A Decade of Landfill Capping Experience

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ABSTRACT: Current regulations in the United States require the use of a barrier to be placed over lined landfills to limit the infiltration of surface water to the waste and control the gas emissions from the waste. This barrier must be protected in a manner that ensures an adequate service life of the barrier. This paper reviews the design considerations and observed performance of these final cover systems over the past decade. Additionally, the paper summarizes existing concerns and potential new requirements for landfill caps in the next decade.

KEYWORDS: Landfill, Final Cover, Cap, Barrier, Failure, Field Observations

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Introduction

Current Federal regulations for hazardous waste landfills (CFR § 260) and municipal landfills (CFR § 258) necessitate the final cover of a landfill with a composite liner to include a composite barrier that consists of a geomembrane over an earthen barrier layer. The requirements for municipal landfills were essentially adopted from similar requirements found in the hazardous waste regulations. This includes the use of a composite barrier in the final cover and some form of vegetative or erosion control layer. The composite barrier requirement for municipal landfill covers was clarified by EPA, June 26, 1992 Federal Register, as meaning the use of a geomembrane over 18 inches of a soil having a permeability of less than 1×10^{-5} cm/sec. This was a significant back step from the use of 10^{-7} cm/sec clays common to hazardous waste landfill final covers. However, most State regulators have approached the municipal solid waste (MSW) landfill covers in the same manner that they would final covers used in hazardous waste landfills.

Landfill cap systems must provide, among others, two primary functions: (1) minimize the infiltration of surface water to the waste and control potential landfill gas (LFG) emissions from the waste using a barrier(s); and (2) protect the barrier during its service life using an erosion resistant protective system. This paper reviews the engineering design approach to achieve these two functions, and presents a summary of the performance of existing caps regarding these functions. Specific cap failures are presented to highlight shortcomings of some current cap designs. The paper concludes with a summary of current technical areas of concern and a projection of what caps may look like over the next decade.

Barrier Function

All current regulations require that the permeability of the cap (or rate of surface water infiltration) be less than the permeability of the barrier system (or rate of leachate leakage). This is intended to eliminate what is referred to as the "bathtub" effect i.e., to prevent the containment system from filling up with leachate once active operations are terminated. Thus, in theory, the cap can be designed in conjunction with the actual landfill barrier system used. Such flexibility in cap design is provided for in CFR § 258 (Subtitle D) for municipal solid waste landfills. Thus, the designer of caps for municipal solid waste landfills has considerable flexibility in how the cap is designed to limit surface water infiltration.

Barrier - Water Infiltration

Barriers to limit surface water infiltration are generally constructed using geomembranes (GM), geosynthetic clay liners (GCL), compacted clay layers (CCL) and combinations thereof. In arid and semi-arid regions of the USA, caps that limit surface water infiltration using evapotranspiration, e.g. water-balance, mechanisms can be designed but are beyond the scope of this paper. The technical evaluation of the common barrier systems is performed as follows:

Geomembrane Barrier Only ---- The movement of water through a geomembrane occurs both by diffusion of moisture through the polymer and from flow through defects. Diffusion occurs at a very slow rate. For instance, a 60-mil thick HDPE barrier (k= 2 E-13 cm/sec) with water standing on it will allow approximately 0.04 gallon/acre/day to diffuse through the geomembrane. Significant flow through a geomembrane results from penetrations in the membrane. The flow through a single hole in a geomembrane placed over a porous media is given by (Giroud et al. 1989)

$$Q\ (\text{m}^3/\text{sec}) = 2.66\ a\ h^{0.5}$$

where a is the area (m^2) of the hole and h is the head (m) acting on the geomembrane. For example, given a single hole having an area of 1 cm^2 acted on by a head of 30 cm, the flow through the hole is 2.66 x .0001 x $.3^{0.5}$ or 0.000145m^3 / sec. Using a conversion factor of 264 gal/m^3, this is approximately 3300 gallons/day passing through the hole.

Compacted Clay Barrier ---- The flow of water through a clay barrier, Q, is approximated by Darcy's Law as

$$Q = K\,i\,A$$

where K is the saturated hydraulic conductivity of the compacted clay, i is the flow gradient, and A is the area. This equation is rewritten as

$$Q = K\,[(h+t)/t]A$$

where h is the height of water standing on the barrier and t is the thickness of the barrier. For most soil barrier systems the vertical flow gradient is in the range of 1.5 to 2.0. Assuming K = 1 x 10^{-7} cm/sec, h = 30 cm, and t = 60 cm, the flow of water through a standard two foot thick soil barrier would be approximately 138 gallons/acre/day.

Geomembrane plus CCL ---- A geomembrane overlying a compacted clay barrier (CCL) forms a composite barrier that has advantages over the use of either a GM or CCL individually. The flow through a penetration in the geomembrane is reduced by the presence of the clay and is calculated as follows (Giroud et al. 1989):

$$Q = 0.21\, h^{0.9}\, a^{0.1}\, K^{0.74}$$

where h is the height of water standing on the geomembrane (m), a is the area of the hole (m^2), and K is the permeability of the underlying clay (m/sec). Again using a 1 cm^2 hole, h of 30 cm, and K of 1 x 10^{-7} cm/sec, the flow through the composite barrier can be calculated to equal 0.15 gallon/acre/day.

Geomembrane plus GCL ---- Substituting a geosynthetic clay barrier (GCL) for the CCL the leakage through the composite is calculated by the following, Giroud, 1997:

$$Q = 0.21\, i_{avgo}\, a^{0.1}\, h^{0.9}\, K^{0.74}$$

where: i_{avgo} = average hydraulic gradient

$$= 1+0.1\left(\frac{h}{t_s}\right)^{0.95}$$

a = Area of the defect (m^2)
h = Head acting on the barrier (m)
K = Permeability of the GCL (m/s)
t_s = Thickness of GCL (m).

Most commercial GCLs use a bentonite that develops a permeability of less than 5.3 x 10^{-9} cm/sec in the GRI-GCL-2 test. This test uses an effective confining stress of only 10 psi. Actual confining stresses acting on a GCL in the final cover will be smaller than this and result in slightly higher GCL permeability. The flow through a 1 cm^2 round hole having 30 cm of water standing on it can be calculated to be approximately 0.1 gallons/day assuming a 5 mm (0.2 inches) thick GCL.

Barrier - LFG Transmission

Under the authority of the Clean Air Act, EPA issued New Source Performance Standards (NSPS) and Emissions Guidelines for MSW landfills. These regulations require that large MSW landfills that emit LFG in excess of 50 megagrams (Mg) per year to control such emissions. For the type and size of municipal solid waste landfills being designed today, Federal and most State regulations require that active landfill gas extraction systems be used to control LFG emissions. For these designs, the hydraulic barrier is not required to prevent gas emission as the waste mass will be subjected to vacuum pressures. The barrier then will prevent air intrusion into the landfill and its effectiveness has to be factored into the design of LFG extraction systems. The evaluation of common barrier systems for air entry characteristics is performed as follows:

Compacted Soil Barrier ---- The air permeability of a porous media, i.e. soil, can be estimated for granular soils based on the principle of "intrinsic permeability." A summary of this was recently presented by Thiel, 1999 and showed that the ratio between the water and air permeability of a granular soil can be expressed as follows:

$$\frac{k_1}{k_2} = \frac{\mu_2}{\mu_1} \times \frac{\gamma_1}{\gamma_2}$$

where k is permeability, μ is dynamic viscosity, and γ is the unit weight of the respective "fluid." Solving this equation for water and air, the permeability of a porous media to air is approximately equal to 14 times the permeability of water in that media. This relationship does not hold for silts and clays since fluid polarity and electro-osmotic potentials dominate over gravitational forces in these soils. However, the ratio of 10 would be conservative for such soils because it does neglect the additional restraints to flow.

The flow of air through the cover adjacent to an active gas recovery well under steady state conditions is given by (Radian Corporation, 1989)

$$V_{air} = k_{cover} (P_{atm}\text{-}P_i)/ u_{air} D_{cover} \; = k_{refuse,v,} \; (P_i - P_v)/\mu_{air} x$$

where

V_{air} = air velocity through cover and refuse, m/sec
k_{cover} = intrinsic cover permeability, m2
P_{atm} = atmospheric pressure, Newtons/m2
P_i = interface pressure, Newtons/m2
u_{air} = air viscocity, Newton-sec/m2
D_{cover} = cover thickness, m
$k_{refuse,v}$ = intrinsic vertical refuse permeability,m2
P_v = vacuum pressure in the well, Newtons/m2
x = length of solid pipe in the well, m

Brooks and Corey 1964 also showed that presence of water in the pores of the soil reduces its permeability to gas. This finding can be used to estimate changes in the "intrinsic" permeability of the soil from "dry" state to "moist" state and employed for designing the gas extraction system.

Geosynthetic Clay Liner ---- Gas transmission properties of a GCL have been investigated by Trauger, et al. 1995 and Aubertin, et al. 1999. Trauger measured the rate of diffusion of methane through a hydrated GCL suggests that for LFG having 55% methane, the flow rate through the GCL will be 5.4 to 8.0 x 10^{-10} $m^3/m^2/s$. This same study showed that the flow could increase 3 orders-of-magnitude as the moisture content of the GCL decreased below saturation. Aubertin measured the flow rate of oxygen through a saturated GCL and determined the effective diffusion coefficient, D_e, to be approximately 5.5 x 10^{-11} m^2/s. This study observed that D_e was increased by 6 orders-of-magnitude at lower water contents of the GCL. Based on the above studies, the "intrinsic" permeability for "dry" and "moist" conditions of the GCL can be estimated and used to design gas extraction systems.

Flow Through Barriers - Effect of Geometry and Drainage

The discussions presented earlier on the flow of water through the hydraulic barrier assumed a hypothetical hydraulic head of 1.0 ft (30 cm). Under actual conditions, the head on the barrier will be a function of such designer-controlled factors as:

- Final cover slopes which will affect both surface runoff and the ratio of lateral to vertical flow at the barrier's upper interface.

- Surface soils and vegetation which will affect both surface runoff and the evapotranspiration.

- Permeability of the soil directly above the barrier.

Steep final cover slopes will reduce the quantity of water flowing through the barrier as both increased runoff (hence reduced infiltration) and increased lateral drainage above the barrier will cause the hydraulic head on the barrier to be decreased. Similarly, higher field capacity and lower wilting point of the cover soils and excellent vegetation will all increase evapotranspiration and thus reduce infiltration, again leading to reduced hydraulic head on the barrier.

Steep slopes are often cause of reduced slope stability and even failure of the veneer of protective soils above the barrier. This aspect is addressed in a later section of this paper. Thick vegetation needs enhanced maintenance practices, one aspect often lacking in non-revenue producing facilities. Thus, the designer has a challenge to strike a delicate balance between the factors discussed above to minimize the infiltration into the waste mass.

Barrier Protection

Each of the barrier systems described above must be protected from detrimental environmental impacts over their service life. Exposed geomembrane covers have been used by US Nuclear Regulatory Commission for the past decade, see Photo 1, and have recently been adapted for interim closure of MSW landfills, Gleason et al. 1998.

However, their use is not considered appropriate for final closures. This means that final closures will require an earthen protective layer be placed over the barrier system. This earthen barrier can be constructed either as an armored layer using rip-rap or as a vegetated soil layer. The barrier protection layer must be designed to be erosion resistant and stable over the service life.

Barrier Protection - Erosion Resistance

An understanding of predictive methods for soil loss provides the designer a means of predicting the rate of soil loss and a basis for understanding the role that various erosion control products and field techniques play in limiting soil loss. The most common method of estimating soil loss is the USDA Universal Soil Loss Equation (USLE, USDA, 1965,1978) a nationally accepted equation integrating the research work done during 1935 to 1965 in response to the great dust storms of that time. This equation has been modified to account for several factors and a revised equation (RUSLE) has been published (Renard et al. 1993). USLE or its revised version RUSLE continues to be the primary tool in service and does provide a simple vehicle for a discussion of erosion control basics. The USLE is expressed as follows:

$$X = RKSLCP$$

where X is the annual soil loss in tons per acre, R is a rainfall erosion index that reflects the erosion potential of the precipitation of the region, K is the soil erodibility factor, S is the slope gradient factor, L is the slope length factor, C is a crop or vegetation factor, and P is an erosion control practice factor that reflects the facility's maintenance activities.

Based on USLE, the Civil Engineer has traditionally attempted to limited erosion of landfill final covers to less than 2 tons/acre/year by minimizing several of the variables as follows:

Soil Erodibility, K**:** The ability of a soil to resist erosion increases with both particle size and soil plasticity. Thus, silts are the most erodible of soils since they lack both particle size and soil plasticity. The use of armoring techniques such as rip-rap is an obvious attempt to dramatically decrease K.

Slope, S**:** Soil loss can be limited by requiring initial slopes to be lower than some nominal amount, e.g., 8% for hazardous waste landfills. This practice, however, may not be economical in MSW landfill cover slopes designed to maximize airspace. In addition, often these wastes have a high potential of future subsidence and steeper slopes minimize cover performance concerns related to settlement. It must be noted that steeper slopes result in dramatically increased erosion, mainly due to rilling.

Slope Length, L**:** Landfill designers have long been aware that erosion on long side slopes is significantly greater than predicted from standard test plots. The increased erosion is a function of rill to interrill ratios dependent on the slope gradient. For this reason, the side slopes are reduced in "erosion length" by placing drainage swales or terraces/benches at 25 to 40-ft vertical spacing. This typically reduces the effective slope length to less than

120-ft. Water collected in such swales must then be brought down the slope in some form of engineered conveyances.

Crop Management Factor, *C*: To a Civil Engineer this means grass! This variable can range from a value of 0.45 for no ground cover to as low as 0.003 for 100% coverage of grass. Many erosion problems vanish on paper through manipulation of this factor. Unfortunately, the field steps required to develop and maintain a healthy, robust, and high percent ground cover are rarely implemented on final covers.

Erosion Control Practice, *P*: Lacking clear guidance and experience as to how landfill maintenance relates to agricultural or botanical practices, this is commonly assumed to be in the range of 0.9 to 1.0.

Geosynthetic erosion control products can be used to increase the long-term erosion resistance of an engineered feature, e.g., lining a down chute, or providing a short-term role until a self-sustaining vegetative cover is established. The service life or "functional longevity" of the application is important to consider during the selection of an erosion control material since the field life of UV protected products is significantly greater than that of natural products. Non-UV stabilized geosynthetics may have field lives comparable to the natural materials.

Erosion Failure of a Barrier Protective Layer ---- Large-scale erosion of final cover systems for landfill slopes is most likely to occur during the construction of the granular drainage layer. Since the runoff coefficient is initially smaller for permeable granular materials, the sand drainage layer may quickly saturate, see example below, and have nearly 100% runoff. Typical data input to this equation for a 25% slope on a landfill in the mid-west is as follows:

> *A* = average annual soil loss, in tons/acre
> *R* = rainfall and runoff erosivity index (R= 200, typical midwest)
> *K* = soil erodibility factor, tons/acre (K=0.45 sand)
> *LS* = slope-length/steepness factor (LS = 5.8 for 25% slope 100-ft long)
> *C* = cover-management factor (C=1.0)
> *P* = erosion control practice factor (P=1.0)

This predicts a potential annual soil loss of 522 tons/acre/year for the exposed sand drainage layer. This indicates a significant soil loss potential due to surface erosion for the exposed sand. 522 tons/acre/year translates into uniform erosion of more than 2.5 inches. Considering that rill and gully erosion will occur during construction, it is conceivable that in many areas the sand could be completely eroded from the membrane surface. Large scale erosion during construction of the granular material over the barrier and even that of the rooting layer before establishment of vegetation have often been classified in the past as slope failures. However, increasing the shear resistance along weak planes in the veneer of the cover soils is not a defensive measure against such erosion related slope failures. Once the soil layer placed over the lateral drainage sand is fully vegetated, the soil erodibility factor will reduce to less than 0.006. The resulting soil loss will reduce to less than 7 tons/acre/year thus controlling the slope erosion failure.

Barrier Protection - Slope Stability

The stability of the final cover on slopes is influenced by the interface shear strength and the pore gas or water pressures that develop between the layers of the final cover system. Pore gas pressures are not a concern in landfills with operational active gas collection systems as assumed in this paper. This section reviews the evaluation of cap veneer stability, the design of lateral drainage systems to reduce pore water pressures, the use of materials to increase interface friction, and past stability failures of cover systems.

Slope Stability ---- The stability of final cover systems is typically evaluated using an infinite slope model. This is conservative since the individual layers within the cover profile are very thin in comparison to their aerial dimensions and additional restraints such as toe buttressing are neglected. Neglecting seismic forces, the stability of the slope can be conservatively evaluated using an infinite slope model defined by the following equation:

$$FS = \frac{c/(\gamma \times z \times \cos^2 \beta) + \tan\phi\left[1 - u/(\gamma \times z)\right]}{\tan\beta}$$

where FS = factor of safety, γ =unit weight of barrier protective material(s), c = cohesion, ϕ = angle of friction of the assumed failure interface or surface, z = vertical depth to the assumed failure interface or surface, u is the pore water pressure above the barrier <u>or</u> the gas pressure below the barrier, and β is the slope angle. Again, this model neglects potential toe buttressing forces and the tensile strengths of the geosynthetic components.

Caution should be exercised in defining the interface shear strength under the low normal loads typical of caps. Previous field and laboratory work by Giroud, et al. (1990) has shown that the interface friction between a geomembrane and a geotextile at low normal loads can be misinterpreted from direct shear tests data performed at higher normal loads. Giroud's data from both textures and smooth HDPE geomembranes, geonets, and geotextiles demonstrates that an apparent cohesive component of the interface shear strength actually disappears if data is accurately obtained under the low normal loads. The frictional component, however, does increase at the low normal loads. Design values of shear strength should be obtained under normal loads that replicate field conditions.

Neglecting cohesion, the slope stability can then be expressed as follows:

$$FS = \frac{\tan\phi\left[1 - u/(\gamma \times z)\right]}{\tan\beta}$$

This equation is shown on Figure 1 for the 4H:1V slope common to municipal landfill final covers, a factor of safety of 1, and a range of interface friction, soil cover weights, and excess pore water pressures.

Lateral Drainage System ---- The design of the pore water pressure drain underlying a saturated cover soil layer was first presented by Paruvakat et al. 1990 and Thiel and Stewart 1993. Both approaches are practically the same with the pore water pressure on the potential failure plane being assumed as triangular between two successive drainage outlets. The assumptions regarding the pore pressure magnitudes are different, however. It must be realized that because of the variables involved, the magnitude of pore water pressure cannot be assessed with sufficient accuracy. These variables include storm intensity and duration, degree of saturation of the cover soils at the time of precipitation, runoff, evapotranspiration, hydraulic properties of vegetative and drainage layers, cover geometry, efficiency of the drainage outlet, etc. HELP model or similar water balance models may provide reasonable flow through the drainage layer for design purposes if the design storms are conservatively chosen. Designers must provide adequate drainage capacity above the barrier layer. This is particularly important if the slopes are steeper than 20 percent, and if the interface between the barrier layer and the drainage layer is the weakest plane in terms of frictional characteristics.

Assuming that the design storm lasts long enough to saturate the vegetative layer, the quantity of water, Q_{in}, infiltrating into a unit width of drainage composite having a length L and slope β is given by

$$Q_{in} = k_{veg} L\ 1 \cos\beta$$

The flow capacity of a drainage layer is solved for using Darcy's Law as follows:

$$Q_{out} = kiA = ki\,(tx1) = [kt]\,i$$

where t is the thickness of the drainage layer, i is the flow gradient, and $[kt]$ is transmissivity, Ψ. The transmissivity of a geocomposite drainage layer is obtained from laboratory testing. It is important that Ψ be obtained at normal stress levels, boundary conditions, and gradients that reflect actual field conditions. Additional reduction factors for creep deformation of the drainage core, biological clogging of the geotextile, etc. may also be considered. Thiel and Stewart's model is limited to slopes steeper than 20%. For flatter slopes, the model developed by McEnroe (1993) should be used. It should be noted, however, that the simple model developed by Thiel and Stewart is conservative at the flatter slopes and therefore could be used in design of such slopes.

A factor of safety for the drainage capacity, FS_{dc}, of the geocomposite drainage layer can be defined as follows

$$FS_{dc} = Q_{out} / Q_{in} = (\Psi I) / (k_{veg} L \cos\beta)$$

The authors recommend a minimum factor of safety of 8 (overall drainage safety factor plus reduction factors) for lateral drainage systems in final covers. It is important to understand the impact of both Y and L on the hydraulic factor of safety. Design implementation of this equation is typically integrated into the side slope swale systems commonly used to limit surface erosion and slope length L, see Thiesen and Richardson, 1998. The geocomposite drainage layer is designed to drain into each swale. To ensure that *L* will be defined by the actual spacing of the swales, the drainage geocomposite must not be continuous across a swale.

The typical range of required transmissivity is easy to estimate based on typical slopes of 4H:1V to 3H:1V, a vegetative soil permeability of 1 x 10^{-4} cm/sec, and a vertical spacing of swales from 8 to 12 meters (25 to 40-ft.), and an assumed long-term service reduction factor of 8.0. Using these assumptions, the required transmissivity ranges from 6.0 x 10^{-4} to 1.6 x 10^{-3} m^3/sec-m. This transmissivity serves as a good rule-of-thumb for designers to use to check their values. Note that the required transmissivity is directly proportional to the permeability of the vegetative cover soil. When a geocomposite drain is designed based on this "maximum inflow criteria", the maximum head over the underlying liner is less than the thickness of the geocomposite.

Construction of Landfill Caps

Soils are placed over geomembranes in landfill liners and covers as drainage layers or protective soil layers. These soil layers typically are 1-2 feet in thickness. Low ground pressure bulldozers are commonly used to push these soils upslope to form a layer of uniform thickness. The use of low ground pressure (LGP) bulldozers eliminates excessive puncture stresses in the geomembrane. However, an equally or even more important aspect is the shear stresses that will be caused along the interfaces of the geomembrane. These interfaces are the weakest planes in the system and hence the potential failure surfaces. If the geomembrane's lower interface has less shear resistance than its upper interface, then the geomembrane may be subjected to tensile stresses. Tensile stresses will occur if localized slip occurs or if the displacement required to mobilize shear resistance along the lower interface is significant. The potential for localized slip will be the maximum during construction.

In view of the desirability to avoid stressing the geomembrane, there is a recognized need to evaluate the stress conditions during construction. Current methods to evaluate the effect of construction equipment on the stability of landfill liners and final covers do not recognize the local over-stressing possibilities because only the overall stability of the complete slope length is determined. Use of the weight of the construction equipment alone does not practically impact the computed factor of safety since the increased driving force is accompanied by an increase in the normal stress and a corresponding increase in the resisting force. Also, an arbitrarily assumed braking force is generally considered to be distributed over the whole slope length and thus results in very small effect on the computed factor of safety.

The potential for very large local interface shear stresses exists when an overzealous equipment operator tries to push too much material upslope at one time. If the resulting shear stresses below the geomembrane exceed the interface shear strength, localized slipping will result. When this occurs, the geomembrane will be subjected to tensile stresses. Geomembranes such as polyvinyl chloride (PVC) or linear low density polyethylene (LLDPE) are flexible (i.e., low modulus and large failure strains) and likely to be stretched upslope of the location of cover soil placement. Wrinkles will be formed directly under the equipment. Ultimately the tensile strains in the upper reaches of the slope could reach failure strains and cause tearing of the membrane. The unfortunate part of this scenario is that most construction quality assurance monitors are not likely to recognize this situation until it is too late. They usually focus on the soil thickness beneath the tracks and look for evidence of geosynthetic damage below the blade or the tracks. Paruvakat and Sturzl, 1998 developed an approach to recognize potential local overstressing of membranes under the cover soils.

Analysis of Construction Forces

The shear stresses transferred to the geosynthetic interfaces by the tracks of the low ground pressure bulldozer during cover soil placement over geomembrane are illustrated in Figure 2 (Paruvakat and Sturzl, 1998). The total shear force transferred by the tracks to the soil below must be equal to the passive pressure on the blade of the bulldozer for equilibrium. So, if the operator of the equipment were to push a high wall of soil ahead of him, instead of shaving off the soil pile gradually, the passive pressures could exceed the interface strength along the potential failure surface and failure of the geomembrane could result.

The factor of safety can be expressed as follows:

$$\textit{Factor of Safety During Construction} = \frac{(\gamma z + \sigma)\ \cos\beta \tan\phi}{(\gamma z + \sigma)\ \sin\beta + s}$$

where γ is the unit weight of the soil being placed, z is the thickness of the soil layer below the equipment track, and s is the shear stress transmitted to the interface due to the passive resistance of the soil being pushed.

The total force generated by the bulldozer, P, can be assumed to equal the passive resistance of the soil being pushed as follows:

$$P = 2\ \gamma h^2\, B\, K_p \cos\phi$$

where B is the width of the bulldozer blade, h is the height of soil being pushed, and ϕ is the friction angle between the blade and the soil being pushed . The actual stress transferred to the underlying geomembrane, s, is influenced by bulldozer=s area of contact, A, with the geomembrane.

The geomembrane contact area is a function of the thickness of the soil layer being placed and the vertical stress distribution of the track load through that layer. The vertical stress distribution effect is approximated by assuming a 1(horizontal):2(vertical) distribution of the vertical stresses generated by the bulldozer. This leads to a reduction factor for vertical stress distribution, c, defined as

$$c = (l \bullet b) / (l+1)(b+1)$$

where l and b are the length and width of the bulldozer's tracks. Assuming a typical soil placement thickness of 1-foot, the above equations can be solved for the maximum height of soil that can be pushed with a factor of safety equal to one:

$$h = \sqrt{\frac{4\,bl\,(\gamma\,z + \sigma)\,(\cos\beta\tan\phi - \sin\beta)}{c\,\gamma\,B\,k_p\cos\delta}}$$

where σ is the vertical stress at the interface due to the weight of the bulldozer, B is the width of the bulldozer's blade, δ is the angle of friction between the blade and the drainage sand. Using the above relationship, the maximum thickness of cover soil that can be pushed by a bulldozer before the occurrence of localized failure along the interfaces of liner or cover systems can be estimated. It is apparent that the critical thickness, h, is a function of the equipment characteristics (B, b, l, s), soil unit weight (γ), slope angle (β), interface strength (ϕ), and cover soil strength (k_p, δ). These variables are shown on Figure 2.

Typical Application ---- Figure 3 shows the maximum height of soil that can be safely pushed by a CAT D3B LGP bulldozer for an interface having a friction angle of 18° and a range of slope and soil properties. Note that the allowable "push" height decreases with increasing slope and increasing soil shear strength ($K_p$cos). Interestingly, the critical blade height on typical side slopes equal to or steeper than 25% is actually less than the height of a typical bulldozer blade! Thus, even limiting the height of soil being pushed to the dozer blade may lead to construction stresses that produce tension in the geomembrane. For a common 25% side slope, Figure 4 shows that the critical soil push-height increases with increasing weight of the bulldozer; unfortunately so does the dozer blade height. For typical slope conditions, only approximately half of the blade should be used for pushing soil.

<u>Case Histories</u>

Case One: Stability Failure During Construction

In the fall of 1996, during construction of a 12-acre final cover in northern Michigan. The geomembrane and an overlying sand pore pressure relief layer slid off of a 4H:1V final cover side slope at several locations, Photo 2. The completed final cover was to include the following layers (bottom-to-top): a sand gas collection layer, a composite barrier formed by a PVC geomembrane over a reinforced GCL, an 18-inch thick sand drainage layer (γ=107 pcf), and a 12-inch topsoil/vegetative support layer. Failure of the system occurred during placement of the sand drainage layer over the geomembrane. Close inspection of the failed cover indicated that the hydrated GCL had not moved and that the failure interface was between the PVC geomembrane and the GCL. Direct shear tests performed on this interface indicated shear strength of approximately 16-18° with an apparent cohesion of 11 psf. The failure was in the form of the geomembrane stretching and finally tearing up slope of the area where the sand cover was being placed. Several wrinkles in the geomembrane were seen originating from the area of soil placement and spreading diagonally upward. The contractor attempted to prevent slides by placing thicker sand cover at the toe region but tearing of the membrane continued. A moderate rainfall was occurring during this time-period.

Two failure theories were developed during the post-failure investigation. These two theories are presented here to demonstrate the complexity in the interpretation of such failures. The two theories also clearly indicate specific areas of concern to the designer.

Theory One – Construction Damage ---- The field observations closely resemble the construction damage scenario described in the previous section. This scenario happens when an overzealous equipment operator tries to push a large height of soil in front of the bulldozer blade. Unfortunately the QA monitor was not monitoring for the thickness of cover soil being pushed and could not verify this. He however stated that while standing on the membrane up slope from the bulldozer, he could feel as though the membrane was being pulled away from under his feet. Figure 3 shows that pushing more than 12-inches of sand by the CAT D3B LGP dozer placing the sand cover would result in the PVC geomembrane being placed in tension. Based on this observation, construction related damage was presented as one potential cause of failure.

Theory Two –Interface Pressure Induced Failure ---- Complicating matters further was another mechanism which might also have played a role in this failure. Photo 3 shows a small gas "whale" observed after the sliding failure had occurred. Gas pressure measurements made in one of the "whales" showed a gas pressure of approximately 6-inch water column (0.21 psi or 1.45 kPa). Figure 1 indicates that for a 4H:1V slope and an interface friction angle of 18°, that an excess pressure, u, of only 37 psf (7-inch of water pressure) will lead to failure of the cover. Such a head can easily develop between the barriers of a composite liner system placed on a slope. Since this level of gas pressure is close to the value of u that would cause failure, an alternative failure theory focused on the presence of excess gas pressures as the cause of failure.

Had one aspect or the other in the above case history been absent, we could have concluded as to a definite failure mechanism. But, it was not to be. The case history, however goes to show that even design and construction using state of the art methods may lead to unacceptable behavior. We still have to understand many aspects of the behavior and interaction of new geosynthetic materials.

Case 2: Stability Failure During Service Life

Completed in the spring of 1995, this cover was 14.5 acres and capped a closed MSW landfill on a military base. This final cover had 6-inches of topsoil, 18-inches of the silty-fine sand vegetative support layer, and a single bonded geonet drain over a HDPE smooth geomembrane. The cap had slopes that ranged from 5% to as steep as 3H:1V. Hydroseeding of the landfill cover was completed in late May, 1995. Subsequent storms in through the summer resulted in significant cover and vegetative support erosion on the steep areas of the cover and an indication that portions of the steeper slopes had actually moved slightly. A significant program of erosion control was implemented and an attempt was made to upgrade the toe drains and to install intermediate pipe drains to relieve the geonet drain. Two storms in late October 1995 caused a cover veneer failure in the steeper portion of the cover. Additional drainage was added to the geonet in this region and the cap was rebuilt. In February 1996, a warm front combined with additional precipitation caused melting of snowfall that had accumulated on the cap resulting in additional erosion, cracking, and movement of some portions of the cap. Examination of the areas of failure showed that the cover soils were saturated and the geonet had not slid on the geomembrane.

Photo 4 shows the significant cracks that formed on the slopes of the cap. While most of the cracks appeared to be related to erosion, many of the cracks on the steeper slopes did not run down hill and were clearly tension cracks. Figure 5 shows an example of a tension crack. The vegetation was obviously torn apart and the walls of the crack are very sharp even in the highly erodible silty sand vegetative support layer. It was also obvious that tension cracks would quickly convert to erosion rills if surface water was present when the tension cracking occurred.

Multiple groups of "experts" examined the cap to determine if it could be repaired or if a major rebuild of the cap was warranted. In general, these studies forced on the adequacy of the geonet lateral drainage system beneath the cap. While erosion was an obvious problem, the general consensus was that it could be remedied with surface structures that would not require removal of the existing cap. Failure of the lateral drainage layer however would require removal of the cap.

A HELP analysis was performed to evaluate the geonet design and the moisture in the vegetative support layer. The geonet drainage lengths ranged from 300 feet at 8.5°to 700 feet at 5.7° and the transmissivity of the geonet was 4 gal/min-ft. This analysis indicated an acceptable inflow into the geonet and little change in the moisture content of the silty sand vegetative support layer. An alternative analysis was performed assuming the silty sand had saturated. The permeability of the silty sands was measured to be 5 x 10^{-4} cm/sec. Using the Theil and Stewart model previously presented, the flow into the 300 feet at 8.5°section of geonet is given by

$$Q_{in} = 300 \times 1 \times \frac{5x10^{-4}}{2.54x12} 7.48 \times 60 = 2.20 gal/\min - ft$$

The flow capacity of the geonet on the 8.5° slope is given by

$$Q_{cap} = \Psi i = 4.0 \tan 8.5° = 0.59 \text{ gal/min-ft}$$

Clearly saturation of the cover soils would produced more inflow into the geonet than it was designed to accommodate. Field excavations also indicated that the hydraulic capacity of the geonet had been reduced by (1) a significant volume of fines that had moved from the silty sand vegetative support layer through the geotextile into the geonet, and (2) root penetration into the geonet. Figure 6 shows root penetration observed in the geonet.

A filter analysis was performed using the silty sand and the nonwoven geotextile component of the geocomposite lateral drain. The geotextile manufacturer reports the fabric has an apparent opening size (AOS) for which 95 percent are smaller (0_{95}) of 0.212 mm (#70 U.S. sieve). Based on the test results, the vegetative support soil layer material had an 85 percent passing (D_{85}) of between 0.300 to 0.150 mm (#50 to #100 U.S. sieves). This results in a $0_{95}/D_{85}$ ratio of only 0.7 to 1.4. This is below Carroll's criteria for $095/D_{85}$ of 2 to 3 (Koerner, 1998). Having a coefficient of uniformity (D_{60}/D_{10}) of approximately 1.1, the Federal Highway Administration (FHWA) standards (Holtz, 1997) recommends that $O_{95} \approx D_{85}$. Thus, the sand marginally meets the FHWA requirements.

The uniform gradation of the vegetative support soil layer makes formation of a soil filter bridge near the geotextile portion of the geocomposite difficult. With approximately 50 percent of the vegetative support soil layer material being able to pass through the geotextile component of the geocomposite, a significant amount of fines could pass through the geotextile before the larger particles could form a filter bridge. Based on observed problems, the drainage system was not adequately designed with respect to soil the internal stability of the soil.

Several significant lessons are learned from this failure: 1) the HELP model may significantly underestimate the saturation of layers above the lateral drainage layer and the rate of infiltration into the geonet. This means that the unit gradient design method for lateral drains (Thiel, 1993) may be appropriate for lateral drains design in most caps. These findings are also supported by research by Soong and Koerner 1998. 2) The use of an 'unstable' soil next to a lateral drainage geocomposite can significantly reduce the available transmissivity due to a failure of the geotextile to retain fines. The ability of a soil to form an internal filter must be confirmed before it is used adjacent to a geocomposite drain. The use of Carroll's or FHWA particle retention criteria with unstable soils is not recommended. 3) The potential for root penetration into the geocomposite must not be neglected. This may require evaluation of roots depths of plants native to the site and corresponding adjustment of the cover soil thickness. Additionally, it is felt that an over designed drainage layer will not contain sufficient available moisture to attract roots. All of these factors warrant a very conservative factor of safety when designing the hydraulic capacity of a lateral drainage system for a cap. Based on these evaluations, the cap was removed from the landfill and reconstructed using a larger transmissivity for the lateral drainage system and a stable soil adjacent to the geotextile of the drainage composite.

Summary

During the last decade considerable progress has been made in understanding how the final covers of Solid Waste Landfills can be designed and constructed. Engineers have welcomed a variety of new products such as drainage geocomposites from the geosynthetic manufacturers and used them to great advantage. One such advantage is the longer drainage lengths that can be used for the subsurface drainage. Constructors have come up with ingenious methods to reduce impacts on geosynthetic components and not to violate designer's assumptions regarding stress conditions. The result of all these has been to reduce considerably, from the beginning of the decade to the end of the decade, the number of unforeseen incidences at the construction site during construction as well as afterward. Thus this had been a decade everybody involved can be proud of. Now what is the prognosis for the current decade? Well, we think it will be more of the same. Some more unexpected and costly failures, not because of the incompetence or negligence of the people involved, but due to a lack of complete understanding of the variables involved. Much of this uncertainty relates to how the field conditions are impacted by weather and construction shortcuts that violate the stress conditions assumed by the designer. Increased interactions between designers and constructors is reducing the construction related gap considerably. But, the weather related issues are something for the owners of the facilities to think about. Is it desirable for them to spend more money and employ very conservative design and construction approaches to account for extreme weather conditions or is it better to design for some damage during construction, which can be repaired at some cost, during extreme weather? Some more failures are also inevitable because of new products that will appear in the market and will be used. These failures are to be considered as full scale experimentation though somebody has to pay for the failures. Questions will be raised as to who should that somebody be, owners, Designers or the material manufacturers or all of the above?

The greatest development we foresee in this decade will come from the challenge posed to the designers and material manufacturers by the concept of bioreactors instead of containment type landfills. These new facilities will demand reusable final covers to pay for the new technology and to reduce waste of the cover materials. We are excited about the possibilities. And, if the last decade was any indication, success is sure to be ours.

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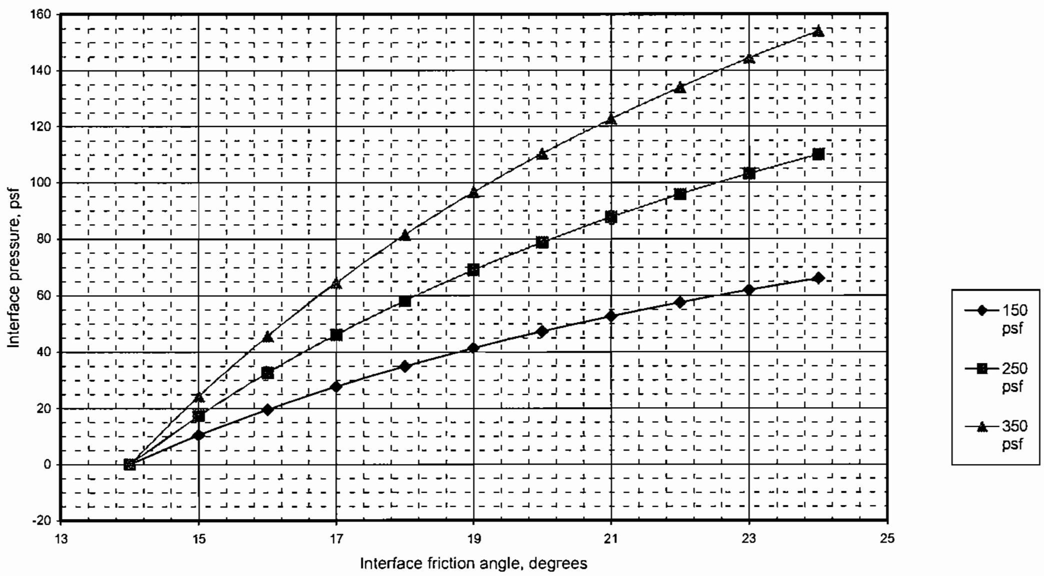

Figure 1 Allowable Interface Pressure for 4H:1V Slopes

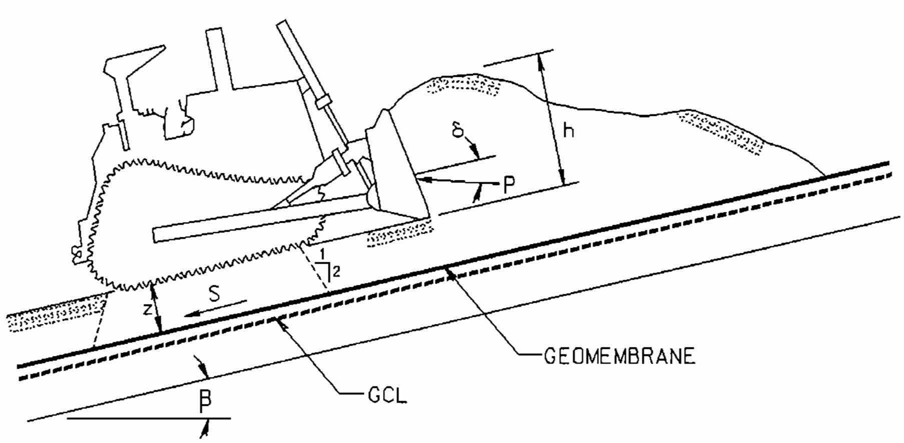

Figure 2 Shear Stress During Cover Soil Placement

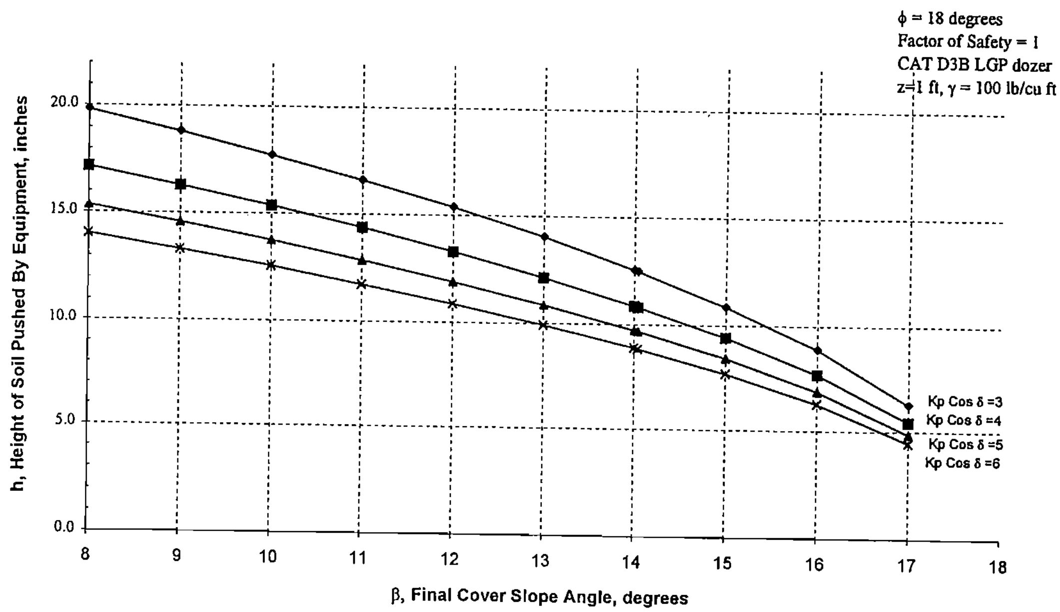

Figure 3 Acceptable Height of "Push" Soil for CAT D3B LGP Dozer

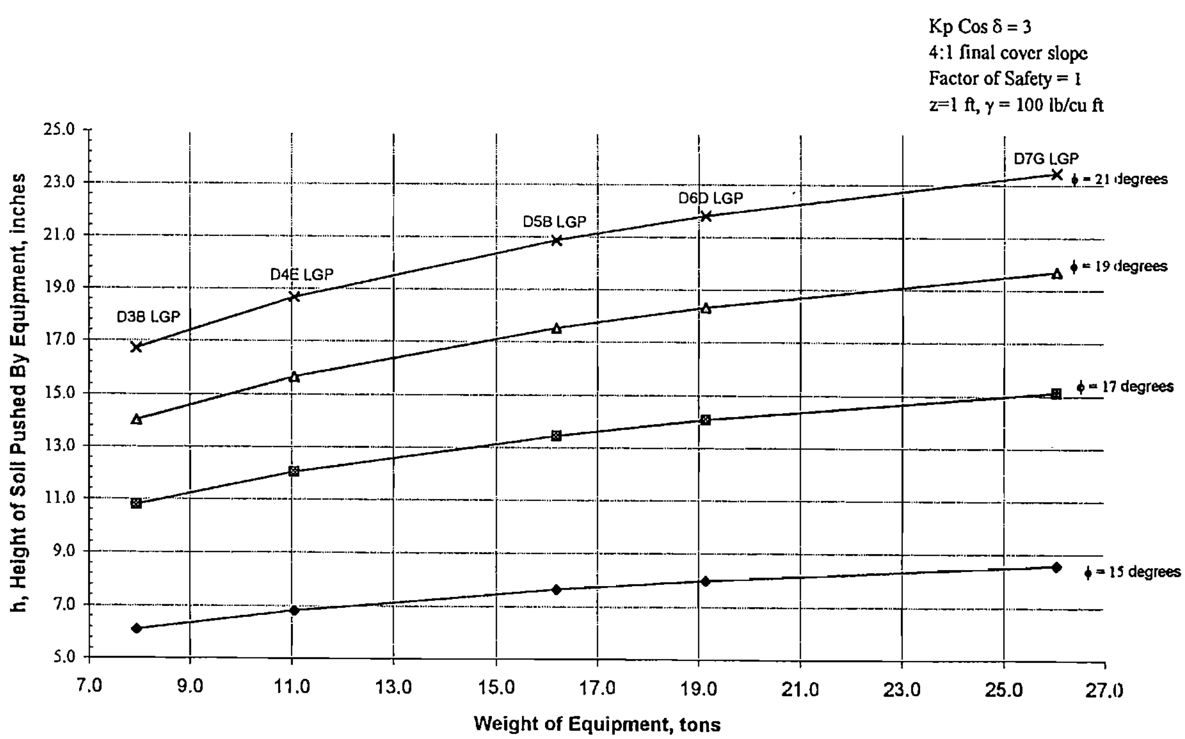

Figure 4 Allowable Height of Soil Pushed by Equipment

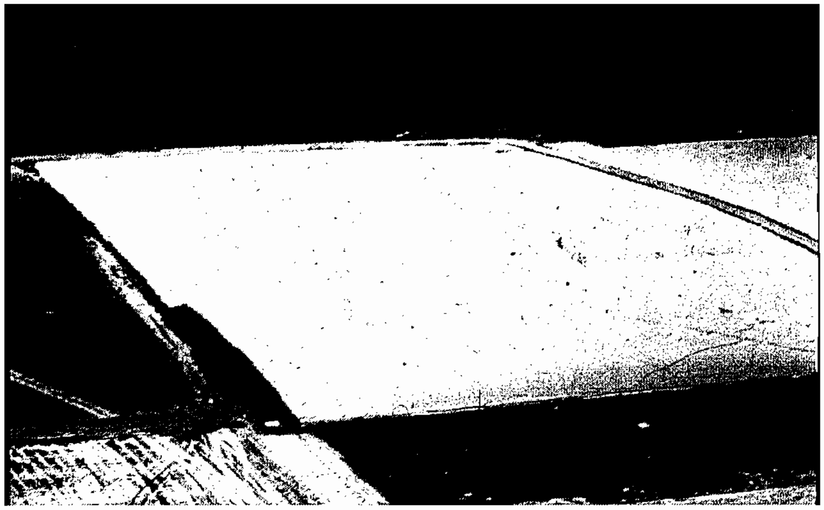

Photo 1 NRC Exposed Geomembrane Cap

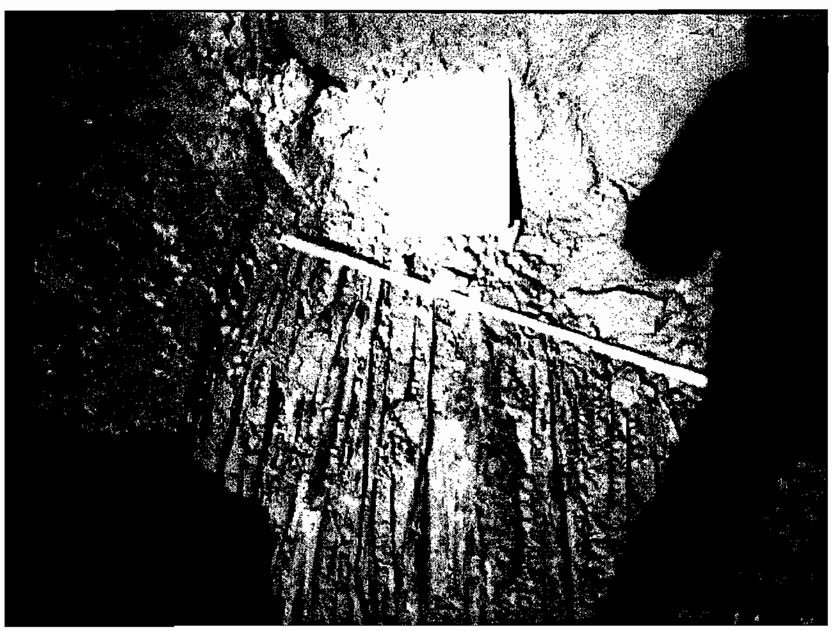

Photo 2 Localized Failure of Geomembrane in Cap

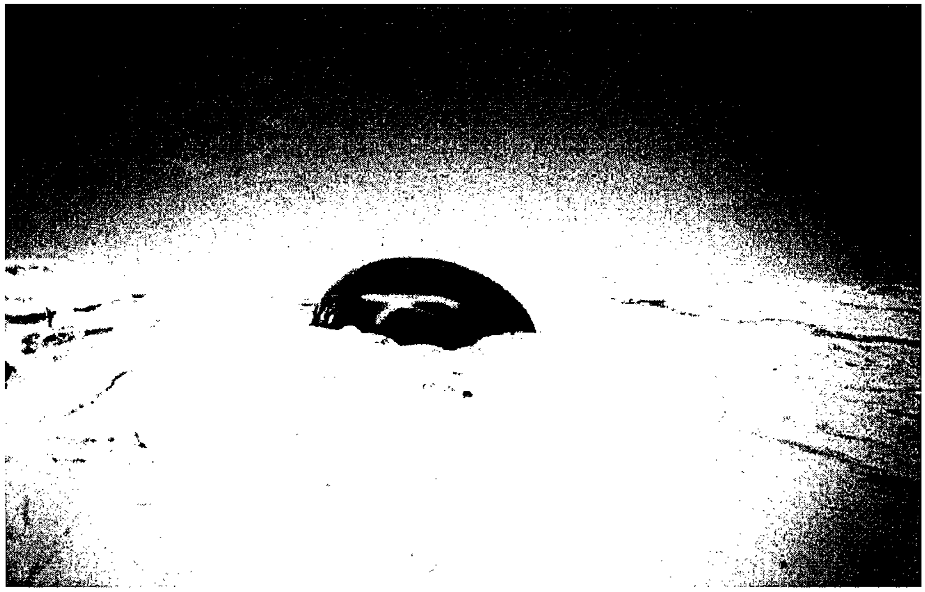

Photo 3 Small Gas “Whale” on Cap Failure

Photo 4 Surface Cracks in Cap That Slid

Figure 5 Slide Related Cracking of Cap

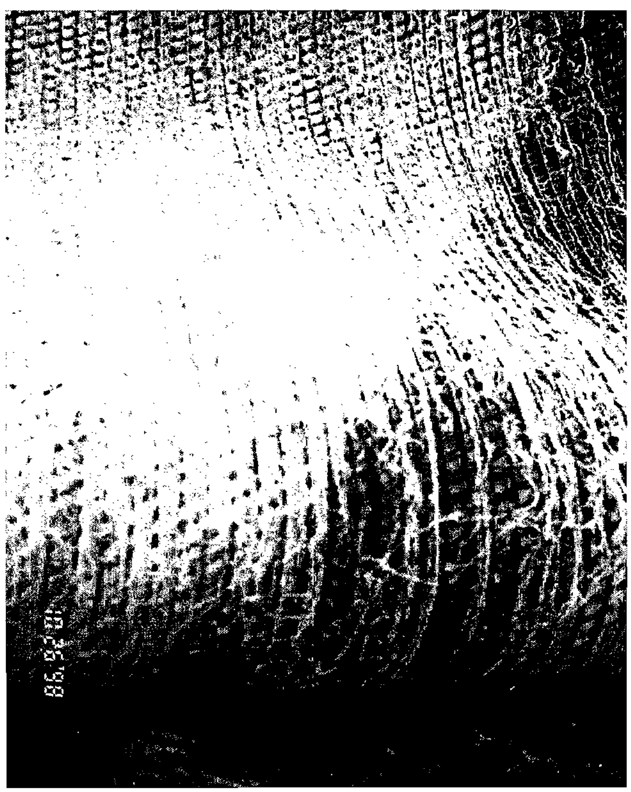

Figure 6 Root Clogging of Geonet