LANDFILL REMEDIATION

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Introduction

Contemporary waste containment cells rely on a layered system of soil liners, synthetic liners, and liquid collection layers to prevent the migration of leachate generated in the waste to the surrounding subgrade. Such systems have been in common usage for 10 years in RCRA related waste containment cells, but are just now achieving similiar usage in municipal solid waste (MSW) waste containment facilities. This paper discusses three landfill failures and the remediation efforts being performed. The waste category and type of failure for the three cases are as follows:

1 - MSW Landfill, General Foundation Failure,

2 - Industrial Landfill, Sidewall Failure, and

3 - CERCLA Closure, Cap Stability Failure.

The MSW case will illustrate the need for design review of daily landfill operations. The remaining cases deal with stability problems inherent in the multi-layered lining systems.

Case 1 - MSW Landfill, Maine

In mid-August 1989, a 500,000m^3 landslide occurred at a commercially operated landfill in central Maine. The landfill material consisted of municipal solid waste (MSW) that rested on a thick deposit of marine clay-silt which provided a natural barrier to leachate seepage.

The movement lasted about 15 seconds. During the slide, huge vertical crevices formed in the landfill. Trash dropped 6

to 9 meters into scarps formed in the underlying clay as the soil slid out from underneath the landfill. The landslide occurred following a 10 day period when over 125 mm of rain fell. During the slide, 6 large crevices opened up in the trash pile. Some of the crevices were 15 meters wide and up to 9 meters deep. Soil was disturbed by the landslide up to 100 meters beyond the original toe of the landfill. Due to remolding, some of the clay lost 90% of its original undrained shear strength. At some locations, the remolded clay and silt flowed over undisturbed soil at a shallow depth. Analysis of the slide indicated that a rotational failure first occurred under the original landfill slope. The rotation left steep unsupported slopes within the trash pile and the underlying clay and silt. Blocks of trash and clay then followed the direction of the initial movement.

While the marine clay and silt offers an ideal natural barrier to the seepage of leachate, the strength of the soil limits the weight of fill which may be placed on top of it. As the landfill expanded, monitoring wells were installed and laboratory tests on soils were run. Some of the monitoring wells included field vane shear tests (ASTM D2573-72) and 76mm Shelby tube sampling. Laboratory testing included classification, strength, and consolidation testing. Figure 1 shows typical laboratory Atterberg and consolidation test data, as well as vane shear data, for the marine clays and silts.

Using the vane shear data and what was thought to be a reasonable value for the density of the landfill, a height limitation of 17 meters was placed on the existing MSW landfill in mid-1986. With fill above that level, it was calculated that the factor of safety against a slope failure would be below 1.25 for short term conditions and that was not acceptable. Laboratory testing subsequent to the landslide and back-calculations from the slide itself have shown that the field vane test values were in fact considerably <u>lower</u> than the shear strengths developed in the clay-silt.

However, another factor that strongly influenced the stability of the landfill slopes was the density of the landfill material. In the early stages of the operation, the owner had little historical on-site data to indicate the landfill density. Consequently, a density that seemed appropriate, based on the appearance of the fill was used. A value of 590 kg/m^3 (1000 lb/cy) was estimated and this value seemed to be corroborated by historical information. In retrospect, it should have been recognized that landfill technology was changing. More compactive effort was being applied in an effort to squeeze greater amounts of trash into limited landfill space. In addition, more daily cover material (sand and gravel) was being added to control odor, birds and blowing trash. These factors all contributed to a much higher density than originally anticipated and used in the stability analyses that were originally performed.

By mid-1987, weight and volume data was available to

indicate the density of the trash and cover was on the order of 1250 kg/m^3 (2125 lb/cy). At that time, the height of the landfill was nearing 12 meters. The reader will recall that an earlier 17 meter height limitation was based on an analysis that used a landfill density of 590 kg/m^3. Without strength increases in the clay, the computed factor of safety against a slope failure would have been less than 1 with the height at 17 meters. Considering clay strength increase, the minimum calculated factor of the landfill slopes was approximately 1.25 with the height at 12 meters and the density at 1250 kg/m^3.

As an additional tool to monitor the stability of the slopes while the fill height was gradually being increased, slope inclinometers were installed on three sides of the MSW landfill. Those were the east, south and north sides. The owner recommended against placing the inclinometers on the west side. The company reasoned that since expansion to the west was thought to be imminent, inclinometers in that area would quickly be in the way of new landfill construction. In hind sight, it was to the west that the inclinometers would have been most useful. As discussed below, slopes in that direction ultimately failed because of the expansion construction activities.

From late 1987 to early 1988 to early 1989, the height of the MSW landfill was gradually increased to about 18 meters. Biweekly readings on the inclinometers indicated a maximum lateral movement of 19mm per year. This rate was judged to be high, but acceptable.

In early 1989, a re-analysis of the landfill slope stability was performed. The re-analysis used the latest height and density information, and extrapolated strength data from the field van shear data. The re-analysis indicated that the safety factor for the landfill slopes was very close to 1. To increase the safety factor, the owner decided to step back the slope at the present fill height and add a berm where possible around the landfill. Berms were built on the east and south sides of the landfill to add counter weight to the slopes. Waste piles to the north and south also provided buttressing in those directions. Again, however, the owner was reluctant to add a stabilizing berm on the west side of the landfill due to planned westerly expansion.

Construction began on the westerly expansion in the late Spring of 1989. Trees cleared, the topsoil was stripped from the clay and silt, and all weathered soil was removed below the topsoil. Some of the weathered soil was mined for cover material for other landfills. Since digging into the clay and silt would also increase the capacity of the landfill, the plans called for the removal of 2 to 2.5 meters of soil in the expansion area. Because the new area was to be lined and the original area was not, a leachate collection trench was dug adjacent to the toe of the old landfill.

In hind sight, it was probably obvious that removing strong soil at the toe, which was supporting the existing landfill

slope, and then cutting a leachate collection trench deeper into the ground at the toe, were not prudent steps to take. Following a 10 day period when over 125mm of rain fell, the landslide occurred.

To permit more accurate back-calculation of the clay and silt shear strengths under the landfill just before the slide, the owner performed a dozen large-scale density tests and 6 direct shear tests in the trash. Each density test involved digging about 8 m^3 of trash out of the fill cross sectioning the excavation to measure its volume, and then weighing the excavated material. The results of the density tests indicated an average density of 1534 kg/m^3 (2600 ld/cy). Those values compare reasonably well with the overall density calculated from 1989 tipping data, truckloads of cover material hauled to the site and volume change computed from different photogrammetrically produced topographic maps (1503 kg/m^3).

To measure the shear strength of the trash, the owner constructed a 1.5m^2 square shear box. The box was loaded with large concrete blocks to vary the normal force in the test. Figure 2 provided a summary of the results.

Summary...The predicted stability failure at this MSW landfill demonstrates the need for ongoing engineering review of the operations of such facilities. Additionally, the measured density of the MSW greatly exceeded that predicted by general historical data. Thus, as even greater efforts are being expended to maximize airspace utilization, the designer must improve such design assumptions. Design of a new MSW lined cell is proceeding for this facility. Future stability will be ensured by limiting the depth of waste and slope of the working face. These limits are being established using slope stability analyses using the measured waste densities and shear strengths.

Case 2 Industrial Landfill, Ohio

During the construction of an industrial landfill in Ohio, a layer of cover soil being placed over the synthetic liner collapsed. This collapse resulted in much of the synthetic liner being dragged to the base of the sideslope. The design profile of the sidewall liner system is shown on Figure 3. As is commonly the case, the sidewall liner system was the product of both state regulatory demands and the designers original intent. Interestingly, the failure occurred between the HDPE liner and the lower slit-film woven geotextile.

Just such a failure had concerned the design engineer. Early calculations indicated that the cover soil would be marginally stable if the slope length was less than 79 feet. To provide a greater margin of safety, the designer required that no more than 15 feet of cover soil be placed in advance of the waste.

As construction progressed, concern was expressed regarding the ability of heavy equipment to operate on the dredge spoils to be placed within the cell. Fearing the future inability to

advance the cover soil protecting the liner, a field decision was made to place the entire cover layer. A sliding failure occurred as placement of the cover soil neared completion and prior to placement of waste in the cell.

Post failure laboratory testing indicated that the coefficient of friction between the HDPE liner and the slit-film woven geotextile was approximately 9-degrees. This confirmed that the weight of the cover soil was carried by tension in the upper geotextile and the liner, by the frictional components between the layers, and by the compressive strength of the cover layer itself. An analysis was performed to estimate the minimum cover soil cohesion required to maintain a minimum factor-of-safety against sliding of 1.0. Figure 4 shows the results of this analysis and the range of cohesion values actually obtained from samples of the cover soil. The cause of the failure became evident when field surveys indicated that slope lengths exceeded 120-feet.

Remediation of the sideslopes involved replacement of the smooth HDPE liner with textured HDPE, and the use of nonwoven geotextiles. Both measures dramatically increased the interface friction angles between the geotextiles and the liner. This successfully reduced the load being carried within the plane of the cover soil. Additionally, note that the geonet drainage layer was bonded to the geotextiles bounding it. This was necessary to prevent placing the geonet in tension.

Case 3 - CERCLA Cover, Connecticut

In a CERCLA closure common to the northeast, sludges generated from the closure of settling lagoons at an electro plating operation were to be consolidated within the footprint of the original lagoons and secured with an impermeable cover. While no specific regulatory criteria exists for CERCLA covers, EPA has generally assumed that RCRA Minimum Technology Guidance provides a reasonable minimum cover profile. This results in a cover that contains the following layered systems:

- Low-Permeability Barrier Layer,
- Drainage Layer, and a
- Protective Layer.

The design profile for this cover and the slope toe drainage detail are shown on Figure 5.

The low-permeability barrier was an effective composite formed by the 30-mil PVC geomembrane and the bentonite mat. However, the bentonite mat has an upper surface composed of a woven polypropylene geotextile. As in the previous case study, the coefficient of friction between a geotextile and a smooth geomembrane typically ranges from 9-12 degrees. Thus the cap profile constructed on the design 3H:1V slopes would either be unstable or would rely on the tensile strength of the filter fabric and the geomembrane.

Just prior to letting bid documents, a geogrid was added to the cover profile. The geogrid was placed immediately above the filter fabric and was intended to carry the weight of the

overlying cover soil. While incorporated into the project specifications, the engineer did not modify the drawings to indicate the proper placement of the geogrid. Fortunately, the small size of the cap, < 1/2 acre, allowed the geogrid to be run continuously across the breadth of the cap.

While no failure occurred in this cap, the success was due to the small size of the cap and not to the technical ability of the designer. No stability calculations had been performed and no geogrid installation guidelines were prepared. Interestingly, the EPA review process did not detect these omissions.

Summary

The rate of failures within waste containment systems is increasing. This increase rate can be directly traced to the following factors:

- The need for the design engineer to establish operational guidelines that ensure the stability of the facility as waste is being placed, and
- Sliding instabilities generated when two geosynthetic materials are used in contact on slopes.

Both designers and regulatory reviewers must ensure that stability calculations are prepared for construction profiles, operational conditions, and closure profiles. Such stability calculations should be used to establish operational guidelines for placement of waste within the waste containment system.

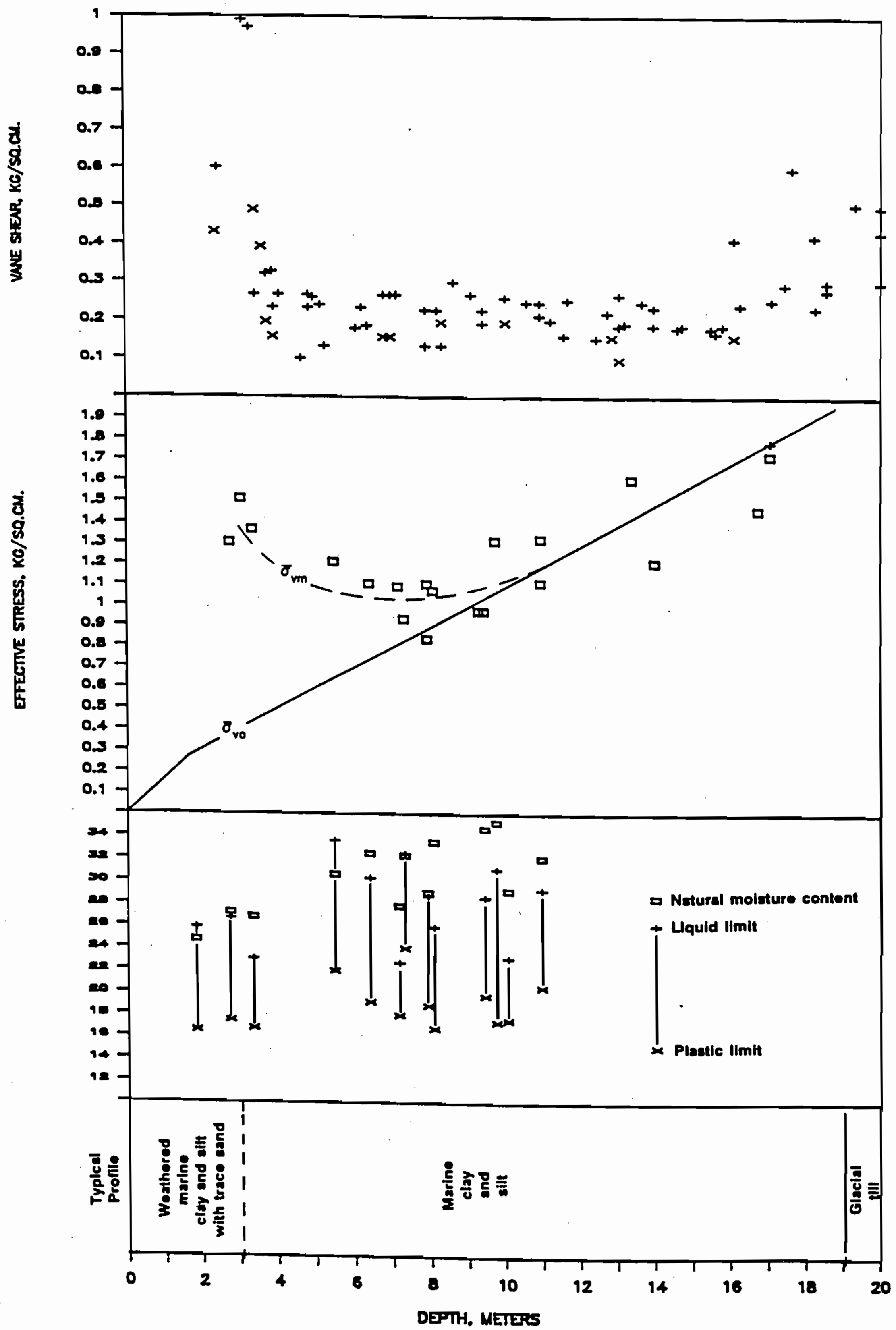

Figure 1 Case 1 Soil Properties Beneath Landfill

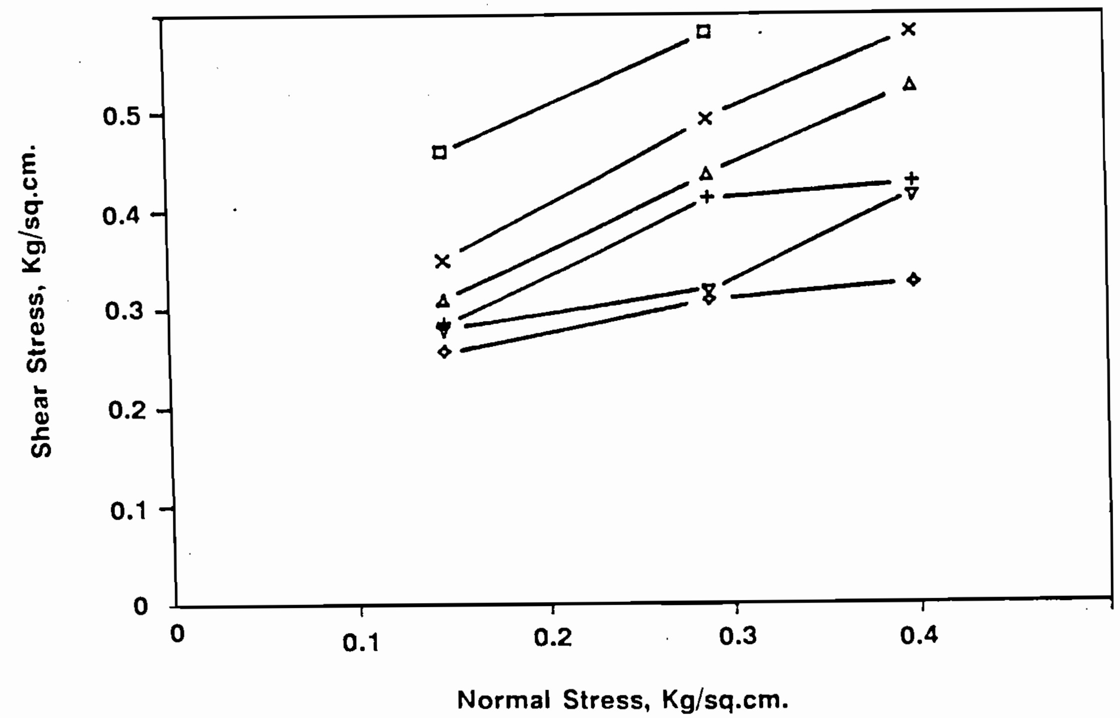

Figure 2 - MSW Direct Shear Test Results

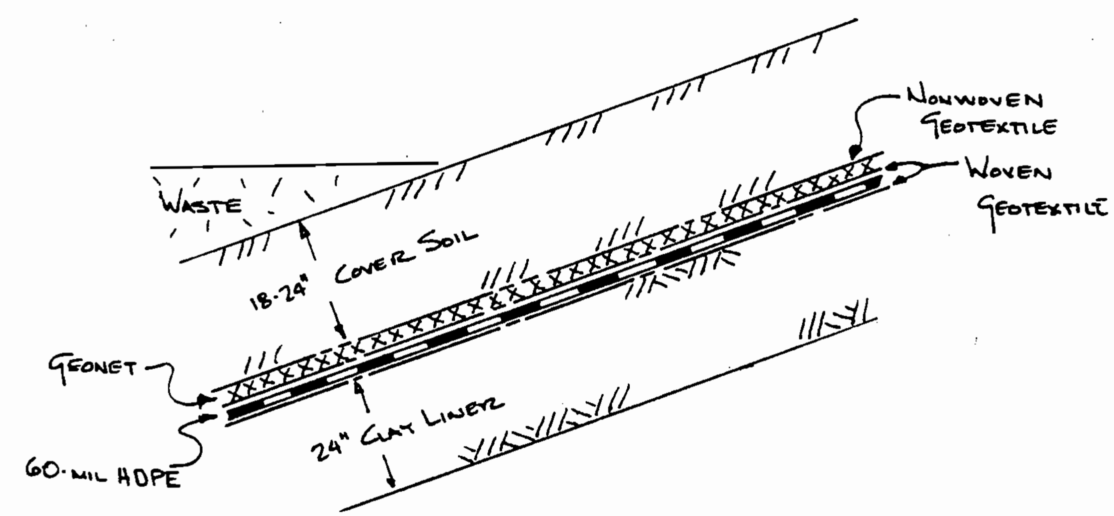

Figure 3 Failed CERCLA Liner System - Case 2

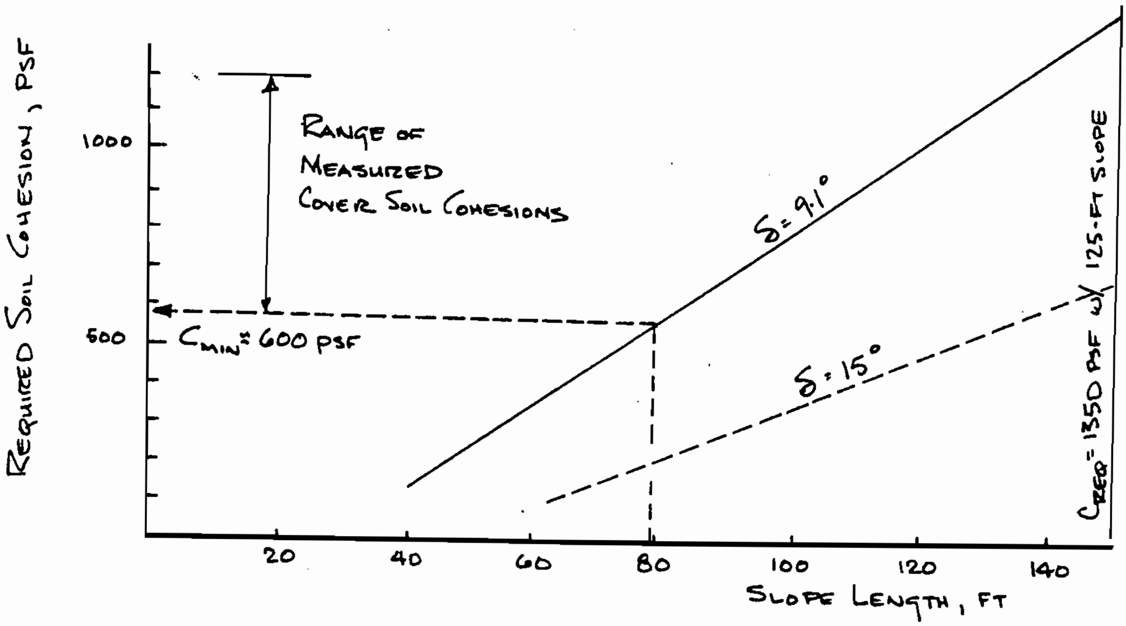

Figure 4 Cover Soil Shear Strength vs
Stable Slope Length - Case 2

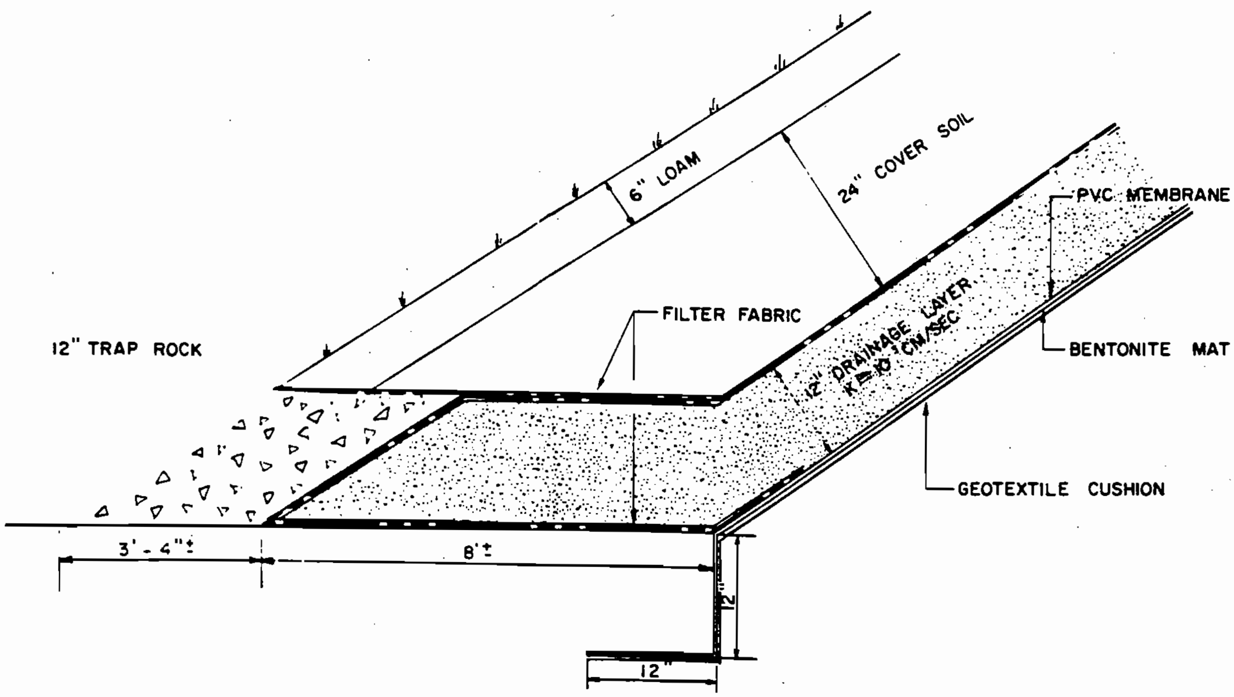

Figure 5 Initial CERCLA Cover System - Case 3