Improvement in Slope Stability Performance of Lined Heap Leach Pads from Design to Operation and Closure[1]

A.J. Breitenbach[2], M.S., P.E.
AB Engineering Inc., Littleton, Colorado, USA

1.0 INTRODUCTION

Gold, silver and copper ore heap fill structures generally have existing or planned final fill heights in the range of 100 to 200 feet (30 to 61 meters). Some of the highest heap fills exceed 300 feet (91 meters) in vertical height, especially at the copper mine sites. The ore heap fills consist of granular crushed or run-of-mine rock materials placed in controlled loose lifts and wetted during leaching operations for removal of the target ore metals.

ABSTRACT: The exterior slopes of high ore heap fills are typically constructed as steep as practical during mining operations to maximize the amount of ore tonnage on lined or partially lined leach pad foundations. The maximum allowable overall heap slope angle with benches is determined in design for maintaining stable slope conditions to the planned ultimate heap height.

A major environmental concern for heap fill reclamation and closure is the long-term slope stability performance of the relatively high ore heap fills at the end of operations. This paper presents a comparison study of typical design to post-mining slope stability conditions for geomembrane-lined heap leach pads. The comparison study considers the operational change in heap fill and liner strengths with respect to high fill load conditions from initial construction to final operation in an idealized study section. The design to operation and closure study shows an increase in the long-term heap slope stability with time, which is in general agreement with the historical performance of geomembrane-lined fill structures.

The gold and silver heap leach pads have been constructed and operated within the last 20 years with geomembrane-lined foundations. The copper heap leach dump operations have generally changed to geomembrane-lined foundations and interlift liners within the last 5 to 10 years. A small percentage of the lined heap fill operations are currently in the final stages of leaching and/or rinsing for reclamation and closure.

This paper focuses primarily on the long-term slope stability of heap fill structures on geomembrane-lined leach pads at the end of operations for post-mining closure. The underlying heap foundation conditions are assumed to be high strength and stable for this study. Some aspects of the heap slope stability analyses in this study may be applicable to the post-mining conditions of leach dump and waste dump fill slopes, as well as lined landfill structures.

---

[1] Published in GFR Engineering Solutions, Vol. 22, No. 1, January/February 2004. Translation to Spanish and Portuguese provided by Vector Engineering, Inc.

[2] Allan J. Breitenbach is a consultant to Vector Engineering, Inc. and has over 25 years of geotechnical engineering experience in site investigations, site selection, design and construction of tailings dams, heap leach pads, and geomembrane lined pad and pond facilities for the mining industry. Mr. Breitenbach has designed more than 80 gold, silver and copper heap leach projects in North America and overseas. The international tailings dam and heap leach project locations include Australia, Burma (Myanmar), Canada, Chile, Guyana, Mexico, Nicaragua, Panama, Peru, Romania, Spain, Thailand, Turkey and Venezuela.

2.0 BACKGROUND

2.1 General

Limited published information is available concerning the estimated long-term slope stability performance of high fill structures on geomembrane-lined foundations. This section provides an overview of heap construction with references to the historic performance and past field and laboratory test studies of granular fill materials and geomembrane liner systems under high fill load conditions. Photographs of typical leach pad construction and operation are shown on Photos 1 to 5.

2.2 Heap Construction

The construction of heap fills involves placement of precious or base metal ore materials in controlled individual loose and relatively dry fill lifts stacked at the natural angle-of-repose. Each ore lift surface is wetted uniformly during leaching by using irrigation drip emitters or sprinkler sprays. Leaching is generally conducted in 30 to 120 day or longer leach cycles with barren or recirculated alkaline (gold and silver) or acidic (copper) process solutions.

The maximum rock particle size of the granular ore materials range from run-of-mine cobble and boulder rock sizes to crushed sand and gravel sizes. The crusher operations may include agglomeration as needed to provide a more efficient distribution of fines (minus No. 200 sieve size material) for improved permeability and recovery of the target metals. The individual ore lifts are offset with benches along the exterior slope, as required for establishing the overall stable design slopes for operations. A schematic section of the exterior ore heap slope is shown in Figure 1.

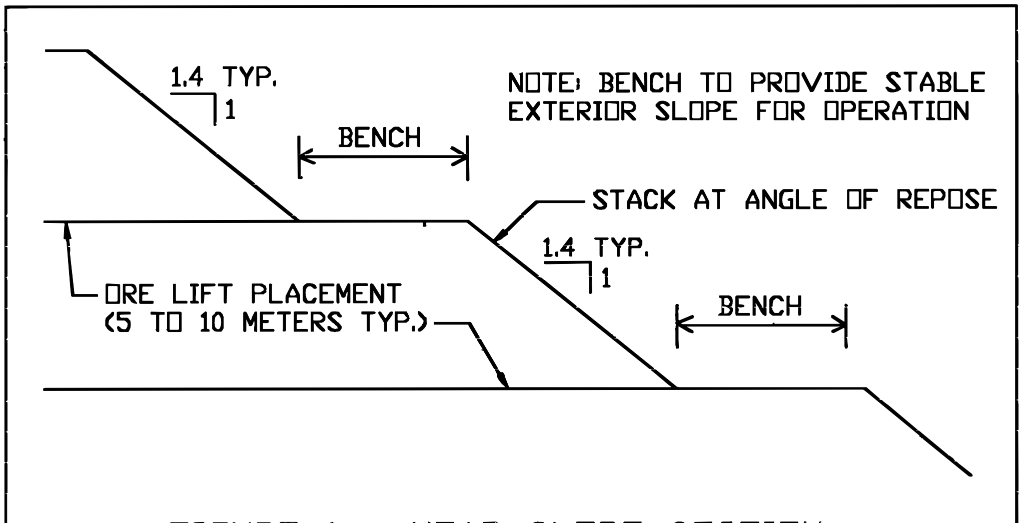

FIGURE 1 - HEAP SLOPE SECTION

Each individual ore lift is typically placed at 15 to 30 feet (5 to 10 meters) in height and thoroughly wetted during prolonged leach cycles with releaching of underlying ore lifts. The controlled leach application flow rates are in the range of 0.002 to 0.005 gallons/minute/square foot (less than 0.2 liters/minute/square meter), which keeps the granular ore materials moist and unsaturated. The multiple ore lift fill placement and controlled wetting from leaching causes the fully drained granular fills to consolidate and gain strength with each lift from self-weight settlement and densification. The ore heap fill settlements from controlled loose ore lift placement and wetting generally range from 7 to 10 percent for leached silver and gold ore heaps and 10 to 15 percent for leached copper ore heaps. The relatively long operational leach cycles allow any excess foundation pore pressures in the subgrade beneath the liner system to dissipate with time between each successive heap ore lift load.

2.3 Historic Performance

The past slope failures on geomembrane-lined fill structures, such as solid waste landfills, heap leach pads, and reclamation cover fill caps, have shown that liner induced slides generally occur at the planar geomembrane liner interface contact with weaker underliner or overliner materials. One of the earliest and most known geomembrane induced slope failure was the Kettleman Hills landfill slope failure in Northern California in 1988 (Mitchell et al. 1990). Several major landfill slope failures occurred between 1988 and 1997 in North America, Europe, Africa and South America (Koerner and Soong 1999). Several less known leach pad slope failures occurred between 1985 and 1993 at mine sites in North America, South America and Australia (Breitenbach 1997). The Northridge earthquake in Southern California in 1994 (Matasovic et al. 1995) and subsequent earthquakes in Chile and Peru in 1995 and 1996 gave some insight into the seismic behavior of high fills on geomembrane liner systems.

The historic performance of fill structures on geomembrane liner systems indicates that translational (lateral movement) wedge slip failures generally occur along the planar liner interface contact with soils or geosynthetic materials. However, heap leach slope failures differ from landfill failures in that the slope failure generally occurs during the initial ore heap fill lift placement operations, rather than at the higher heap fill lift heights. The only exceptions for higher fills, concerning both lined heap leach pads and landfills, include high fill structures with either weak foundation conditions beneath the lined facility or excessive hydraulic conditions within the containment materials above the impervious liner system.

The known slope failures from weak foundation conditions beneath the liner system included a combination of one or more of the following: re-activating an existing landslide surface, thawing of frozen subgrade soils, overly wet subgrade (natural soils or underliner fills), subgrade subsidence in compressible natural soils or low density fills, and excavations at critical downgradient fill toe areas that caused unloading of toe support materials. The known slope failures from excessive hydraulic conditions above the liner system included a combination of one or more of the following: intense rain storm events, poor internal containment material drainage, excess solution application on the surface or injection into the fill material, or solution pipeline breaks near exterior fill slopes. These high fill slope failure exceptions are rare in occurrence and can be eliminated or mitigated at the end of operations into closure.

2.4 Past Test Studies

2.4.1 General

The granular soil and rock fill strength for the design of most ore heap fills can be conservatively estimated from a literature review of past large-scale field and laboratory test studies. However, past slope failures on geomembrane-lined fill structures indicate that liner interface test strengths should be determined from site specific materials to accommodate planned design, construction and operation conditions.

2.4.2 Granular Fill Strengths

The difficulty in accurately sampling and testing the larger rock sizes in the heap fills for strength, density and gradation generally limit the engineer to past experience and published test results for comparison to site specific conditions. The estimated strengths for coarse grained granular soil fills or rock fills can be determined by surveyed field measurements of the natural angle-of-repose of the actual fill slope surfaces or a review of past large-scale laboratory direct shear box and triaxial cell strength testing. The density and gradation of coarse granular soils and rock fills can be determined from large-scale field test studies (Breitenbach 1993).

The rock size (small versus large), rock quality (weak versus strong), rock shape (round versus angular), particle size distribution (poorly graded versus well graded), fines content (clayey and silty versus clean), plasticity (plastic versus non-plastic), moisture content (wet versus dry), confining stress (low versus high), and relative density (loose versus dense) influence the granular fill material strength properties. The relative change in strength with respect to rock particle size, distribution and relative density for fine to coarse grained soils is illustrated graphically in Figure 2 (NAVFAC 1982). The graph shows that the densification and increase in rock particle size from silts (ML) to sands (SM, SP) to gravels (GP,GW) significantly increases the shear strength of the fill materials. The soil classification shown in the graph is according to the Unified Soil Classification System (ASTM D-2487).

Large-scale rock fill and granular soil fill test strength studies by Leps (Leps 1970) and later by Marachi (Marachi et al. 1972) and Barton (Barton and Kjaernsli 1981) are summarized graphically in Figure 3. These test summaries show the influence of high confining stresses, which tend to decrease the rock fill strengths with increasing fill heights. The rock fill test strengths in Figure 3 are more applicable to the run-of-mine and primary crusher ore rock fills, but generally show that the increase in strength from densification of coarse grained soils and rock fills eventually is offset by the increase in vertical confining stress loads. In other words, a granular heap fill placed in controlled loose lifts, wetted during leaching, and subsequently loaded by the next successive ore lift should gain in strength to some optimum fill depth, at which additional loading decreases the fill strength.

The optimum fill depth for determining changes in the granular heap fill strengths can be indicated by laboratory load consolidation and tri-axial shear testing. The optimum depth selected for this study is defined as the heap fill depth at which additional incremental changes in fill density are minimal under continued ore lift loading, as simulated by equivalent loads in laboratory consolidation testing.

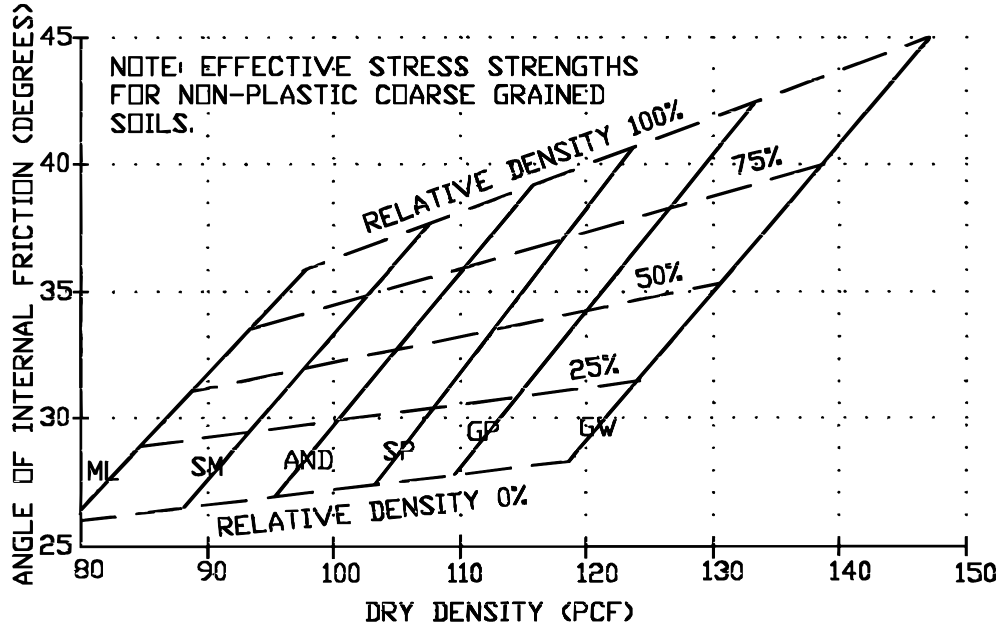

FIGURE 2 - FRICTION STRENGTH VS. DENSITY

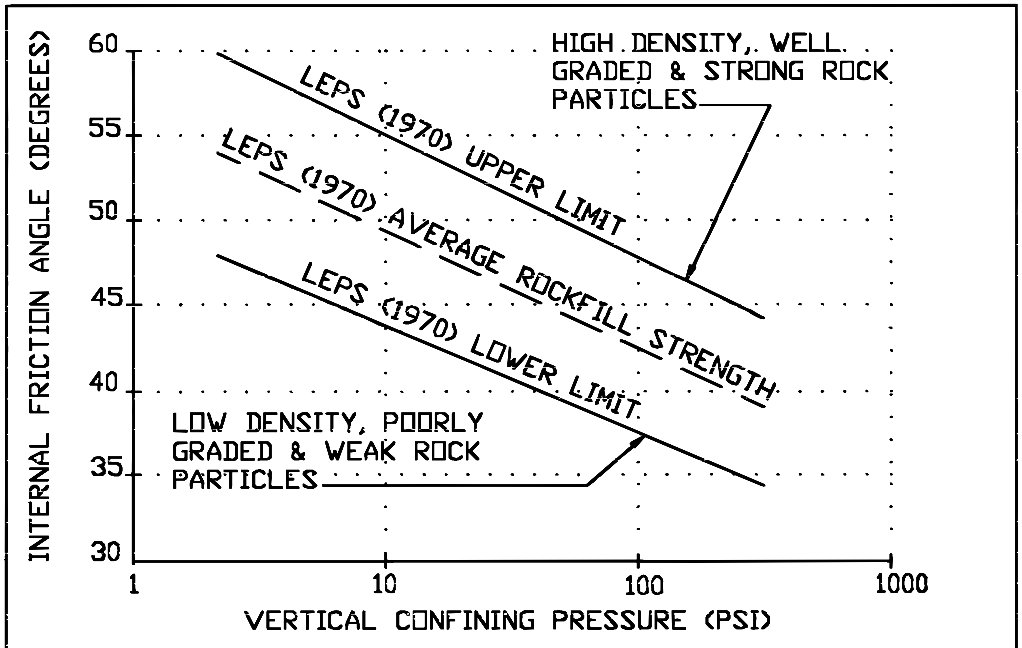

FIGURE 3 - ROCKFILL STRENGTH VS. PRESSURE

A determination of the fill strength with respect to depth requires an understanding of the site specific consolidation characteristics of the wetted fill materials. Large diameter laboratory consolidation test results of crushed angular minus 0.5 inch (12.7 mm) leached copper ore and minus 2.25 inch (57.2 mm) gold ore rock particles, loaded to an equivalent 200 to 300 feet (61 to 91 meters) heap fill height, are shown in Figure 4. The consolidation tests show a rapid densification of the loose ore materials with initial loading, and generally no significant additional increase in the fill density for fill height loads beyond about the 200 feet (61 meters) equivalent fill height.

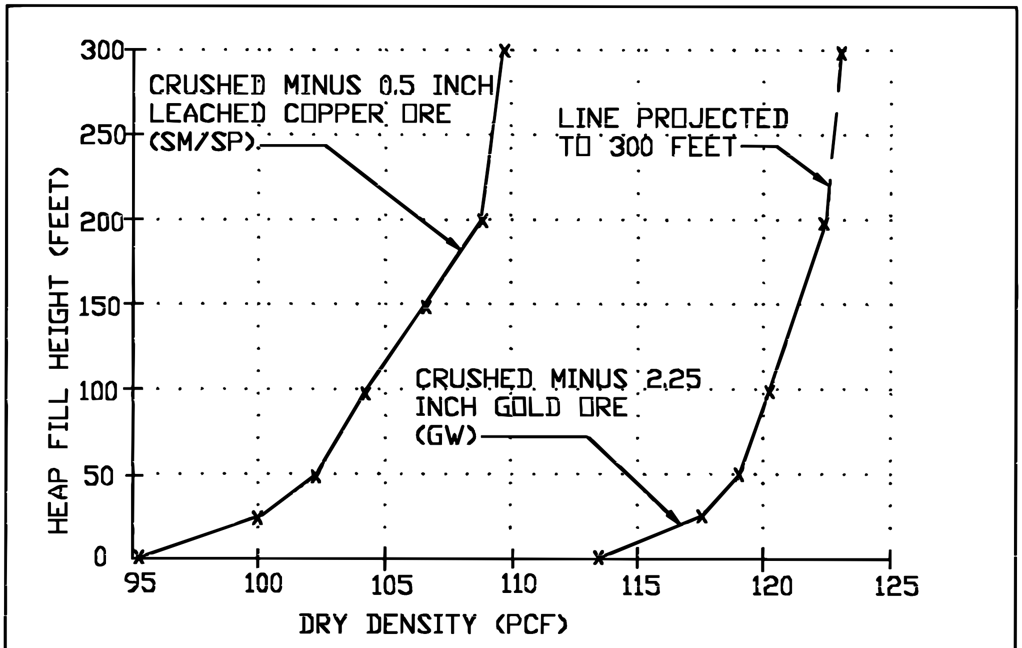

FIGURE 4 - FILL HEIGHT VS. DRY DENSITY

The laboratory test guidelines summarized in Figures 2, 3 and 4 indicate that a loosely placed and wetted granular fill should rapidly increase in strength during subsequent fill lift placement to an optimum depth, and eventually begin to reduce in strength under higher confining stress loads. Therefore the idealized study section will assume an incremental increase in heap fill strengths to a depth of 200 feet (61 meters) for analyses.

2.4.3 Liner Interface Strengths

Past large-scale laboratory direct shear box test results from studying the long-term liner performance under high fill loads indicate the geomembrane liner interface contact with underlying and overlying fill materials gains strength with time (Breitenbach and Swan 1999). The increase in liner interface peak and residual friction strength with respect to time is mainly due to two conditions: 1) the apparent affect of high load deformations or dimpling on a micro scale in the planar geomembrane liner surface contact with overlying and underlying soils and 2) a reduction in low permeability underliner soil excess pore pressure conditions with time. The measured cohesion or apparent adhesion strengths in the study decreased with time and were assumed to be negligible for conservative long term liner strength conditions.

The high load liner deformations cause the critical interface sliding failure surface to shear through a portion of either the underlying or overlying soil materials in a composite soil and geomembrane liner system. This dimpling affect results in an overall measured increase of typically about 5 degrees of interface liner friction strength following a 24 to 48 hour load consolidation period on a composite liner system at optimum underliner soil moisture conditions, as shown in Figure 5. Note that overly wet low permeability underliner soil moisture conditions can result in low initial interface friction strengths from high excess pore pressure conditions (like during placement of the first ore lift load on an overly wet composite liner system). However, the test results in Figure 5 show that the interface strength can recover over 48 hours of time to optimum moisture strengths at zero time load conditions for a moderate plasticity clay underliner soil. Pore pressure conditions in the composite liner system at closure would essentially be zero at the downhill toe section of the heap fill for maximum liner interface strength conditions at closure.

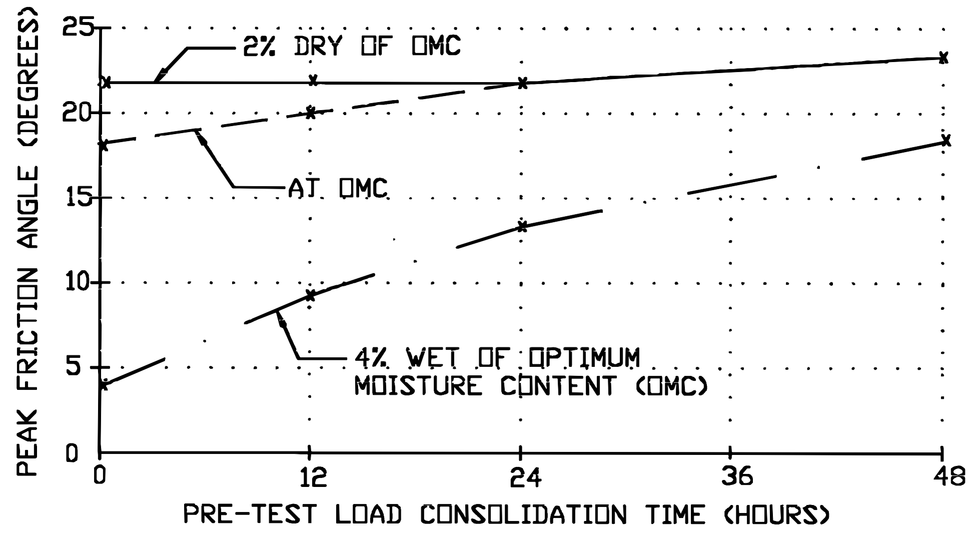

FIGURE 5 - INTERFACE LINER STRENGTH VS. TIME

The measured dynamic laboratory geomembrane liner interface strengths from simulated cyclic earthquake conditions on several composite geomembrane liner systems indicates the interface strength remains constant or increases from dynamic earthquake loads (De and Zimmie 1997, Nicola and Filippo

1997). Therefore dynamic interface strengths were not considered for this post-mining slope stability study.

3.0 SELECTED STRENGTHS

The selected strengths for long-term heap fill slope stability analyses are based in part on the background information obtained from the historic performance and test studies, and in part on site specific material characteristics and engineering judgement in the design and construction of heap and mine waste dump fills. For example, copper ore heap fill strengths are generally weaker than silver and gold ore heap fill strengths due to copper acid leaching and accelerated chemical weathering, which affect the rock quality of the granular fill particles. Typical ore fill strengths were selected for this analyses and require adjustment to site specific conditions.

The selected ore strengths for design analyses are generally conservative and do not account for the densification and gain in heap fill strengths with depth or the added interface liner strength from the non-planar liner dimpling affect. For simplification purposes, moderate and uniform heap fill and liner strengths were assumed for the design slope stability analyses with both smooth (design) and dimpled (construction) interface liner strengths, as presented in Table 1. Note that the interface dimpling strength assumed for construction is dependent on high fill loads, as well as the moisture condition of the low permeability underliner soils in contact with the geomembrane liner, as shown on Figure 5.

Table 1 – Assumed Design and Construction Strength Parameters for Idealized Section

Material type	Heap height		Moist density		Friction angle (degrees)	Cohesion	
	(feet)	(meters)	(pcf)	(g/cm3)		(psf)	(kPa)
Liner surface (smooth)	0 to 300	0 to 91	62.4	1.00	18	0	0
Liner surface (dimpled)	0 to 300	0 to 91	62.4	1.00	23	0	0
Heap layer	0 to 300	0 to 91	115	1.84	36	0	0

The selected operation strengths for post-mining slope stability at closure reflect both the anticipated relative increase in wetted and consolidated ore strengths and densities with depth and the liner dimpling affect. The long-term stability section assumes four heap layers increasing in fill density and strength with depth for analyses, as presented in Table 2.

Table 2 – Assumed Operation Strength Parameters for Idealized Section

Material type	Heap height		Moist density		Friction angle (degrees)	Cohesion	
	(feet)	(meters)	(pcf)	(g/cm3)		(psf)	(kPa)
Liner surface (dimpled)	0 to 300	0 to 91	62.4	1.00	23	0	0
Heap layer 1	0 to 25	0 to 8	115	1.84	36	0	0
Heap layer 2	25 to 50	8 to 15	119	1.91	38	0	0
Heap layer 3	50 to 100	15 to 30	121	1.94	39	0	0
Heap layer 4	100 to 300	30 to 91	122	1.96	40	0	0

The assumed change in moist density at depth for the four heap layers is modeled after the gold ore dry density consolidation curve shown on Figure 4. Site specific adjustments can be made to the assumed heap densities reflecting dry versus moist conditions, crushed versus run-of-mine rock sizes, chemical weathering of leached and spent ores, or other factors influencing the heap density at the end of operations to closure.

4.0 IDEALIZED STUDY SECTION

The idealized design and operation study sections for slope stability focus on the relative change in ore heap fill densities and related fill and liner strengths under static wedge failure conditions. The idealized study sections assume a maximum heap height of 300 feet (91 meters), 2 horizontal to 1 vertical overall exterior slopes, a flat geomembrane liner foundation surface grade, and fully drained conditions with no phreatic water level. The foundation was assumed competent and stable beneath the heap fill and liner system.

The static wedge planar failure conditions were analyzed at vertical heap height intervals ranging from 25 to 300 feet (8 to 91 meters) in vertical height for both the design and operation study sections. The assumed failure wedge extends from the heap crest height to the interior liner surface on a flat planar grade, and then to the heap toe area, as determined by the Rankine Method. The maximum design and operation study sections for analyses are shown in Figures 6 and 7, respectively.

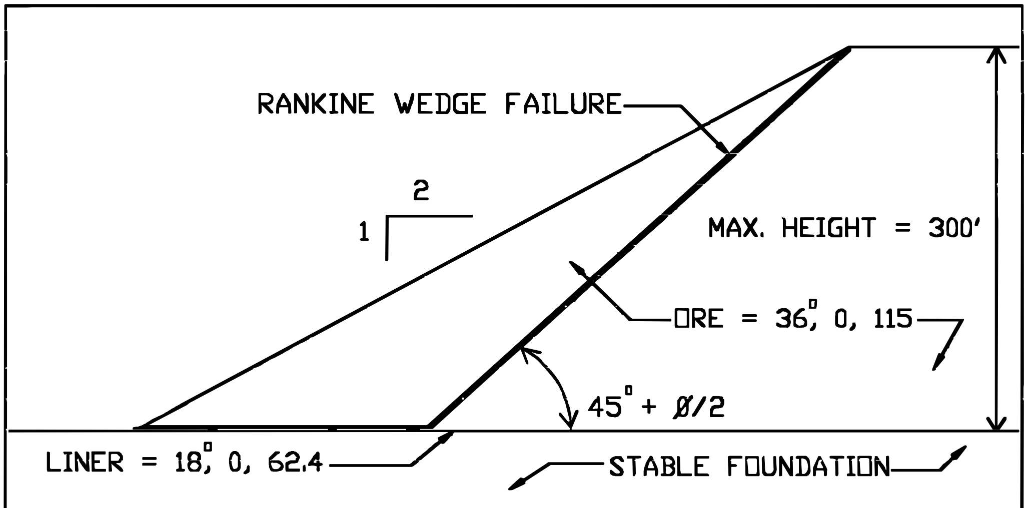

FIGURE 6 - DESIGN STUDY SECTION

Note that the maximum load height of fill materials on a wedge slope failure surface is generally less than 50 percent of the heap height. Therefore the fill strengths deeper than 100 feet (30 meters) in Figure 7 were assumed the same for simplification purposes. The Rankine Method for the wedge failure surface, as well as sloping leach pad liner surface grades and heap exterior slopes and heights, can be adjusted accordingly to find the lowest safety factor for site specific conditions.

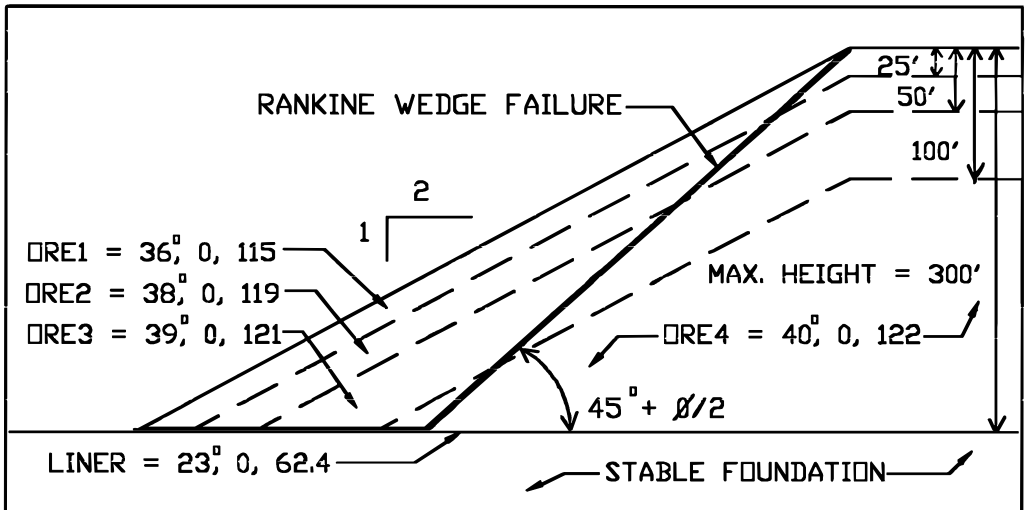

FIGURE 7 - OPERATION STUDY SECTION

5.0 STABILITY ANALYSES SUMMARY

A summary of safety factors (SF) versus fill heights for design, construction and operation wedge failure stability analyses is shown on Figure 8. The stability analyses summary reflects static wedge failure planes from the crest of each incremental heap height to the geomembrane liner surface. Underliner foundation conditions were assumed to be stable for this analyses.

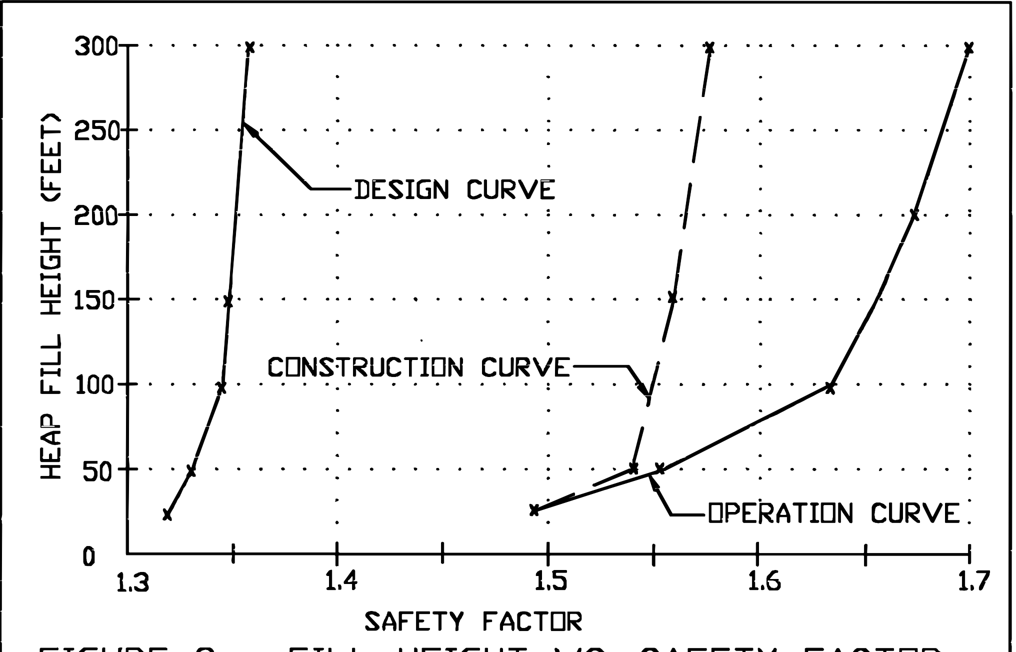

FIGURE 8 - FILL HEIGHT VS. SAFETY FACTOR

The design stability analyses show a slight increase from SF = 1.32 to 1.36 for a conservative uniform ore fill and liner interface strength over an incremental fill height of 25 to 300 feet (8 to 91 meters), respectively. The analyses indicate that the heap should become more stable with each successive ore lift height, assuming no change in strength parameters and the overall exterior heap slope angle. This is in agreement with the performance of heap fills, where most of the slope failures occurred during initial ore lift placement rather than at the maximum heap height.

The long term construction and operation strengths of both the granular fill and liner strength were analyzed next to reflect field conditions and measured laboratory liner strengths subjected to more than 12 hours of consolidation loading. Modifying the interface liner friction strength by 5 degrees to reflect micro scale liner dimpling under a high heap fill load condition significantly increases the heap stability from SF = 1.36 (design) to SF = 1.57 (construction) at the 300 feet (91 meters) fill height. Additionally modifying the ore fill strength to reflect operational wetting and densification of the ore heap fill increases the end of operation stability analyses from SF = 1.57 (construction) to SF = 1.70 (operation) at the 300 feet (91 meters) fill height.

Note that safety factors under relatively low load conditions of 50 feet (15 meters) or less should not be considered in the comparison of design to construction and operation safety factor curves. The leach pad liner surface should have less non-planar dimpling under low load conditions for anticipated minimal changes in interface strength conditions beneath the initial ore lift. The densification and change in strength of ore materials in the first ore lift above the pad liner system also would not be significant until subsequent ore lift loads are applied. Therefore the idealized study section comparison of design to construction and operation safety factor curves should only be applicable for multiple ore lift construction representing high load conditions.

6.0 CONCLUSIONS

The design stability analyses for construction of loosely placed ore heap fills is typically conservative for safe operations and do not consider the increase in fill and liner strength with time during operations. The operational conditions for the fully drained granular ore fill materials include wetting and consolidation under controlled ore lift placement and leaching operations. The slope stability summary results show the most critical safety factors occur at the startup of operations during the initial ore lift placement. The heap stability increases with continued ore lift placement, assuming stable foundation conditions beneath the heap fill and liner system. The overall increase in the ore heap slope stability for the idealized study section from design to final operations shows about a 20 percent increase in the safety factor at the maximum heap height for closure.

The long-term slope stability of high fills on geomembrane liner systems for closure is anticipated to increase with time, assuming the slopes are protected from erosion and the heap fill remains fully drained on a stable underlying foundation. The stability analyses summary results for an idealized study section are in general agreement with the historic performance of high fills on geomembrane liner systems.

7.0 REFERENCES

Barton, N. & Kjaernsli, B. (1981); “Shear Strength of Rockfill”, Proceedings of ASCE, Geotechnical Testing Journal, Volume 7, pp. 873 to 891.

Breitenbach, A.J. (1993); “Rockfill Placement and Compaction Guidelines”, Proceedings of ASCE, Geotechnical Testing Journal, Volume 16, pp. 76 to 84.

Breitenbach, A.J. (1997); “Overview Study of Several Geomembrane Liner Failures Under High Fill Load Conditions”, Geosynthetics ’97 Conference Proceedings, Industrial Fabrics Association International, Long Beach, California, Volume 2, pp. 1045 to 1062.

Breitenbach, A.J., & Swan Jr., R.H. (1999); “Influence of High Load Deformations on Geomembrane Liner Interface Strengths”, Geosynthetics ’99 Conference Proceedings, Industrial Fabrics Association International, Boston, Massachusetts, Volume 1, pp. 517 to 529.

De, A. & Zimmie, T.F. (1997); “Factors Influencing Dynamic Frictional Behavior of Geosynthetic Interfaces”, Geosynthetics ’97 Conference Proceedings, Industrial Fabrics Association International, Long Beach, California, Volume 2, pp.837 to 849.

Koerner, R.M., Soong, T.Y. (1999); “Stability Analyses of Ten Landfill Failures”, Proceedings 2nd Austrian Geotechnical Congress, Austrian Engineering and Architects Society, Eschenbachgasse, Vienna, pp. 9 to 50.

Leps, T.M. (1970); “Review of Shearing Strength of Rockfill”, Proceedings of ASCE, Soil Mechanics Journal, Volume 4, pp. 1159 to 1170.

Marachi, N.D., Chan, C.K., & Seed, H.B. (1972); “Evaluation of the Properties of Rockfill Materials”, Proceedings of ASCE, Soil Mechanics Journal, Volume 1, pp. 95 to 114.

Matasovic, N., Kavazanjian Jr., E., Augello, A.J., Bray, J.D., & Seed, R.B. (1995); “Soil Waste Landfill Damage Caused by 17 January 1994 Northridge Earthquake”, Woods & Seiple, (eds), California Department of Conservation, Division of Mines and Geology.

Mitchell, J.K., Seed, R.B., & Seed, H.B. (1990); “Kettleman Hills Waste Landfill Slope Failure”, Volume I: Linear Systems Properties, Geotechnical Engineering Journal, ASCE, 116 (4), pp. 647 to 668.

NAVFAC (1982); “Correlation of Strength Characteristics for Granular Soils”, Department of the Navy, Naval Facilities Engineering Command, Alexandria, Virginia, Soil Mechanics Design Manual 7.1, Figure 7, p. 149.

Nicola, M. & Filippo, M. (1997); “Behavior of Geogrids Under Cyclic Loads”, Geosynthetics ’97 Conference Proceedings, Industrial Fabrics Association International, Long Beach, California, Volume 2, pp. 961 to 983.

Photo 1 – Stacking and Leaching of Ore Heap on Lined Leach Pad (Arizona)

Photo 2 – Placement of Drain Cover Fill on Leach Pad Liner (Nevada)

Photo 3 – Dozer Spreading Drain Cover in Controlled Lift (Nevada)

Photo 4 – Heap Leaching with Drip Emitters on Ore Surface (Northern Peru)

Photo 5 – Lined Expansion of Leach Pad for Heap Operations (Montana)