \times 10^{-2}$ and developed fully (Fig. 12).

Based on the geological structure and the stress state in the rock mass, certain failure modes appear to be more likely than others in large scale slopes (Sjoberg 1996). The most important failure modes to consider are rotational shear failures and the secondary failure modes associated with these. Large scale toppling failures have also recently been observed in high pit slopes (Sjoberg 1996). Outward and downward movement at the crest and bulging at the toe would indicate a plane or circular failure. Slope failure would be indicated by the presence of tension cracks at, or near the crest of the slope. The development of such cracks is evidence that the movement of the slope has exceeded the elastic limit of the rock mass. However, it is possible that mining can safely continue under these conditions with the implementation of a monitoring system (Wyllie & Mah 2004).
Figure 11  Vectors of total displacements, maximum shear strain contours and zero tension elements for K=0.3.

Figure 12  Vectors of total displacements, maximum shear strain contours and zero tension elements for K=2.

Regarding the results of numerical models (Fig. 11) the shear failures are most probable mode of failure for k=0.3 and the hazards of instability should be focused in the upslope of the mine. On the other hand, Figure 12 suggests that the possibility risks of instability for k=2 consist of both shear failure and circular deep-seated failure.
Zavodni & Broadbent (1978) concluded that almost all large scale failures occurred gradually. Serious slope instabilities were almost always accompanied by the gradual development of tension cracks behind the slope crest and measurable displacements (Sjoberg 1996). A diligent monitoring program is required for the mine to have an insight into the displacement rate and therefore to understand the risks of the instability.

7 Results

The potential failure zones of slope of Chador-Malu iron open pit mine for the final pit after 30 years of operations have been investigated. Various numerical models were constructed with different horizontal stress ratios to assess the influence of this parameter on the failure mechanism of the slope. The risk of large-scale slope instability has augmented and hazards control has become complicated as a result of increase in the horizontal stress ratio. For K=0.3, the maximum shear strain occurred in the upslope and the tensile failure occurred in few benches. By increasing the horizontal stress ratio the shear strain contour expanded and tensile failure continues up the slope and also behind the crest. Yielded zone in the toe of the slope as a result of high horizontal stress ratios cause slope failures. Moreover the tensioned elements in the crest and other parts of the slope suggest tension cracks and tensile failure mechanism. For low horizontal stresses, numerical results suggest relatively low risks of instabilities.

Stability analysis of final slope utilizing the limit equilibrium method showed safety factor of 1.11 while the finite difference model for K=0.3 demonstrated lower safety factor (SF=0.99). The locations of the failure surface in the numerical analysis were slightly different from that in the limit equilibrium model. Concerning the results of the numerical models, the shear failures are most probable mode of failure for low stress ratios and the hazards of instability should be focused by a great amount in the upslope of the mine. On the other hand, the possibility risks of large instability for high stress ratios consist of both shear failure in the upslope and circular failure from the toe. Numerical results illustrated that displacement is the best indicator of the failure during the excavation processes. It is obviously important to organize monitoring system for the mine.

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9 References


