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Slope stability at Chador Malu and optimization of the monitoring systems

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ABSTRACT: Choosing the accurate slope of open pit, both from the stability and financial point of view, is by far the most vital part of mines design procedure. On the other hand, monitoring systems could be utilized so as to understand the behavior of rock mass. In the current study, slope stability of chador malu open pit was assessed. In this research, various models for both current pit with 50m depth and also final pit have been investigated and the appropriate monitoring systems were recommended.

1. INTRODUCTION

The stability of natural rock slopes or mine walls is always of great concern in the field of rock engineering. The excavation of jointed rock slopes may cause instabilities and interrupt constructions.

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 [1].

There are various methods to analyze and design slopes. These methods include empirical, probabilistic, limit equilibrium and numerical methods. On the other hand instrumentation and monitoring systems could be employed by engineers not only to evaluate real situations of slopes also to inform personnel before occurrence of any instability.

Limit equilibrium and numerical analysis techniques have specific advantages and disadvantages inherent in their respective methodologies. Stead et al. [2] reviewed these methods in detail with respect to their application to rock slope analyses [3].

In this study the limit equilibrium and 2-D distinct element methods have been utilized to design slope of Chador Malu iron open pit mine for present pit with 50m depth and besides for the final pit after 30 years of operations. The purposes of analyses were to gain an insight into the deformation mechanism and stress redistribution of the slope and to investigate the potential failure zones of 30 years pit and to recommend monitoring systems for the mine.

2. CHADOR MALU IRON OPEN PIT MINE

Chador Malu iron open pit mine is situated in 120 km northeast of Yazd city in central Iran. This mine is formed of north and south anomalies. Fig 1 shows the plan view of final pit. According to the initial design, the final pit is like a heart with a width of 960m and depth of approximately 225m. Employing drilling information and simple models, the overall slope angle of 54° and bench slope angle of 70° with a 15m bench height were recommended [4]. Due to complex geology and ground conditions and different geotechnical properties of rock masses and considering the presence of fault and sheared zones, pit walls might have stability problems with the same angle of slope and similar circumstances. Therefore the slope of mine should be evaluated considering various geological conditions and geotechnical properties so as to design wall slopes.

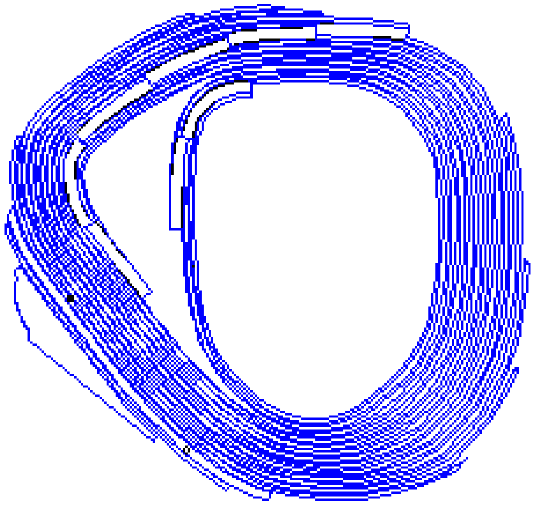

Fig. 1. Final pit design of Chador Malu mine.

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 [5].

The area of Chador Malu mine has a complex and complicated conditions due to tectonics activity and complex geology situations. In order to carry out slope stability analyses, the field geology, structural study, geotechnical borehole drilling, various rock mechanics tests, surface and underground water study have been conducted. Five geotechnical domains have been defined based on the structural, lithology, RMR conditions and the orientations of the pit walls. Fig 2 shows various geological units and five geotechnical domains of pit.

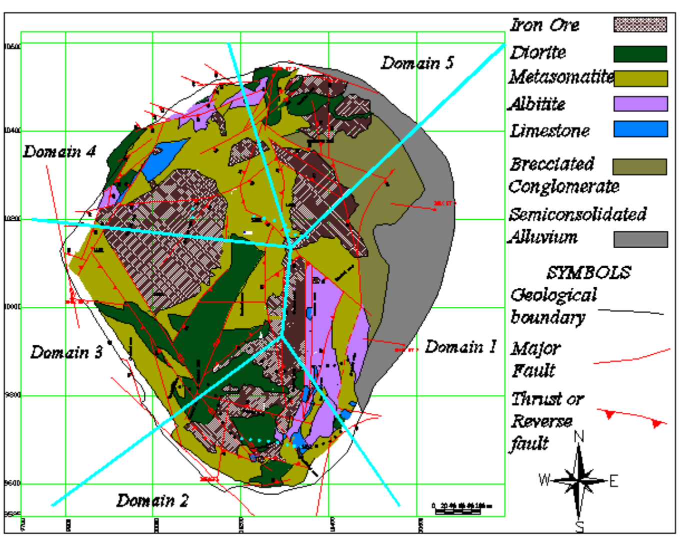

Fig. 2. Geological units and geotechnical domains of final pit.

Two main joint sets J1 (dip direction of 20, dip of 68) and J2 (dip direction of 95, dip of 80) were identified and the rock mass conditions of the pit have been defined by Rock Mass Ratings (RMR) method. The rock mass quality is generally poor to fair and varies from very poor (alluvium and fault material) to fair (Iron ore, granitic dykes and diorites) [6].

In order to evaluate the slope stability of the mine, two sections have been chosen from the southwest sector of the mine. These sections of the mine which are positioned in the third geotechnical domain, would reach to the final pit wall sooner than other sections. The southwest wall consists of diorites with intercalated undifferentiated metasomatites and albitites underlain by more competent unaltered metasomatite and iron ore located at the toe of the slope [6]. Summary of the rock mass properties for the third domain are tabulated in Tables 1 and 2.

Table 1. Rock mass properties of section I

Rock Type	C (KPa)	Φ (Deg)	ρ (KN/m^3)	E (MPa)	υ
Granite	203	56	24	1454	0.22
Albitite Metasomatite	64	34	27	504	0.25
Metasomatite	131	47	27	659	0.26
Diorite	148	49	27	862	0.25
Poor Ore	120	46	26	709	0.25

Table 2. Rock mass properties of section k

Rock Type	C (KPa)	φ (Deg)	g (KN/m^3)	E (MPa)	υ
Granite	203	56	24	1454	0.22
Diorite Albitite	120	46	26	709	0.25
Metasomatite	109	43	27	542	0.24
Diorite	148	49	27	862	0.25

* C= cohesive strength, φ= friction angle, g = density, E= modulus of elasticity, ν = poison's ratio.

Third domain is highly fractured and the average mining adjusted rock mass ratings (MRMR) for the southwest wall is between 28 -39 that indicate a very poor to poor quality rock mass.

4. LIMIT EQUILIBRIUM METHOD

In the current research, static analyses of Chador Malu iron open pit mine have been carried out using both limit equilibrium and distinct element methods. Two sections (sections I and K) from southwest sector of the mine have been chosen since this part of the mine would reach to the final pit sooner.

The limit equilibrium method (LEM) is widely used in the stability analysis of rock slopes [7, 8]. 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 [9]. 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 [7]. Fig 3-6 show the chosen sections, different materials in these sections and also slope stability analyses of current and final designed pit slopes employing the Slide software. The safety factors of limit equilibrium models in section I are 1.31 and 1.12 for current and final pit respectively. These parameters for the section K are 1.52 and 1.37.

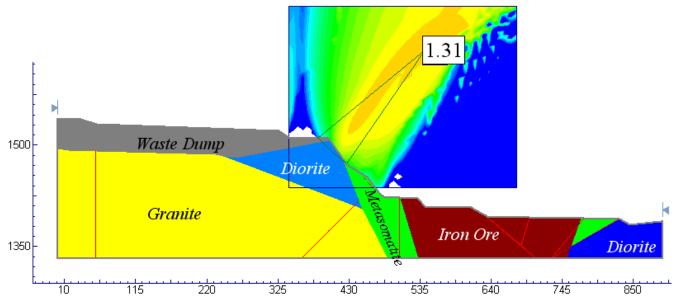

Fig. 3. Calculated safety factor of current pit in the first section.

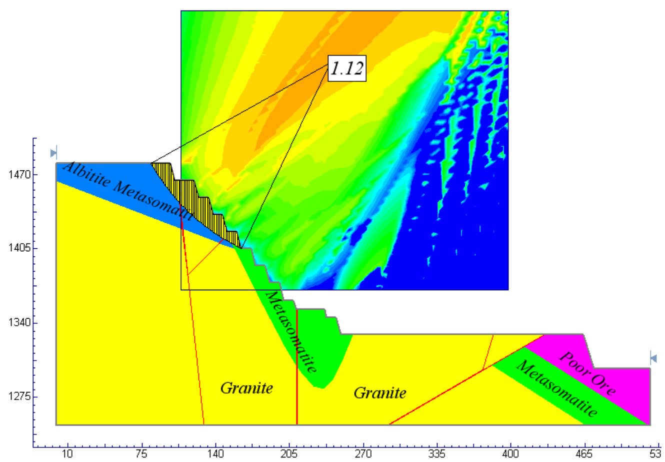

Fig. 4. Calculated safety factor of final pit in the first section.

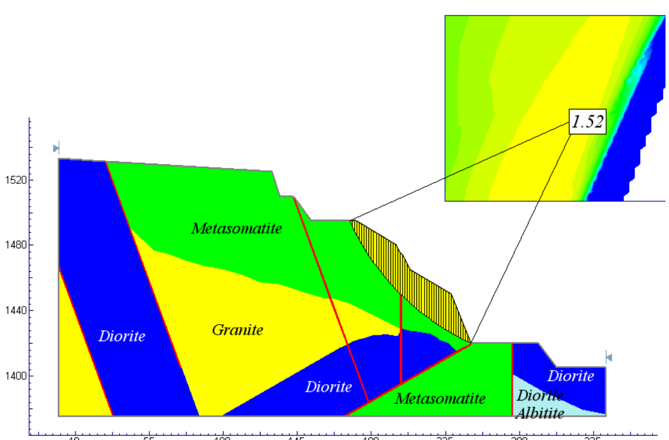

Fig. 5. Calculated safety factor of current pit for the second section.

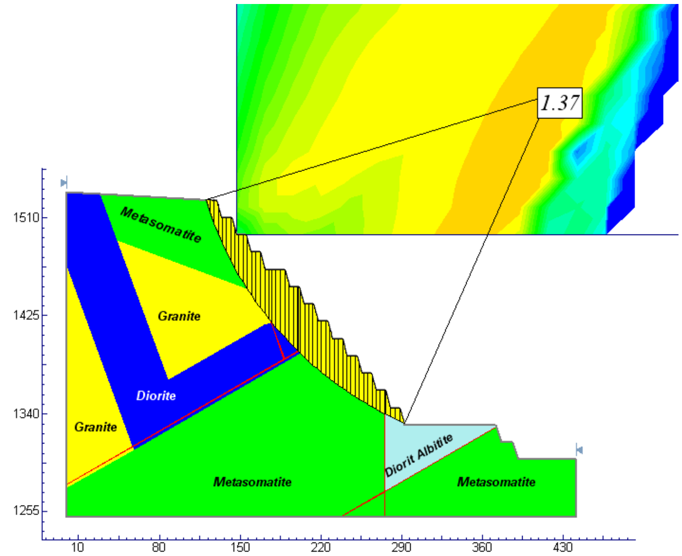

Fig. 6. Calculated safety factor of final pit in the second section.

5. NUMERICAL ANALYSIS METHOD

Eberhardt et al. [3] 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 show that their formulations can capture certain aspects of progressive shear plane development.

Numerical analysis methods such as the finite element method, finite difference method and distinct element method are efficient techniques to assess and judge the stability situation of slopes. In these methods, one can consider deformations, stress redistribution and then appraise critical load, failure zone and failure types or the behavior of slopes.

The distinct element method (DEM) was first proposed by Cundall [10] to study the movements of granular assemblies.

In this research, two-dimensional Universal Distinct Element Code (UDEC) was used so as to assess the mentioned purposes and in addition to compare the results of LEM with DEM.

The LEM assumes only one sliding face, whereas the DEM can handle two or three sets of joints and the sliding may occur along any joint. The set of joints, which have the weakest strength or the most dangerous orientation, is critical to the slope stability, but other sets of joints can reduce the integrity of the jointed slope. If a jointed rock slope has more than one failure mode, the LEM cannot describe all multiple failure modes while the DEM can [11].

5.1. First section (I)

The dimensions of UDEC model for Chador Malu iron open pit mine are 230m by 525m in the first section. In this section fourteen separate sequences of excavation have been considered. Fig 7 shows the UDEC model, slope geometry and fault systems.

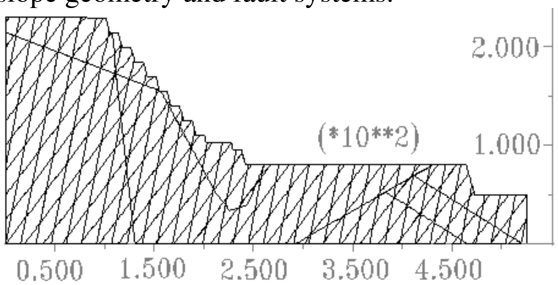

Fig. 7. Distinct element model for the first section.

Following Figures show the results of slope stability analyses of final pit slope employing the UDEC. The maximum displacements reach to 1.6m in the final step (Fig 8).

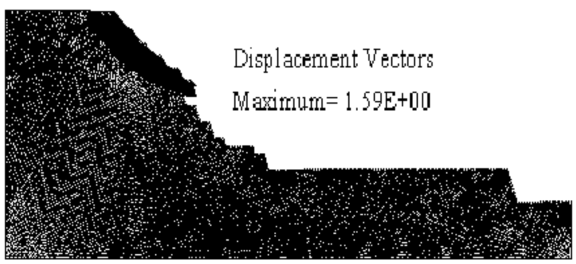

Fig. 8. Maximum displacement vectors in the first section.

Knowledge of the stress state in a slope is essential in order to understand the mechanics of slope behavior. The stresses acting upon a structure in comparison to the strength of the structure govern the stability of that structure [12].

In the current study, vertical stresses have been calculated from the overburden weight and the stress ratio was determined 0.3. High stress distributions have been seen in the bottom of the slope. The major principal stresses were between 1~7Mpa (Fig 9).

In numerical modeling, some parameters including the maximum shear strain, plastic zones and yielded elements have been evaluating so as to assess the possible surface of failure. Fig 10 shows the shear strains, yielded zones and tensile failure in this section.

Regarding the results of numerical model a surface failure can be seen. Moreover the tensile failure elements in numerical models suggest tension cracks. Safety factor of the model by DEM analyses was less than one recommending the slope stability problem.

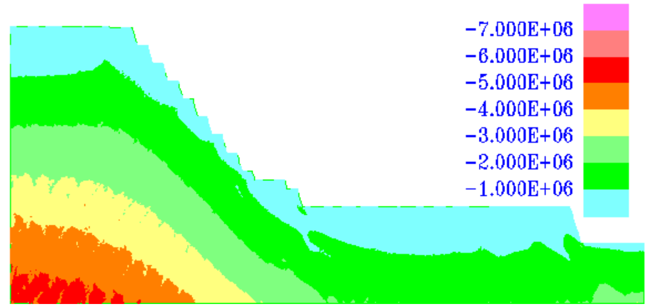

Fig. 9. Major principal stress in the first section.

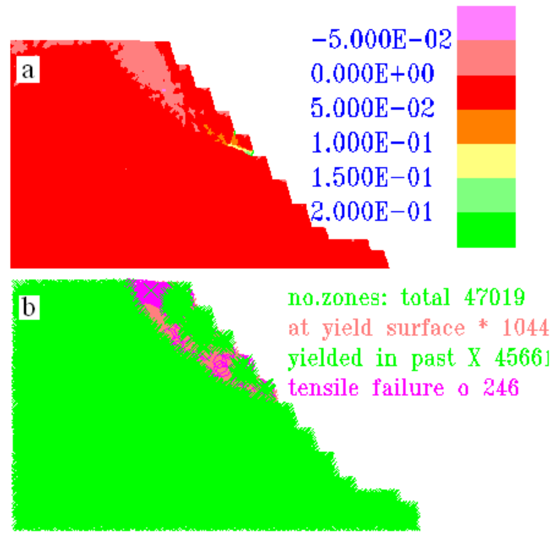

Fig. 10. a). XY strain contours and b). plastic indicators in the first section.

5.2. Second section (K)

In the second section, seventeen sequences of excavation have been chosen and dimensions of model were 283m by 441m. Fig 11 shows the slope geometry and fault systems which has been analyzed by DEM.

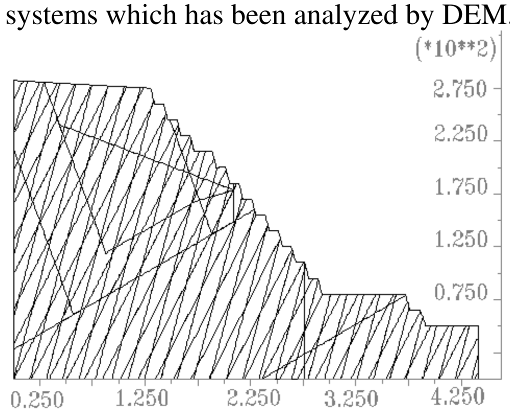

Fig. 11. Distinct element model for the second section.

In the second section, major principal stresses were calculated between 1-9Mpa (Fig 12). Fig 13 shows the shear strains in this section which encompasses approximately all benches and suggest failure beginning from the toe of the slope. Yielded zones or tensile failure did not occur in this section considering DEM analyses. The maximum displacements reach to 12cm in the final step. Fig 14 shows the vertical displacement contours.

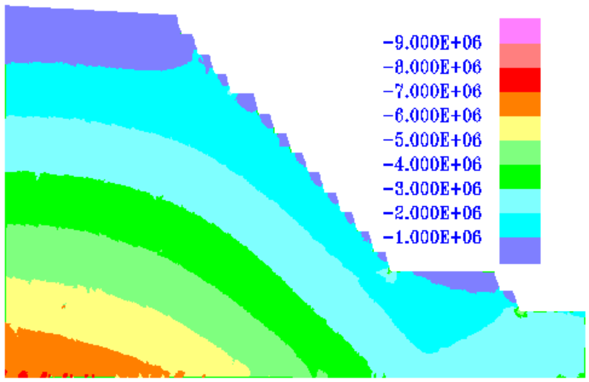

Fig. 12. Major principal stresses in the second section.

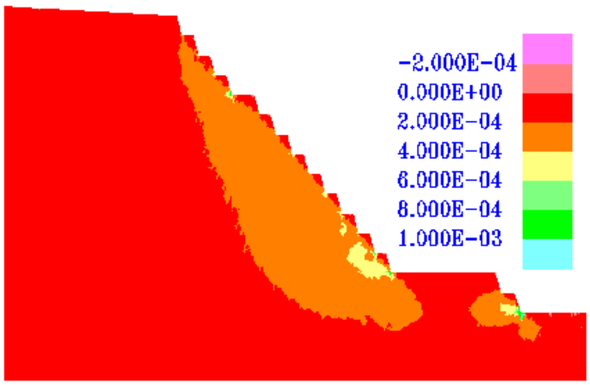

Fig. 13. XY strain contours in the first section.

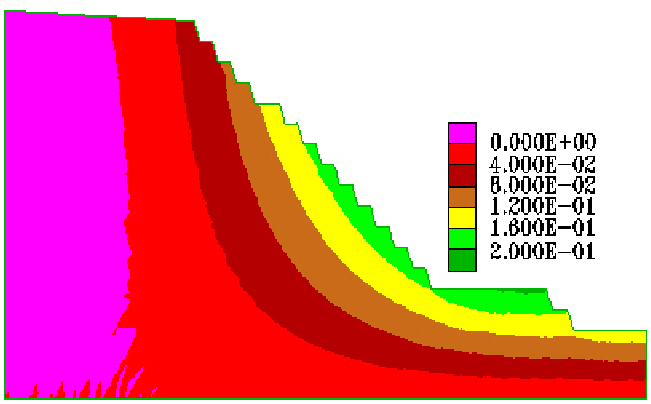

Fig. 14. Y displacement contours in the second section.

6. LEM VERSUS DEM

Li et al. [11] compared the results of LEM with the results of DEM. They suggested that the factor of safety obtained with the distinct element can be lower than that obtained with the LEM mainly due to the consideration of faults both along the sliding surface and also other directions.

In this step, the failure surfaces of LEM were compared with the DEM results. Fig 15 shows the surface failure of both methods for the first section. Based on this figure, it is clear that DEM could find bigger surface failure. The place of surface failure behind the crest using numerical model is in greater distance than from LEM. This distance is approximately 32m behind the crest whereas LEM shows a surface which is approximately 17m behind the crest.

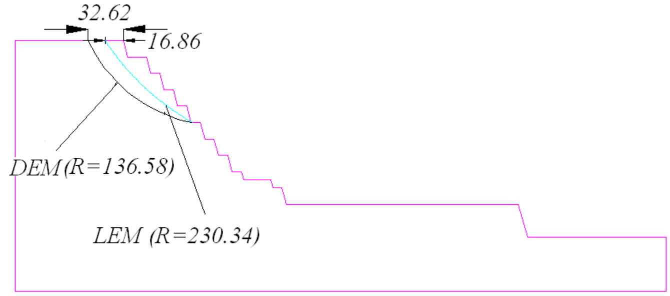

Fig. 15. Suggested failure surfaces of LEM and DEM analyses in the first section.

Fig 16 shows the surface failures in the second section. Regarding DEM results (plastic indicators and yielded zones) it was seen that no failure surface would occur but based strain contours, surface failure with weak probability could be suggested. Despite the fact that LEM shows a surface which is approximately 14m behind the crest but the surface of DEM is not deep seated and the failure would begin from the toe of the slope. Safety factor of the model by DEM analyses was remarkably higher than one recommending the stable slope in this section.

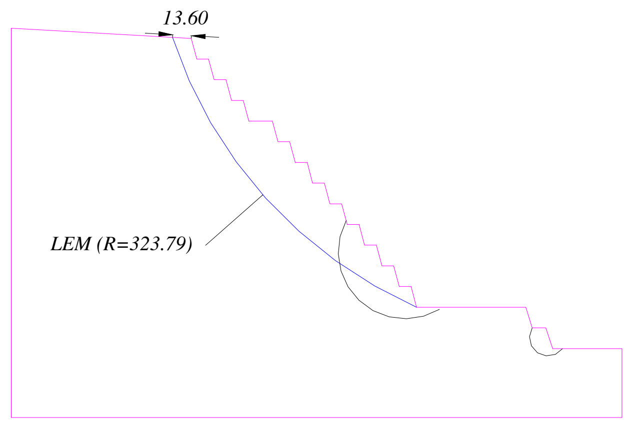

Fig. 16. Suggested failure surfaces of LEM and DEM analyses in the second section.

7. MODES OF FAILURE

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 [12].

The most important failure modes to consider in large scale slopes 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. Plane failure or wedge failure could develop if major faults of very large dimensions are present in the rock mass. It is difficult to determine in advance what modes of failure that could occur in a certain open pit and how discontinuities interact to form a failure surface [12].

Outward and downward movement at the crest and bulging at the toe would indicate a plane or circular failure, whereas horizontal movement at the crest only would be more indicative of a toppling 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 [1].

In the first section, considering the shear strain contours (Fig 10.a) which shows extremely high values, rotational shear failure in the first five benches could be the most likely failure mode. This recommends high possibility of circular failure.

In the second section, the shear strain contours (Fig 13) have low values and it is clearly obvious that rotational shear failure have a weak effect on the most part of the section. Regarding this figure, one can suggest that surface failure would starts from the toe of the slope with low possibility.

Zavodni and Broadbent [13] 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. They defined two failure stages for large scale failure: a regressive stage and a progressive stage. A better definition of these two phenomena would be stable and unstable failure [12].

Based on Zavodni [14] a regressive failure is one that shows short-term decelerating displacement cycles if disturbing events external to the slope, such as blasting or water pressure, are removed. Conversely, a progressive failure is one that displaces at an increasing rate, with the increase in rate often being algebraic to the point of collapse, unless stabilization measures are implemented. Operations can be continued below slopes experiencing regressive movement, but it is necessary that the mining be conducted for short periods with frequent pullbacks, with care being taken to identify the transition to a progressive failure [1].

With continuing excavation, regressive slope displacements may occur in a cyclical accelerating/ decelerating fashion. As strain levels increase, strain softening may lead to plastic (non-recoverable) deformation and progressive failure development. Displacement rate (velocity) is commonly considered the best indicator of the failure process [5].

It is obviously important to recognize the onset of progressive failure, which will require a diligent monitoring program and careful analysis of the results [1].

Taking the extremely high values of displacement vectors (Fig 8), the amount of plastic and yielded elements and also the occurrence of tensile failures (Fig 10.b) into consideration, the progressive failure could happen in the first section.

In the second section, regarding maximum displacements (12cm) and also vertical displacements (Fig 14), no yielded elements or tensile failures have been seen which could suggest a regressive failure.

8. MONITORING

Monitoring is used in mines in order to anticipate possible acceleration or failure of a moving slope mass. Available instruments include precise survey stations and prisms, wire and rod extensometers, inclinometers, tiltmeters, Global Positioning System (GPS) devices and geophones to record the intensity of ground noise [5]. The objectives of a slope monitoring program are to maintain safe operational practices, provide advance notice of instability and provide additional geotechnical information regarding slope behavior [15, 16].

Call et al. [17] proposed that mining could be continued despite large scale displacement if the failure is stable, the failure mechanism well-defined, and monitoring is performed continuously [12]. There are several well-documented cases of slope monitoring at open pits where mining continued for several months below the moving slope [1].

A large rock slide rarely moves as a completely rigid block. Fragments of various sizes, often situated around the perimeter of a central coherent or semi-coherent mass, can be subject to localized movements such as minor slides or topples, bulging or buckling of beds, rotations and possibly abrupt, local deformation adjustments caused by crack opening. The result can be random, chaotic displacement that is of itself unsuitable for detection of any large-scale trends. It is not unusual for local movements to contradict the movement trend of the main slide body, both in space and time. On-going re-assessment of the failure mechanism and its relation to the geometry and structure of the slope is important and this also helps to optimize the placement of instruments in view to avoiding confusion caused by local movements [5].

As Vamosi and Berube [18] and ACG [19] reported, the most suitable monitoring method on large slopes is surveying which employ electronic distance measurement (EDM) equipment. This method could be utilized where access to the slope is hazardous or there is a need to make frequent and precise measurements [1].

Since tension cracks are an almost universal feature of slope movement, crack width measurements are often a reliable and inexpensive means of monitoring movement [1].

In the southwest sector of the Chador Malu iron open pit mine, EDM surveying or laser imaging should be carried out. With reference to the numerical results, measuring crack widths with steel pins, wire extensometer or vibrating wire strain gauges is completely useful and practical only in the first section.

Sub-surface measurement of slope movement is often a useful component of a monitoring program in order to provide a more complete picture of the slope behavior. The main purpose of these measurements is to locate the slide surface or surfaces, and monitor the rate of movement [1].

In designing monitoring system and instruments for the Chador Malu iron open pit, main emphasis was on faults and bedding plane and the results of models. The purposes of instrumentation were to control groundwater situations near to fault, displacements measurement and the rate of movement in order to explore the failure mechanism to recognize the start of progressive failure.

In order to reach the mentioned targets various instruments including inclinometer, borehole extensometer, open standpipe pizometer and tape extensometer could be employed. Fig. 16 shows the suggested places of instruments for the first section. In this section, following instruments have been designed:

- 50m inclinometer with a bottom elevation of 1425
- 60m open standpipe pizometer
- 9m borehole extensometer with a angle of 55° and two points of measurements
- tape extensometer

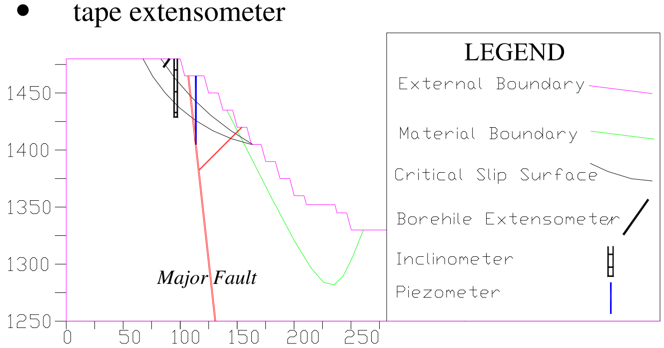

Fig. 16. Suggested places of instruments in the first section.

Fig. 17 shows the suggested places of instruments for the second section. In this section, following instruments have been designed:

- 50m inclinometer
- 50m open standpipe pizometer
- 15m borehole extensometer with a angle of 21° and three points of measurements

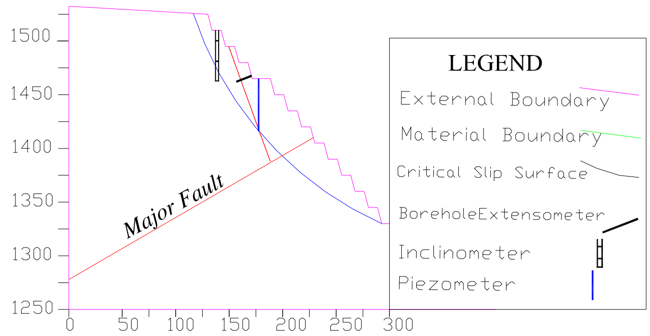

Fig. 17. Suggested place of instrumentation in the second section.

9. RESULTS

Slope stability of Chador Malu iron open pit mine for present pit with 50m depth and besides for the final pit after 30 years have been carried out so as to evaluate the behavior of slope, type of failure and designing the monitoring system.

Taking the extremely high values of displacement, yielded elements and also the occurrence of tensile failures into consideration, the progressive failure is possible in the first section but implementation of a creep model to understand the complete detailed slope behavior is highly recommendable. But in the second section, numerical results suggest a regressive failure.

In the southwest part of the Chador Malu iron open pit mine, surveying should be carried out and regarding the numerical results, measuring crack widths is completely useful only in the first section owing to the fact tension cracks would develop in this part.

So as to monitor the rate of movement, investigating the failure mechanism and to recognize the start of the progressive failure in Chador Malu iron open pit mine, surface and subsurface instrumentation systems were recommended.

10. ACKNOWLEDGEMENTS

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REFERENCES

1. Wyllie, D.C. and C.W. Mah. 2004. *Rock Slope Engineering*. 4th ed. Spon Press. Taylor & Francis Group.
2. Stead, D., E. Eberhardt, J. Coggan, and B. Benko. 2001. Advanced numerical techniques in rock slope stability analysis—applications and limitations. In

Proceedings of the International Conference on Landslides—Causes, Impacts and Countermeasures, Davos. 615–24.

3. Eberhardt, E., D. Stead, and J.S. Coggan. 2004. Numerical analysis of initiation and progressive failure in natural rock slopes—the 1991 Randa rockslide. *International Journal of Rock Mechanics & Mining Sciences.* 41: 69–87.
4. KaniKavan shargh Engineering Co. 2006. First Phase Slope Design Studies of Chador Malu.
5. Rose, N.D, and O. Hungr. 2007. Forecasting potential rock slope failure in open pit mines using the inverse-velocity method. *International Journal of Rock Mechanics & Mining Sciences.* 44 () 308–320.
6. SRK Consulting, KaniKavan shargh Engineering Co. 2006. Chador Malu Phase II Slope Design Studies – Stage5. Design of Mines Slopes
7. Duncan, J.M. 1996. State of the art: Limit equilibrium and finite element analysis of slopes. *Journal of Geotechnical Engineering*. ASCE. 122: 7,577-596.
8. Wang, Y.J., J.H. Yin., Z. Chen, and C.F. Lee. 2004. Analysis of wedge stability using different methods. *Rock Mech. Rock Engng*. 37: 2,127–150.
9. Low, B.K., R.B. Gilbert, and S.G. Wright. 1998. Slope reliability analysis using generalized method of slices. *J. Geotechn. Geoenvironm. Eng*. 124: 4,350–4,362.
10. Cundall, P.A. 1971. A computer model for simulating progressive large-scale movements in blocky rock systems. In *Proceedings of the Symposium. International Society for Rock Mechanics, Nancy*.
11. Li, S.H., J.G. Wang, B.S. Liu, and D.P. Dong. 2007. Analysis of Critical Excavation Depth for a Jointed Rock Slope Using a Face-to-Face Discrete Element Method. *Rock Mech. Rock Engng*. 40: 4,331–348.
12. Sjoberg, J. 1996. *Large scale slope stability in open pit mining - a review*. Technical report. Division of rock mechanics. Lulea University of technology.
13. Zavodni, Z.M, and C.D. Broadbent. 1978. Slope failure kinematics. In *Proceedings of the 19th US Symposium on rock mechanics, stateline, Nevada*, 86-94.
14. Zavodni, Z.M. 2000. Time-dependent movements of open pit slopes. In *Slope Stability in Surface Mining*, ed. W.A. Hustrulid, M.K. Mccarter, D.J.A. Van Zyl. 81–87.
15. Call, R.D. 1982. Monitoring pit slope behavior. In *Proceedings of the third International conference on stability in surface mining, Vancouver, 1-3 June 1981,* Society of Mining Engineers, A.I.M.E. 229-248. New York:
16. Call, R.D., and J.P. savely. 1990. Open pit rock mechanics. In *Surface Mining*, ed. B.A. Kennedy, 860-882.
17. Call, R.D., T.M. Ryan, and R.C. Barkely. 1993. Geotechnical Analyses for Open Pit mining in areas of large-scale slope instability. In *Proceedings of the Innovative Mine Design for the 21st century, Kingston, 23-26 August 1993*, 45-56. Rotterdam: Balkema.
18. Vamosi, S., and M. Berube. 1987. Is Parliament Hill moving? Canadian Dept. Energy Mines and Resources, *Geos Magazine.* 4, 18–22.
19. Australian Center for Geomechanics (ACG). 1998. *Integrated Monitoring Systems for Open Pit Wall Deformation*. MERIWA Project M236, Report ACG: 1005–98.