Interface Shear: Towards understanding the significance in Geotechnical Structures

G.C. Howell & A.H. Kirsten
SRK Consulting. Johannesburg, South Africa

ABSTRACT: It is well known that low strength natural materials under foundations and in slopes can cause bearing capacity and slope failures. With the increased use of manufactured materials such as HDPE liners and geotextiles, similar conditions can inadvertently be built into the structure. These interface shear aspects occur between liner and geotextile, liner and soil, geotextile and soil and even within the confines of the geotextile. Injudicious use of low interface shear materials can lead to excessive deformation and even catastrophic failure. The many potential interface shear planes that can exist in geotechnical structures are considered with reference to 20 years of published research and actual recent shear box tests carried out for validation purposes for lined structures. The paper further considers a simplistic calculation method using block limit equilibrium and finite element simulations to understand the problems that exist in these structures. The concept of 'excess shear' which leads to overstressing of liners is also discussed.

1 INTRODUCTION

Recent failures of geotechnical structures have again highlighted the requirement of engineers to fundamentally understand the materials that they are working with. Unlike other spheres of engineering where material properties are known to a large extent within a defined band (concrete, steel, aluminum), geotechnical materials are variable and subject, amongst other issues, to changed conditions due to pore pressures and seismic excitation. In one such case (Independent Expert, 2015), an apparently stable structure subject to pore pressures and susceptible low strength foundation clay strata, lead to failure. The lessons learnt were that the geotechnical investigation failed to adequately identify the properties of the clay in the foundation and the design engineers failed to appreciate the significance of the location and strength of the material within their structure. Another case recently of a tailings dam failure in Brazil, lends weight to the issue of fundamental engineering appreciation for such structures.

Here is Southern Africa there have been numerous examples of slope and bearing capacity failures that show the same lack of appreciation of engineering principles that are evident in other incidents. In particular, the existence of weak subgrade materials, weak or gouge filled joints in otherwise strong rock and layered materials in stockpiles and tailings dams are examples of these issues. Even so, more and more, engineers are using 'new age' materials in designs for practical and economic reasons, without possibly comprehending the associated risks. HDPE, LLDPE, PVC, geosynthetics and geotextiles in general, in combination with soils in structures, represents an instance of such risks unless judicious appreciation of their interaction is clearly understood.

The objective of this paper is mainly educational in terms of two themes:

- By presenting the interface friction properties of commonly used manufactured liners with soil and other geosynthetics in order to highlight the relative strength (rather weaknesses) inherent in these interfaces. This has been achieved by interrogating literature and own results from direct and ring shear testing done on such interfaces. Comments and cautions are provided to guide the design engineer in new applications; and
- By presenting an approach to conceptualizing the mechanics of a slope stability design problem that includes interfaces in a coherent, practically and understandable way by using a first principles approach. By so doing the physical importance of the design parameters can be visualized and appreciated. The method uses a limit equilibrium, or static equlibrium, approach by balancing forces in a slope, with the objective of understanding how the energy in structure is sustained by the parent material itself, by the interface and how liner tension is developed as a consequence.

The overarching objective is to foster an appreciation for the importance of clearly 'defining the problem' in fundamental terms before attempting a solution. Clear understanding of the problem from the outset is a prerequisite for a solution.

2 PROBLEM DEFINITION

Figure 1 has been drawn to indicate 5 (of many) possible low strength planes within a slope (natural or manufactured). 1 represents a foundation plane of weak material; 2 represents a weak interface at ground level, possibly as a result of the insitu strength of the material, or from a manufactured interface such as a geomembrane; 3a and b represent inclined planes of viable angle, either from natural jointing (in a rock slope), deposition of variable low strength materials in a tailing dams or a manufactured interface surface in a constructed embankment; 4 represents similar horizontal natural or manufactured interfaces; 5 represents a common circular failure; and 6 represents a piecewise linear failure plane resulting from failure along the base interface and inclined through the body of the material.

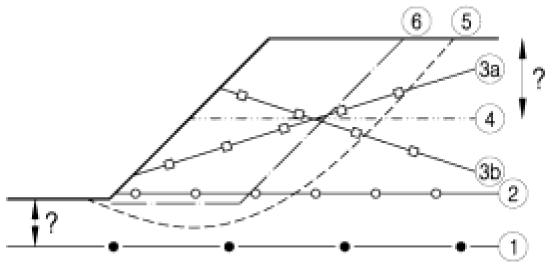

Figure 1: Potential failure planes in a slope

In this paper, we will be concerned mainly with interfaces 2 (a manufactured interface nominally along the base of a dam or embankment) and 6 the resulting failure plane along the base and through the parent material, but other combinations of interface layers and failure planes are also addressed intrinsically but not specifically in the discussion that follows.

From a design perspective, there are a number of issues that need to be addressed. These include the following:

1. The interface shear strength along the potential failure plane;
2. The development of deformation and strain along the failure plane; and
3. The consequent development of tension in the geomembrane.

These aspects have to be fundamentally understood for any design. Interface shear strength properties are sourced from soils laboratory testing and from experience with the use of these materials. Section 3 considers these properties for a range of geomembranes and textiles with interfaces to soils (granular and clay) and to other geomembranes. Section 4 considers fundamental concepts, while the approach covered in Sections 5 and 6 assists in defining the fundamental analysis aspects in a simple but effective limit equilibrium treatment.

3 INTERFACE SHEAR PROPERTIES

A potential weak zone (either natural or manufactured) consists of 2 or more interfaces that need to be assessment individually and collectively, since the interface with the minimum shear strength will dominate the overall behavior. In addition, a weak zone can be due to the nature of the material itself, for example a clay layer, or the bentonite clay in some GCL products that are not thermally locked within the geotextile carrier layers. The most common situations with manufactured materials occur on the upper and lower surfaces of the interface that is created between the geosynthetic, the soil and/or other geosynthetics.

The information that is given here is derived from literature and from physical testing on the interface properties carried out by the authors' colleagues during the course of projects. However, it should be noted that this information is purely indicative for use in preliminary design only. Once the interface materials have been chosen, then careful and thorough physical testing of the sandwich of materials to be used in the design is essential, since variations in actual behavior can be expected.

Only interfaces from commonly used material are reported. The current data base available to the authors includes 143 tests direct and ring shear tests on various interfaces. With time, this data will be augmented to include additional information from other physical testing sources.

Although subgrouping is difficult since (particularly for soil) the descriptions are not always clear, an attempt has been made to assess the data base in terms of the following materials:

- HDPE-S and HDPE-T: HDPE geomembranes (Smooth and Textured)
- LLDPE-S and HDPE-T: LLDPE geomembrane (Smooth and Textured)

interfaced with:

- Granular Soil (USCS classification S and M)
- Cohesive Soil (USCS classification C)
- Geotextile (Needle punched)
- GCL (thermally welded needle punched).

It should be noted that geotextiles are produced in various grades and types including needle punched and woven fabrics. Only needle punch geotextile of any grade is reported here. Similarly, GCLs are produced in various grades and types. Only thermally welded needle punched GCLs are reported here. It should also be noted that some GCLs are not thermally welded (meaning that the upper and lower geotextile carrier layers are not physically connected by needle punching strands through the clay (usually bentonite) and welding them in place. Thermal welding creates an internally stable 'lattice' structure within the GCL layer, otherwise internal interfaces exist that themselves can have very low frictional properties, sometimes as low as 1° for bentonite itself.

Tables 1 and 2 give the statistical representation from the database in terms of mean, (standard deviation) and [number] of tests available for peak and residual friction angles for various interfaces under saturated conditions.

Table 1. Interface Peak Friction Angle Properties

Interface	Soils		Geosynthetics	
	Granular	Cohesive	Geotextile	GCL
HDPE-S	21.26 (8.03) [5]	10.64 (6.75) [5]	11.33 (3.51) [3]	8 (-) [1]
HDPE-T	30.51 (6.52) [18]	20.65 (6.75) [27]	25.14 (4.87) [8]	24.2 (13.0) [2]
LLDPE-S	26.63 (0.99) [4]	8.53 (2.55) [3]	10 (-) [1]	- (-) [-]
LLDPE-T	35.89 (5.04) [7]	33.53 (12.92) [9]	26 (-) [1]	- (-) [-]

Table 2. Interface Residual Friction Angle Properties

Interface	Soils		Geosynthetics	
	Granular	Cohesive	Geotextile	GCL
HDPE-S	15.05 (5.05) [4]	10.49 (5.57) [4]	10.5 (2.12) [2]	- (-) [-]
HDPE-T	26.42 (6.85) [16]	19.47 (11.56) [24]	17.08 (1.61) [4]	10.1 (-) [1]
LLDPE-S	21.7 (6.29) [4]	9.07 (2.70) [3]	9 (-) [1]	- (-) [-]
LLDPE-T	26.10 (5.50) [7]	28.08 (11.17) [9]	17 (-) [1]	- (-) [-]

The implications of Tables 1 and 2, are inter alia noted below. Textured geomembrane improves the interface shear by at least 10° in all cases. The variability of the results given by the standard deviation in ( ) shows that the mean value used in analyses cannot be directly justified and that a more realistic design value is the mean less 1.2 to 1.5 times the standard deviation. LLDPE appears to give more consistent results than HDPE. This is probably due to the lower density and softer modulus of the LLDPE which allows more mechanical frictional interaction between the soil (whether granular or finer grained) and the geomembrane. The peak and residual values are also interesting showing that the residual friction angle is some 2 to 5 degrees less than the peak in general for HDPE material. It is however larger (10 degrees) for the LLDPE textured material. Whilst an explanation is not immediately obvious, this could be due to the collapse of the textures under strain because of the softer modulus of the material.

Compare the friction angles from Tables 1 and 2 with of USCS graded materials for gravels (G) (φ = 33 to 40°), Sand (S) (φ = 31 to 38°), Silt (M) (φ = 30 to 33°) and Clay (C) (φ = 22 to 30°). It is apparent that interfaces are significantly inferior to clay in most cases.

The information indicates that extreme caution must be taken by the designers to fundamentally assess the interface effects on the structure. It is also indicative that the structure must be maintained well within the limits of the peak shear stress and strain parameters and 'elastic' range, since once the peak is reached, the residual nature of the interfaces will not sustain the loading. Alternatively put, interfaces of the type considered here exhibit significant strain softening behavior and can never be considered to be elastic perfectly plastic as would be the default in some numerical models.

4 FUNDAMENTAL STRESS CONCEPTS

Conceptually speaking, due to gravity, every structure is represented by a potential energy state that is counteracted internally by the strength (or strain energy) capacity of the system. Alternatively put, strain energy is induced in a structure by virtue of its size and shape which is sustained by the strength of the multiple 'elements' that make up the whole, provided that the fundamental principle of capacity (C) always being greater than demand (D) is never violated. If one element in the system (of many elements) is unable to sustain the demand placed on it, then, like a weak link in a chain, failure will occur. This sounds obvious, but it is still surprising how often this principle is forgotten.

Fortunately most structures exist in a state of redundancy, meaning that no one element alone is responsible for accepting and sustaining the full demand on it. Demand is 'shared' (in a complex way) between many elements in the structure. Should one element not be able to accept the full demand placed on it, then it will 'shed' the excess demand to other elements in the structure. The other elements can therefore 'accept' an increase in stress/strain within their own capacity, or 'shed' the excess demand to the next element in line. This is the phenomenon of system shakedown, or stress/strain relaxation/redistribution and is accompanied by plastic strain deformation.

4.1 *Model representation*

In the design of geotechnical structures, the development and positioning of a failure plane is an essential requirement in conceptualizing the failure mechanisms within the structure.

Modeling using finite element, finite difference or discrete element software programs are favoured by 'new age' engineers. But these methods are just 'black boxes' to some and injudicious use can lead to serious flaws in the design and must be used with caution. Yet they are essential to develop an appreciation for deformation and stress trajectories in the detailed design phase.

Limit equilibrium (LE) methods, while not capable of providing detailed stress, strain and deformation output, fundamentally concentrate on block stability by ensuring that the forces are in equilibrium subject to the simplistic assessment of shear stress capacity usually modelled in terms of the Mohr-Coulomb failure criterion. LE is nothing else than the balance between demand, represented by mass and gravity, and capacity, represented by shear stress/force 'available' on a defined failure plane by virtue of its shear stress capacity integrated along the defined failure plane.

LE should not however be underestimated as a tool to fundamentally understand the nature and mechanisms of failure that prevail in a structure. The method requires that a failure mechanism or block model be defined inductively whereupon the full suite of applicable forces are assembled to represent the state of force equilibrium on the block. This is instructive in two aspects: i) it forces the user into inductive (as opposed to deductive) reasoning but asking the question "what could be the mechanism of failure" and ii) it requires the calculation of basic first principles forces and counter forces. The benefit of this thinking will be demonstrated in the section of Translational analysis below.

4.2 *The 'Excess Shear' Concept*

'Excess shear' is not a physically reality, but a numerical construct/concept that greatly enhances the understanding of the mechanics of a problem particularly when the problem involves plastic strain and the redistribution of stress in the structure. The state of stress cannot physically exist outside of the failure surface (Mohr Coulomb for example) and in reality, as the failure surface is approached (from below), stress and strain redistribution occurs which ensures that the stress state is maintained at least at the failure surface. In numerical analysis, however, the stress state at every point is first calculated and then checked against the failure state. If it is found to be within (below) the failure surface, then calculation proceeds. If a stress point is outside (above) the failure surface, then the 'excess shear', that measure that is above the failure surface, is redistributed to other points in the vicinity so that the overall stress state is maintained at least at the failure surface.

To investigate this concept further, consider a structure built from homogeneous material which is in equilibrium under all applied loads and stresses. Introduce an inclined plane into the structure which has the same capacity as the parent material. The equilibrium condition is not altered and the structure is stable. Now, progressively decrease the strength capacity of the plane until failure is reached – failure plane. The difference between the equilibrium and failed state is defined here as the "excess shear" in the system. "Excess shear" in the system contributes to the lack of 'factor of safety', but if the capacity is not available and redistribution to maintain equilibrium is not possible, then numerical (and physical) failure is precipitated (Howell, 1992)

There are a number of options that the designer can choose to deal with the problem:

- Include reinforcement in the system to contribute to reducing the excess shear;
- Reduce the overall potential energy/strain energy in the system by changing the geometry (shallower angles, reduced height, for example)

In the former case, added reinforcement in the modern geotechnical context includes the addition of geomembranes and geogrids that provide a tension component to improve shear strength (effective cohesion) and to 'consume' the excess shear. This is an important aspect since the tension that is generated must be sustainable by the geomembrane, remembering that the strain deformation of geomembranes at maximum stress is of the order of 100%, far above the strain development in the structure. Therefore, strain compatibility and strain limitation is necessary in the design, which in turn means limiting the stress in the element. This potentially is a complex design procedure and is not easily conceptualized or understood in modelling (with FE/FD), but must be considered in order to adequately account for the ultimate and serviceability limit states. LE formulations can assist greatly with visualization of this issue as will be considered in the next section.

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4.3 *Homogeneous materials*

To demonstrate the excess shear concept further, consider the states of stress in a structure (Figure 2). The M-C failure criterion is represented by the line $\tau = c_m + \sigma_n \tan(\varphi_m)$ where τ is the shear stress, σ_n is the normal stress and φ_m is the friction angle for the material (denoted by the subscript m). Note that any other relevant failure criterion other than M-C could be used as it does not change the fundamental concepts.

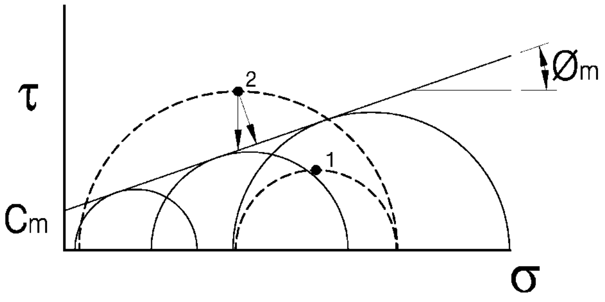


Figure 2: State of stress and M-C failure criterion

It is immediately obvious that the state of stress can either be inside the failure envelope (point 1) or outside (point 2). The stress state at point 2 is physically impossible as it lies outside of the failure envelope. But conceptually the vertical distance that the stress point is outside the failure envelope, is an indication of the state of 'excess shear' that exists and that needs to be accounted for, either by redistribution or by accommodation by other means, such as tension in the liner.

Integration of the effects over the continuum defines a failure zone that describes the failure surface in the structure and typically shown in Figure 3.

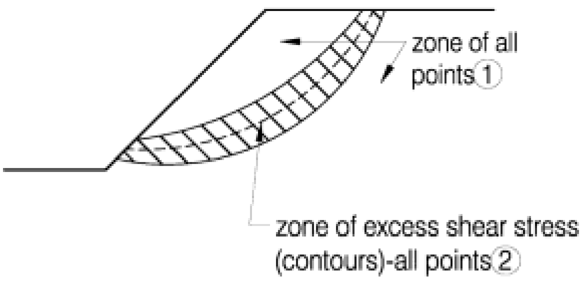


Figure 3: Failure zone in a continuum

4.4 *Constructed interface*

Install/construct into the continuum an inclusion that has strength parameters (φ_i, c_i) as represented in Figure 4 as an additional sub-horizontal potential failure surface.

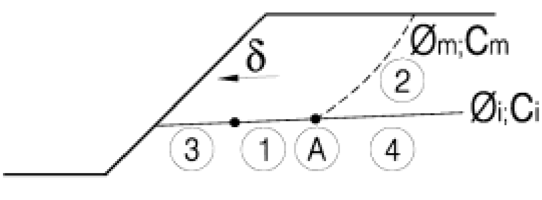


Figure 4: Zoned constructed continuum.

Clearly, whereas the stress state of the homogeneous slope was in equilibrium, the new constructed structure may have stress states that violate the failure conditions along the inclusion that lead to excess shear. The net effect of this is that shear deformation (or block deformation in Figure 4) will occur sympathetic with the excess shear component that exists in the system.

There are 4 identifiable zones in the block model (Figure 6b):

- Zone 1 represents an area where excess shear stress is manifest along the inclusion, leading to deformation and potential failure;
- Zone 2: due to strain deformation, the stress conditions in this zone are altered sufficiently for them to exceed the failure condition in the parent material, leading to the formation of a failure plane;
- Zone 3: similar to Zone 2, but the shear stress development is sympathetic with the mass movement along the inclusion;
- Zone 4 represents the area behind the failure plane where the stresses do not violate either that of the parent material (φm, cm) or the inclusion (φi, ci).

The effect of the zonal nature of the stress field subdivides the continuum into failure blocks in a piecewise linear fashion, where the excess shear stress is translated into extension strain (or tension) at the bifurcation point (A). This clearly demonstrates the dilemma of geometry that the engineer faces in this design conceptualization. Numerical analysis can easily be used to calculate the stresses and forces that develop in the structure and thereby define the block geometry, but the fundamental behavior that the engineer needs to anticipate is best demonstrated by a limit equilibrium approach using a hand or spreadsheet calculated translation analysis.

5 LIMIT EQUILIBRIUM: TRANSLATIONAL ANALYSIS

Qian et al (2002) defined a calculation procedure for translational failures using a piecewise linear limit equilibrium approach in landfill sites. This method has been adapted here to analyze block failures and the commensurate increase in liner tension in constructed structures. The translational (or two wedge) failure analysis was originally used to calculate the factor of safety against possible mass movement along a liner, but in this application, the objective is to reformulate the method to calculate tension in the liner directly. This method can, however, just as easily be used to analyze any generic slope stability problem including (and very specifically) low strength interface problems once the failure mechanism of the slope has been defined. For this purpose, the slope is divided into sectors/regions as shown in Figure 5 with reference to the zones defined in section 4.4 above.

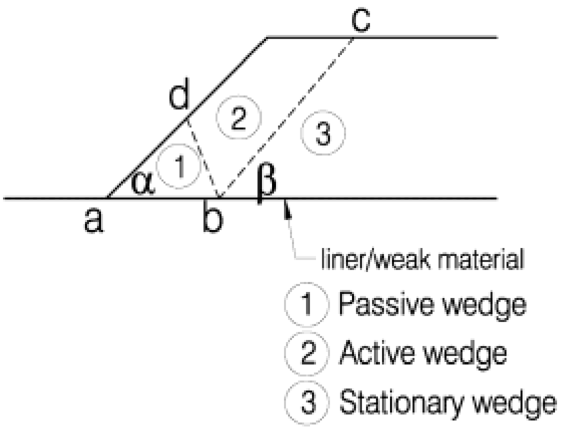


Figure 5: Block zones in a structure

In Figure 5, Sector 1 is called the passive wedge since it is acting to resist deformation, Sector 2 is the active wedge since it is subject to gravitational energy/movement in a lateral direction; and Sector 3 is the stationary wedge held in place by virtue of its location and force equilibrium.

Translational movement takes place when the active wedge (2) under gravity fails on the incipient planes bc and bd and drives the passive wedge laterally along ab. Along these planes, excess shear stress has been developed resulting in deformation which is counteracted by:

- Frictional strength on plane ab
- Internal friction on planes bc and bd
- Tension in the liner at b

Conceptually, the passive wedge is akin to the commonly observed heaving portion of the toe of a slope, the active wedge is the slump that takes place and points c and d are the observable manifestation of shear strain deformation that occur between the wedges on surface. Physically, planes bc and bd are zones of high shear strain/stress where failure takes place. While it can be shown numerically (FE) that the plane bd is inclined as shown in Figure 5, the LE mathematics (below) becomes too complicated for hand or spreadsheet calculation and hence bd is considered to be vertical for this purpose. Line bd is the interface along which equilibrium between the active and passive blocks is calculated. Taking the plane as vertical introduces some minor error, but the principles are still valid and instructive. The resulting model is shown in Figure 6

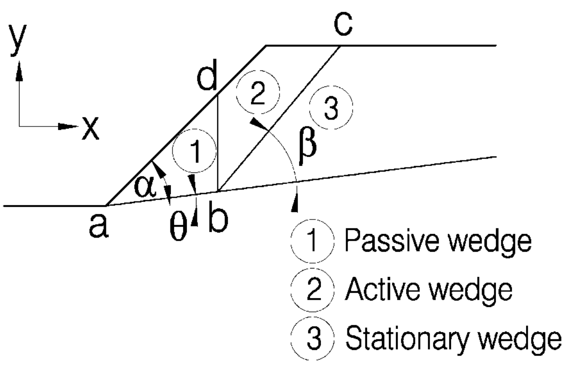
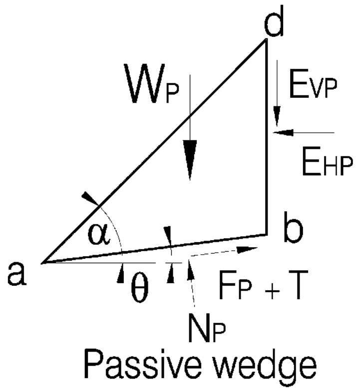
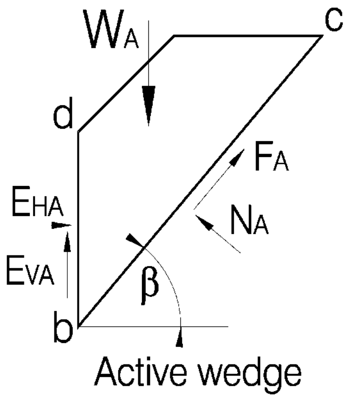


Figure 6: Components of force on Active and Passive Wedges.





With the objective of calculating the minimum T (tension in the liner) to maintain equilibrium, the static force equilibrium equations $\Sigma F_y = 0$ and $\Sigma Fx = 0$ can be derived in terms of the following parameters:

Passive Wedge:

W_p = weight of the passive wedge

N_p = normal force acting on the bottom of the passive wedge

F_p = limiting frictional force acting on the bottom of the passive wedge (subject to MC limit for the interface)

E_{HP} = normal force from active wedge acting on the passive wedge

E_{VP} = frictional force from active wedge on the side of the passive wedge

Φ_P = interface friction angle under the passive wedge

Φ_M = friction angle of the parent material

α = outer angle of the parent material slope

θ = angle of the liner/subgrade to horizontal

Active Wedge:

W_A = weight of the active wedge

N_A = normal force acting on the bottom of the active wedge

F_A = limiting frictional force acting on the bottom of the active wedge (subject to MC limit for parent material)

E_{HA} = normal force from passive wedge acting on the active wedge

E_{VA} = frictional force from passive wedge on the side of the active wedge

Φ_A = interface friction angle under the active wedge β = base angle of the active wedge to the horizontal

General:

W_T = total weight of the active and passive wedges

T = Tension in the liner

Considering the force equilibrium of the passive wedge for $\Sigma F_y = 0$:

$$W_P + E_{VP} = N_p \cos\theta + F_P \sin\theta \quad (1)$$
$$F_P = N_P \tan\emptyset_P + T \sin\theta \quad (2)$$
$$E_{VP} = E_{HP} \tan\emptyset_M \quad (3)$$

When $\Sigma Fx = 0$:

$$F_P \cos\theta + T\cos\theta = E_{HP} + N_P \sin\theta \quad (4)$$

Considering the force equilibrium of the active wedge for $\Sigma F_y = 0$:

$$W_A = F_A \sin\beta + N_A \cos\beta + E_{VA} \quad (5)$$
$$F_A = N_A \tan\emptyset_A \quad (6)$$
$$E_{VA} = E_{HA} \tan\emptyset_M \quad (7)$$

When $\Sigma Fx = 0$:

$$F_A \cos\beta + E_{HA} = N_A \sin\beta \quad (8)$$

Equation (1) to (8) together with the equality $E_{HP} = E_{HA}$ produces a complex expression for T in terms of the known parameters above. The expression, however, for T is best solved using a spreadsheet. This allows the parameters to be tested and a coherent design achieved from the treatment. The fundamental treatment from very basic principles (in this case, static equilibrium) is very powerful in gaining an in-depth understanding of the engineering processes that are involved.

Cohesion and the size of a toe berm to provide additional passive pressure and so render the liner tension to zero can also be simply include by adding a stabilizing term to equation 4.

The tension in the liner is due to the inability of the interfaces to sustain the excess shear stress (or force) that the system, driven by gravity, imposes on it. The interaction between shear capacity along the failure planes and interfaces and the resultant need for reinforcement (in this example, the liner strength) is clearly demonstrated in this treatment.

The corollary is that other more esoteric design calculations can be carried out quickly and efficiently using the spreadsheet. These include, but are not limited to, the assessment of the height of the structure at the material's normal angle of repose (cascaded stockpile) subject to zero tension in the liner; the stacking angle for a given height for zero tension; the height and/or stacking angle for a predefined tension in the liner commensurate with an specified target strain, the relationship of the base/subgrade angle to liner tension; or the calculation of the required tensile reinforcement (geogrid) required for a given set of parameters.

One question that remains is the inclination of the base angle (failure plane, bc) under the active wedge (denoted by β) that gives rise the minimum release energy within the system. Intuitively, using Rankine theort, the angle should be in the range of $45+\varphi_m/2$. To test this intuition, a simple finite element analysis has been performed as shown in the following section.

6 FINITE ELEMENT ASSESSMENT

Figure 7 shows the results of a finite element analysis using Phase[2].

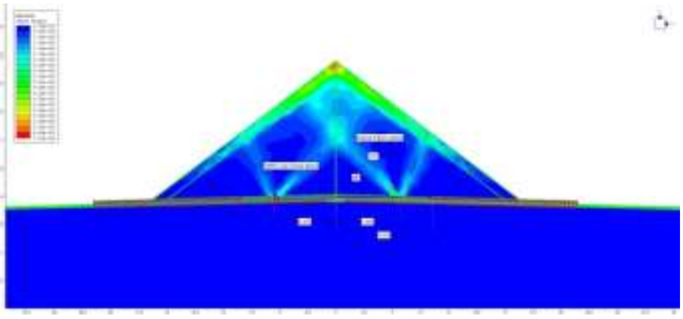


Figure 7: Maximum shear strains for stockpile

The figure shows the zones of maximum shear strain (light coloured diagonal zones) that demarcate the development of failure planes within the stockpile parent material. The mass is shown divided into the passive wedge (Sector 1: lower left), the active wedge (Sector 2: upper material) and the stationary wedge (Sector 3: bottom centre). This example is slightly more complicated than the two wedge LE translational analysis given in Section 5 since the wedges form to the left and right in symmetrical format, but with a little imagination the applicability of the 2 wedge analogy is clearly evident.

The stockpile material friction angle is 37° and that of the interface is 10° in this example. In this case, again with reference to the annotations in Figure 6, the diagonal line bd (line of equilibrium) is inclined at 68° while the base of the active wedge above the stationary wedge is inclined at 47° to the horizontal. From a translational analysis calculation perspective then a good initial approximation for the base angle would be 50° or $45+\varphi_i/2$.

7 CONCLUSIONS

The objective of this paper has been to highlight the intrinsic weakness properties of natural and manufactured interfaces and to develop a fundamental understanding of the nature of the physical mechanisms playing out within a structure supported on such interfaces. In particular, an understanding of the development of tension is a liner is shown by the judicious use of a simple limit equilibrium approach that provides a versatile method for calculation not only liner tension but also other aspects that are required in a design.

The lessons learnt from this treatise are the following:

- Fundamental understanding of the behavior of the interface materials are required to produce a competent design.
- Physical shear testing of the actual samples of the liner system including the soils should be carried out.
- Develop a conceptual model that interprets the essence of the problem physically, that is 'define the problem' both fundamentally and numerically.
- Develop a hand/spreadsheet calculation model that describes the mechanics of the problem and test the design variables accordingly.
- Only once the physical mechanics of the design are fundamentally understood, then resort to more sophisticated numerical modeling.

Moreover, it is vital that a coherent design strategy or philosophy is developed for any problem to be solved. This naturally develops from a fundamental familiarity with the mechanics thereof.

8 REFERENCES

Akber. S.Z.. Hammamji, Y. and Lafleuer, J. (1985). Frictional Characteristics of Geomembranes and Geomembrane-Geotextile Composites, *Proc. Second Canadian Symposium on Geotextiles and Geomembranes, Edmonton, Alberta*, 209-217

Ali F, Salman FA, Subramanaim (2012), Influence of surface texture on the Interfaces Shear Capacity of Landfill Liner *Electronic Journal of Geotechnical Engineering (EJGE)* Vol 17 2012

Ali J, Chen JF, Rotter JM and Ooi JY (2009), Finite Element Prediction of Progressively Formed Conical Stockpiles *Simulia Customer Conference 2009*

Bhatia S and Kasturi G (circa 1996), Comparison of PVC and HDPE Geomembranes (Interface Friction Performance), *Research Report, Dept of Civil and Environmental Engineering, Syracuse University*, www.geomembrane.com/userfiles/filemanager/451/

Blond E and Elie G (2012), Interface Shear-strength Properties of Textured Polyethylene Geomembranes http://www.solmax.com/wp-content/uploads/2012/08/1_ShearStrenghtproperties-PEmembrane.pdf

Effendi R (2011), Interface Frciton of Smotth Geomembrane and Ottawa Sand *INFO TEKNIK* Vol 12 No 1 July 2011

Gourc JP, Reyes-Ramirez R and Villard P (2004) Assessment of Geosynthetics Interface Friction for Slope Barriers of Landfill www.geosynthetica.net/Uploads/GeoAsia04Gourc.pdf

Independent Expert Investigation and Review Report (2015), Report on Mount Polley tailings Storage Facility Breach , January 2015, www.mountpolleyreviewpanel.ca

Howell GC and Kirsten HAD (1992), Kinematic Indeterminacy as basis for the determination of optimum mining procedures *Rock Support in Mining and Underground Construction*, Kaiser and McCreath (eds) 1992, Balkema.

Koerner, G.R. and Narejo, D (2005)., Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces, *GRI Report* # 30, 2005.

Krahn J (2003) The 2001 R.M. Hardy Lecture: The Limits of Limit Equilibrium Analysis" *Can Geotech J* Vol 40 pp 643-660 (2003)

Lupo JF, Morrison KF (2008) Innovative Geosyntheric Liner Design Approaches ad Construction in the Mining Industry *Geo-Frontiers Congress* 2005

Martin. J.P.. Koerner, R.M. and Whitty, J.E. (1984), Experimental Friction Evaluation of Slippage Between Geomembranes, Geotextiles and Soils, Proceedings of the Intl. Conference on Geornernbranes, Denver, Colorado, pp 191 – 196

Narejo and Allen (2009) Final Report Evaluation of Geosynthetic Clay Liners as a Substitute for Compacted Clay Liners in Large Impoundments, *Report for ConocoPhillips by TRI Environmental, Inc*., 2009.

PHASE^2 Finite Element Software for the Analysis of 2D Continua, Rocscience, Canada.

Qian X, Koerner RM and Gray DH (2002), Ch 13 *Landfill Stability Analysis, in Geotechnical Aspects of Landfill Design and Construction*, Prentice Hall, 2002

SLIDE Limit Equilibrium Software, Rocscience, Canada

Vaid YP and Rinne N (1995), Geomembrane Coefficients of Interface Friction *Geosynthetics International* , Vol2 No 1 pp 309-325

Williams, N.D. and Houlihan, M.F. (1987), Evaluation of Interface Frictional Properties Between Geosynthetics and Soil, *Proc. Geosynthetics '87*, New Orleans, USA, pp 616-627.

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