1£th International Mining Congress and Exhibition of Turkey-IMCET 2003, if 2003, ISBN 975-395-605-3

Stability Analysis of the Second Site of the Sarcheshmeh Heap Leaching

S. Karimi Nasab, M. A. Nabizadeh & M. Ranjbar
Mining Engineering Department, Shahid Bcılıonar University, Kermcm, Iran

B. Shaydaie
Hydromelallurgical Division, National Iranian Copper Industries Company, Kcrman, Iran

M. R. Mollaei Fard
Mining Division, National Iranian Copper Industries Company, Kcrman, Iran

ABSTRACT: The second site of the Sarcheshmeh valley leach facility extends over 300,000 irr which may eventually reach heights of nearly 90 m, is situated in the west side of the mine. This paper discusses the stability analysis of heap leaching structure. Hence, it is important to be able to assess the bonding properties of the interface between the geomembrane and inclined surface. The bonding strength results from friction, cohesive forces or combination of the two. Thus the interface friction between texture and smooth HDPE geomembranes and granular soil of cushion layer was investigated by a series of shear box tests. A conventional limiting equilibrium technique based on Carter and Janbu methods was employed using STABL and CLARA softwares. The results of analysis indicated that some profiles along the valley might not be stable due to PLS (Pregnant Leach Solution) level in the heap and blast vibrations.

I INTRODUCTION

Slope stability is an extremely important consideration in the design and construction of heap leach piles. The task of the designer therefore is to minimize the ongoing construction costs of this kind of structure by maximizing the volume of material that the facilities can contain without unduly risking failure (East & Valera 2000).

The second site of the Sarcheshmeh heap leaching is a lined valley leach facility that extend over 300000 m" on a steep valley situated in the western side of the mine which contain 6 million m of low grade copper ore.

Liner systems placed to contain the Pregnant Leach Solution (PLS) often introduce low interface friction angles along the base of the ore heap. Certain geometric shapes associated with valley fills can produce ore heaps for which the heap slopes are stables but the entire ore mass is not.

The most probable causes for geomembrane liner failures under high fill-load conditions, based on post-failure review of available literature, second-hand verbal information, and site or photo observations of 12 known heap leach pad slope failures that have occurred since 1985 (Breitebach, 1998). Another more than 16 failures that have occurred as recently as 2003 are not included due to insufficient information to confirm these failures.

This paper presents the potential for massive heap failure by performing the results of stability analyses using low interface friction angles derived from laboratory test data and a sample cross section typical of some valley fill heap constructed in steep terrain.

2 MECHANISM OF FAILURE

The critical failure surface and factor of safety depend upon the shear strength of the weakest material in the heap, liner and subsoil system. For synthetic materials, the critical failure surface and factor of safety may depend upon the frictional resistance between the ore and geomembrane or between a sand blanket and the geomembrane.

Slope failures on geomembrane liner are far less the three main conditions of instability before or during heap leaching:

a)- sliding along the slope due to low value of the interface friction of the granular veneer with the geomembrane,

b)- tensile tearing of the geomembrane, normally at the crest of the slope where the force is maximum,

c)- failure of the anchorage of the geomembrane when its maximum pullout strength is achieved, Figure 1 (Goure et al,, 1998).

this paper is focussed on condition "'a", establishing the strength characteristics of the geomembrane and second cushion layer interface. Figure 2 illustrate the details of the liner system at Sar-cheshmeh heap leaching.

The bottom lining system of the heap is composed of 2 sections: one section constructed over the rock, which consists of a 0.3 m compacted impermeable.

a) sliding along the liner

b) tensile failure of the geosynthetic

c) pullout failure of the geosynthetic

Figure 1 Different cases of slope failures on geomembrane liner.

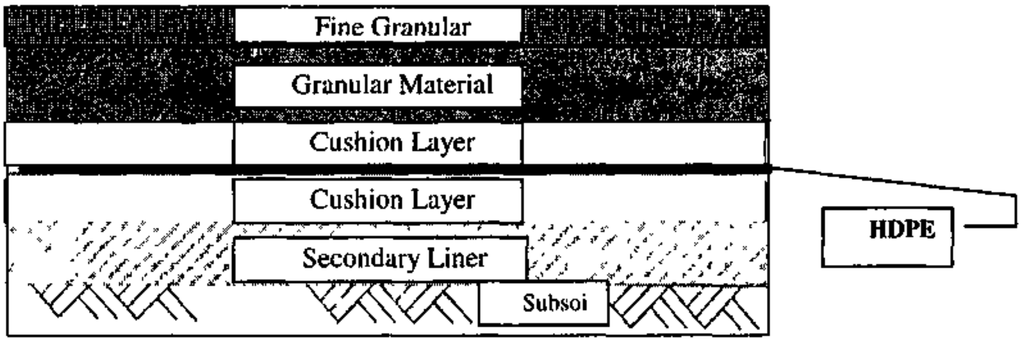


Figure 2 Schematic illustration of base liner.

clay layer that play as a "second liner", and another section constructed over the second liner and comprises a 0.2 m thick fine grained protective "cushion layer".

The 1.5mm HDPE geomembrane liner is laid on the cushion layer. Another cushion layer constructed over the HDPE liner that comprises a 0.2 m thick fine-grained protective. Over the second cushion layer perforated HDPE pipe, was placed within collection ditches with sized gravel, surrounding the pipe to prevent plugging by fines. Finally the liner was covered with 350 mm of select granular material.

2.1 Laboratory studies

Critically important for the proper design of geomembrane-lined side slopes of heap leaching sites, is the soil-to-geomembrane shear strength.

A number of site-specific conditions must be addressed in order to have realistic results, for example: the type and gradation of soil to be placed, the moisture condition during test, the normal stress, to apply, the time for saturation and/or consolidation, the strain rate to use during shear, and the deformation required to attain residual strength, (Koerncr, 1999).

The shear strength developed at a geosynthetic interface is dependent on both the normal stress applied to the interface and the displacement at the interface. Several authors (Seed et. al., 1988; Byrne 1994) have indicated that most geosynthetic interface are strain softening.

Quick direct shear tests of that interface were performed with the material prepared at a range of dry densities and moisture contents representative of the as-placed condition.

2.2 Method and results of shear testing

Test methods to determine the bond strength at a geomembrane soil interface fall into the two broad categories of pullout tests or direct shear tests.

Direct shear testing is dominant since, among other things, there can be difficulty in interpreting results from pullout tests on extensible materials.

Essentially there are five basic methods which can be used for direct shear testing, (Ingold, 1992): fixed shear box, free shear box, large base shear box, central base shear box, and partially fixed shear box

In this research, the method of fixed shear box was performed. This method employ ASTM D532I. This standard on direct shear evaluation of geosynthetic-to-soil, or geosynthetic-to-geosynthetic, recommends a 300 mm x 300 mm square shear box, in which the geomembrane is mounted on a rigid block which is placed in lower half of the shear box. The upper half of shear box is filled with soil which is sheared over the geomembrane below. Measured peak strength of the smooth and textured geomembrane and second cushion layer interface carried out at normal stresses of 0.5. 1. 1.5, 2. 2.5 and, 3 kg/cm A straight line of the Mohr-Coulomb failure envelop gives adhesion (of geomembrane to opposing surface) 0.03 kg/cm and friction angle 19° for smooth geomembrane. For textured geomembrane, the value of 0.07 kg/cm^2 and 31" obtained for. adhesion and friction angle respectively.

The roughened surface of a textured geomembrane results in a significant increase in interface fric-tion with adjacent materials versus the same geomembrane with a smooth surface.

3 STABILITY ANALYSES

At Sarcheshmeh, high density polyethylene (HDPE) in heap leaching is placed in direct contact with cushion layer. This will lead to a component of gravitational force acting in the plane of the geomembrane, which can cause it to slide down the inclined surface. Consequently it is important to be able to assess the bond properties of the interface between the geomembrane and inclined surface. The bond strength which can be made available may be frictional, cohesive or combination of the two, (Karimi Nasab el. al., 2001). Heaps may also fail by sliding along a high slope leach pad because the saturated solution layer in the blanket over the impervious liner lubricates motion. This is also illustrated in Figure 3 (Bartlett, 1995).

The geometrical configuration of the Sarcheshmeh heap area consists of four small valleys Figure 4. So this site is variable and complex and the results ol the analysis do not necessarily apply to the whole site.

Due to some significance in terms of the difficulty of a two-dimensional representation of slope stability analysis, it was therefore decided that stability analysis should utilize a slope stability methodology that incorporated three-dimensional effects for each valley.

A conventional limiting equilibrium technique, based on the Carter and Janbu approaches were employed by using STABL and CLARA softwares for 2-D and 3-D analyses respectively. For non-circular failure surfaces the analyses were conducted using a two-dimensional software of Carter method of slices. The tree-dimensional representation of the base and basal sideslopes used in the stability analysis, one of these results is shown on Figure 5 by CLARA software

Assuming the simultaneous mobilization of peak strength over the entire sliding surface, the predicted factor of safely from the 2-D and 3-D analyses lies within the range of about 1.3 to 1.6, for the zones of saturation above the liner lower than 2 m.

In a number of cases, instability has been due to build-up of high level of PLS in the heap from poor heap drainage or high rates of infiltration due to high rales of leachate application or rainfall (Breitenbach, 1998).

The results of analysis by STABL and CLARA indicated that some profiles along the valley might not be stable due to PLS level in the heap and blast vibrations. So the collection system within the heap must be designed to maintain zones of saturation above the liner at levels as low possible lo provide adequate stability and minimize the risk.

The other controlling factor for the stability of the second site of heap leaching was the interface friction between the geomembrane and the heap in the area of the toe. For providing a satisfactory factor of safety an area of HDPE about 150 m from the toe was identified as requiring textured liner instead of smooth liner, with its higher angle ot friction. The geometrical configuration of the heap area dictate to perform textured liner from the toe to a point ap-proximately halfway up the valley side.

4 CONCLUSION

- The site is variable and complex and the results of the analysis do nol necessarily apply lo the whole site.
- Stability analyses indicate that a textured liner is required from the toe to a point approximately half way up the valley side.
- The presence of fluid pressures acting along all or portions of potential failure plane has the consequence of reducing stability.
- Proper drainage of leach solution above the geomembrane is critical in the performance of the heap leaching project.
- With complex pad geometries such as second site of the Sarcheshmeh heap leaching, some tnal-and-error searching is necessary to locate the more critical sections.

Guiuallv the desııed taetoi of safety is above 1 01 I *S* to piovide sonic nun gin forenoi

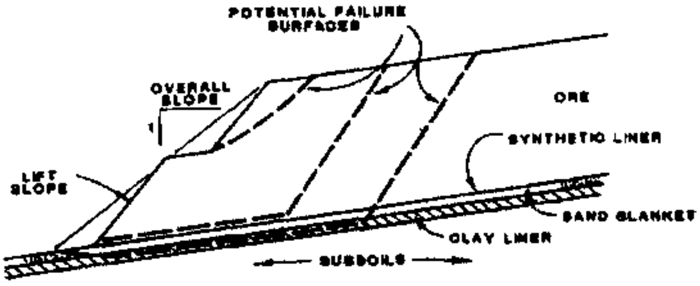


Figure 3 Potential ground failures in a wet ore heap

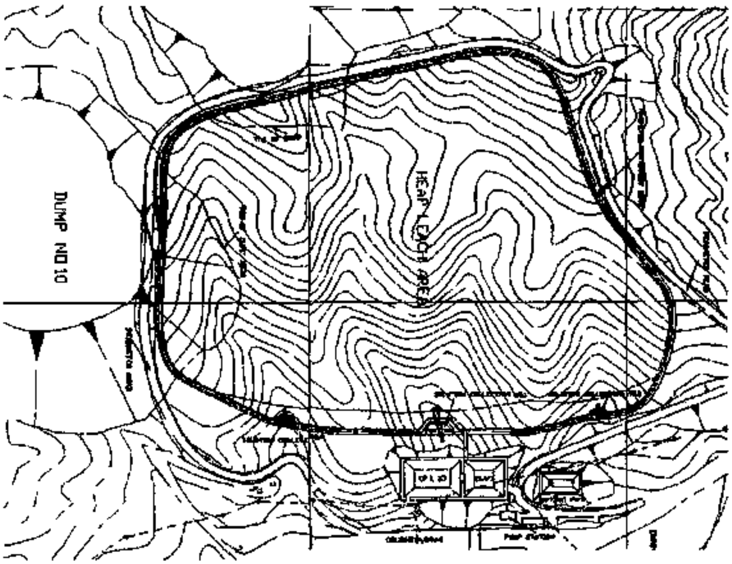
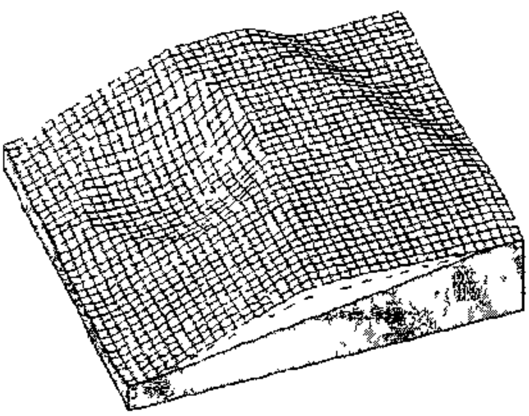


Figure 4 Geometrical configuration of the Sarcheshmeh heap area



Fıguıe i Ciicuhi faıluıe ot 1 D stability analysts by CLARA sottwaie Fs=l S uni Fs=0 789 toi the zones ot satınatıon above the linei lowei than 2 in and moie than4()ni iLspcctively

%

ACKNOWLEDGMENTS

The authors would like to thank National Iranian Copper Industries Company (N.I.C.I.Co.) for supporting this research.

REFERENCES

ASTM D532I-92. Standard Te.si Method lor Deteimining the Coefficient of Soil and Geosynthetic or Geosymhetie and Geosynthetie Friction by the Direct Shear Method. *ASTM Slaiulartl Book.*

Baitlett. Robert W.. 1995. *Solution Minitiji: Lent hing ami Fluid Recovery of Material*.* Gordon and Breach Publishers.

Breitenbach Allan. 1998. Geomeinbrane Pad Liner Failure under High Heap Fill Loads. *Geateclmkal Fahrun Report.* January-February, pp. 38-40.

Byme R J.. Kendal. J. & Biown. S.. 1992. Cause and mechanism of failure. Kettleman hills landfill B-19. unit 1A. Stability & Performance of Slopes and Embankments. *ASCE, Geotecli. Spa.* Pub. No. 31 : II88-1.520.

East Donald R. & Valera Julio E.. 2000. Stability Issues Related to Tailing Storage and Heap l.each Facilities. *Slope Stability in Surface Mining, SME.* Hustruhd William A . McCarter Michael K. Editors, pp. 411-417.

Gourc J.P.. Villard P. & Thomas S., 1998. Geomechanical Aspects of the Use of Geosynthetics in Landfill. *Environmental Geoteclinic\.* Sêco e Pinto Editor. Vol. 4. Balkema. Rotterdam.

Ingold. T. S.. 1992. Friclion Testing. *International Journal Gcolevtilex anil Geomenibranex,* pp. 176-203

Koerner Robert M.. I9V9. *De.sifiiiiiix with Gco.wnl/ietics.* 4"' ed.. Prentice Hall.

Karimi Nasab S,, Atashpanjeh A. & Mollaei Fard M. R.. 2001. Design Considerations of Heap Leaching at the Sarcheshmeh Copper Open Pit Mine.//' *International Mining Congress of Turkey- IMCET.* pp. 513-516.

Seed. R. B.. Mitchell. J. K. & Seed. H. B,, 1990. Kettleman hills waste landfill slope failure. II: stability analysis. J. of Geotech. Engng.. ASCE. 116 (4): 669-689.