GEOTECHNICAL CONSIDERATIONS FOR THE DESIGN OF PIGGY BACKED LINED LANDFILLS

Gregory N. Richardson, Ph.D., P.E.
G.N. Richardson & Associates
Raleigh, North Carolina

W. Allen Marr, Ph.D., P.E.,
GeoComp Corporation
Acton, Massachussets

ABSTRACT: The design of a lined landfill that must be placed over an existing unlined landfill must be designed so that long term settlements of the underlying refuse do not lead to failure of the lined landfill. This paper presents the geotechnical engineering considerations that have been used to ensure the integrity of piggy backed lined landfills regarding settlement and slope stability failures during its service lifetime. Settlement considerations focus on the amount, total and differential, of settlement that will occur in the refuse below the liner as refuse is placed in the lined facility and as the refuse below the lined facility decomposes.

INTRODUCTION

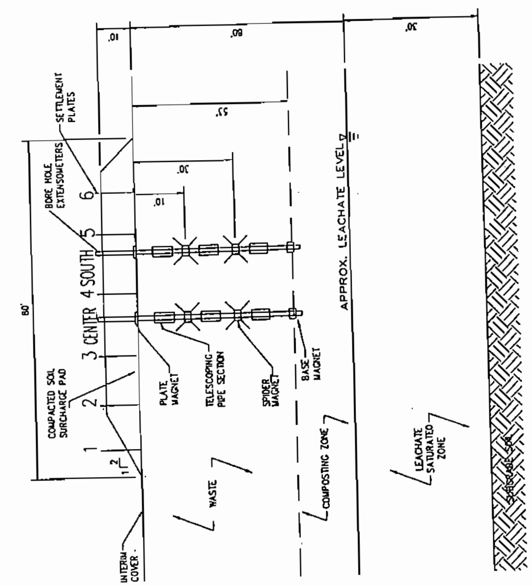
The transition from unlined to lined landfills for municipal refuse has frequently resulted in the loss of significant disposal potential in unlined landfills. This 'lost' airspace can be utilized if it is by placing a lined landfill over the existing unlined landfill. All of the piggy backed facilities constructed to date were built to allow use of significant waste disposal airspace that would have been lost due to new State or Federal lined landfill requirements. The constructed piggy back landfills include those built entirely over an existing landfill and those that are lateral expansions such that the piggy back is only on the slope separating the lined and unlined facilities.

In general, piggy backed landfills allow an environmentally sound closure of large older landfills while meeting current Subtitle D liner requirements.

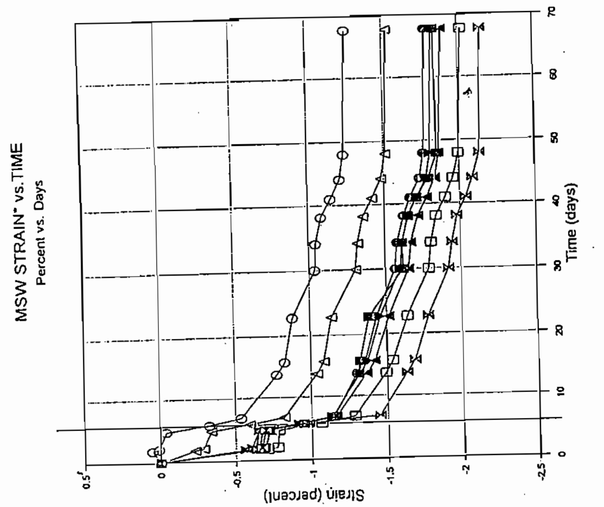
GENERAL SETTLEMENT CONSIDERATIONS

The compressibility of the waste must be measured using actual field loadings generated by test fills. The typical geometry of a MSW test fill is shown on Figure 1. The test fill is typically constructed of daily cover soils and includes a tell-tail device so that the vertical movement of the bottom of the test fill can be measured. The thickness of the waste is measured at the test fill location so that the average compressive strain in the waste can be calculated. The diameter of the test fill is typically the same as the depth of waste. This data is presented as shown on Figure 1. The slopes of the compression verses time curve is evaluated to define the primary and secondary compression ratios, C'. The compression ratio is based on a soil consolidation analogy and is defined as follows:

$$C' = (\Delta H/H) / (\mathrm{Log}\ P_2/P_1)$$



Typical Test Fill Configuration



Typical Presentation of Data

Figure 1 Test Fill to Evaluate MSW Properties where ΔH is the measured settlement, H is the thickness of the waste, P_2 is the vertical effective stress after completion of the test fill and P_1 is the vertical effective stress prior to construction of the test fill. The primary compression index for MSW typically ranges from $0.1 < C' < 0.4$ (Deutsch, 1994, Morris and Woods,1990, Belfiore et. al., 1990, Landva, 1990, and Dodt et al., 1987). Primary compression occurs within 2-8 weeks. The secondary compression index commonly measured is typically much smaller, e.g. 0.01 to 0.02. However, secondary compression may act over a very long time and can produce secondary settlements as large as the primary settlements.

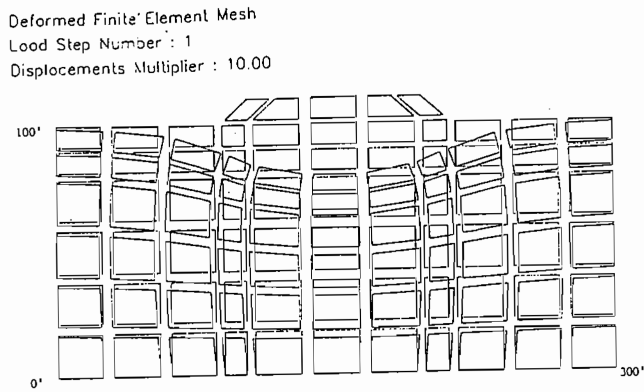
It is commonly assumed that neither the primary or secondary compression indexes reflect degradation of the waste that may occur over the long term. The rate of degradation related settlement is indicated by the rate of landfill gas (LFG) generation and therefore increases with an increase in moisture availability. For lined landfills, the authors experiences have shown that the rate of gas generation is less than for unlined landfills. It is therefore surmised that the settlements due to degradation of the waste will occur at a very slow rate unless the final cover system is damaged so that surface water or precipitation can contact the waste.

FINITE ELEMENT MODELING OF PIGGY BACK EXPANSION

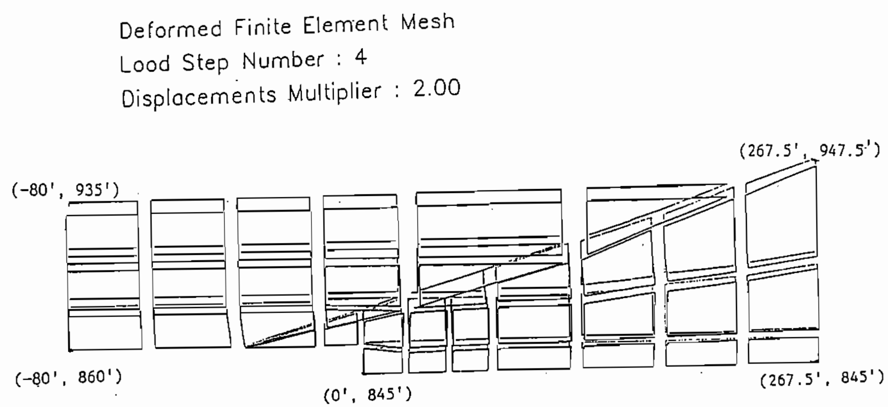
The deformation of the piggy back liner system due to placement of waste in the lined landfill can be predicted using a finite element model (FEM) such as the program FEECON, from GEOCOMP Corporation, Concord, MA. FEECON is able to compute stresses and deformations within the landfill mass due to filling in the proposed lateral expansion (Simon, 1992). The program is able to model the filling of the landfill as a series of lifts. The stresses and displacements of the system are calculated at the completion of each lift of waste. The program updates the node coordinates at the completion of each lift to better model large displacements. The FEM analysis provides several significant improvements over the one-dimensional analysis presented in the previous section: 1- the impact of subgrade irregularities on the strains in the piggy back liners can be better defined and estimated, 2- the amount of lateral spreading that occurs as the result of deformation of the existing waste mound and the resulting impact on the piggy back liner system can be evaluated.

The material parameters used in the FEM analysis are based on adjusting the waste modulus, Poissons ratio, and yield stress to achieve the best fit with the test data from the test fill program. The material properties for the MSW in the existing landfill are estimated by performing a series of FEM analyses of the Test Fill and adjusting the material values to obtain a best fit with the observed settlements. In the example shown on Figure 2, the total ground settlement immediately beneath the Test Fill was approximately 1.72 to 1.80 feet and 0.44 to 0.51 feet at the bottom of the extensometer. Material properties of the FEM model were adjusted to yield a settlement immediately beneath the Test Fill of 1.73 ft and 0.56 at the elevation of the bottom of the extensometer.

A FEM analysis can be performed on the piggy back liner system where it crosses major subgrade irregularities. The FEM study shown on Figure 2 was designed to evaluate the strains in a side slope piggy back liner and the influence of the soil or MSW wedge that transitions over a trench. The abrupt transition over the trench and the potential for large localized strains on the geomembrane was a major concern. Additionally, the FEM analysis was used to evaluate whether structural fill or



FEM Analysis of Test Fill



FEM Analysis of Side Slope Piggy Back Liner

Figure 2 FEM Analysis of Piggy Back Liner Systems

MSW should be placed as the wedge of fill that must be constructed immediately adjacent to the trench. The stepped base represents a best estimate of the depth of trench that will remain when excavation for the base grades of the non-piggy back portion of the lined landfill is completed. Predicted deformations for major lifts of MSW waste placement within the lined landfill are shown on Figure 3. The resulting strains in the piggy back liner system are plotted on Figure 4. The use of soil fill in the wedge reduces the magnitude of the compression strains in the liner but results in an increase in the tensile strain. Both conditions result in peak tensile strains in the geomembrane liner systems less than 1% in the lower portion of the liner.

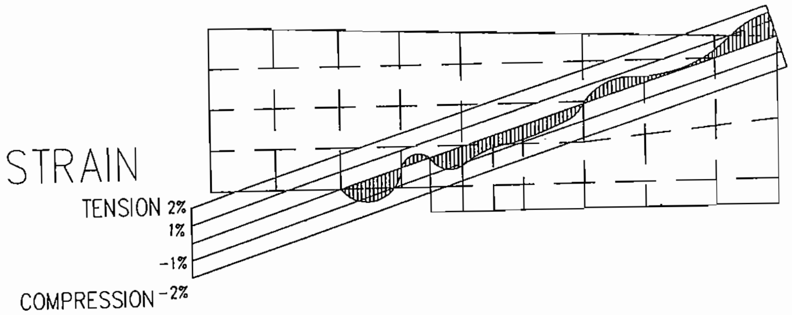


Figure 3 FEM Predicted Strains in Geomembrane Liner

LOCALIZED SETTLEMENT CONSIDERATIONS

The design for general settlements previously discussed does not account for the effect of locally variable compressibility of the waste. Such differences in compressibility are due to the heterogeneous nature of MSW and the field compaction operation itself. Traditionally this analysis is referred to as the "rusted refrigerator" evaluation after the analysis as long required by New York DEC. Here the designer must show that a final cover will remain functional, e.g. limit infiltration, even if a standard refrigerator were to rust out and create a void immediately beneath the landfill cover. Since the piggy-back liner is also placed immediately over old MSW, the analysis is appropriate for the new liner.

The standard design approach is to use the work of Giroud et al. (1990) to compute the tensile stress that might occur in a membrane over the void, then provide a geogrid that has a long-term design

tensile strength greater than this tensile stress. Giroud used the work of Terzaghi and Marston on arching to determine the amount of the overburden stress that acts on the deformed membrane. According to Terzaghi (1943), the vertical stress at the surface of a developing hole can be computed as follows:

$$\sigma_v = \frac{R(\gamma - c / R)}{K * \tan\phi} * (1 - e^{-K * \tan\phi\, H / R})$$

where
σ_v = vertical stress at the surface of the developing hole
R = radius of the hole
γ = unit weight of material over the hole $\phi =$
c = cohesive component of strength of material over the hole friction angle for strength of material over the hole
K = lateral earth pressure coefficient in material above hole
H = thickness of material over the hole

K is assumed to be 1, following the discussion of Terzaghi (1943). $\phi > 20^\circ$

Giroud et. al. (1990) assumed that the cohesion, c, equals zero and K*tanϕ equals 0.5 to obtain a simplified version of the above equation as:

$$\sigma_v = 2R(\gamma \cancel{- c/R}) * (1 - e^{0.5 * H / R})$$

The tension created in the liner system carrying this stress over an open hole can be determined from Giroud et al (1990) as:

$$T_L = \frac{\sigma_v * R}{\sqrt{24 * \epsilon}}$$

ϵ = ALLOWABLE GM STRAIN

Combining the above equations gives:

$$T_L = \frac{2 * \gamma * R^2}{\sqrt{24 * \epsilon}} * (1 - e^{(-0.5 * H / R)})$$

For a 6 foot diameter hole, H>>R, and MSW with a γ of 45-75 pcf, the tension created in the liner system stretching over the hole is 520-870 lb/ft for 10% strain and 740-1230 lb/ft for 5% strain. To protect the liner system from rupturing over the hole, some designers add layers of geogrid to the liner subbase to carry this tension.

Sufficient data are now available regarding the strength of MSW to remove the assumption by Giroud that cohesion of MSW is zero. Kavazanjian et al. (1995) recently summarized strength properties for MSW waste from published lab and field tests on MSW wastes and back calculated from landfill slopes. Kavazanjian recommends a bilinear strength envelope for MSW materials with a cohesion of 500 psf for stresses below 600 psf. Kavazanjian's recommendations agree with the authors observations that MSW has a substantial internal cohesion as evidenced by MSW standing at near vertical faces for heights of 30, 40.... even 90 ft. A cohesion of 500 psf corresponds to a stable vertical face with a unit weight of 45 pcf for a height of about 40 feet.

Returning to the equation for σ_V, the term (γ-c/R) becomes negative for a radius of 3 ft, a cohesion of 500 psf and values of γ less than 166 pcf. Since the γ for MSW is less than 75 pcf, we can conclude that the term (γ-c/R) is always negative for MSW and typical void sizes. The MSW has sufficient internal cohesion that it completely arches over a 6 ft diameter void within the waste. Thus, there is no need for an additional support system to bridge the MSW over a 6 ft hole if H>>R.

There is still a need to support the weight of the liner and leachate collection system over the hole if H is not significantly larger than R. Three cases of a 6 ft hole at a shallow depth beneath the liner are considered in the following table:

Case	10% Strain	5% Strain
2 ft clay and 2 ft drainage layer (γ_v=130 pcf)	740 lb/ft	1050 lb/ft
GCL and 1 ft drainage layer (γ_v=130 pcf)	230 lb/ft	320 lb/ft
GCL and Geocomposite Drain	0	0

The second case can be easily reinforced with biaxial geogrids and the third case requires no extra support. The need for reinforcement can be eliminated if voids are not present in the upper 10 ft of the underlying MSW. This can be confirmed using geophysical methods, e.g. magnetometer survey, to locate metal objects or voids existing near surface in the unlined MSW mass.

SLOPE STABILITY OF PIGGY BACK LINER SECTION

The piggy back liner section must be designed to be stable during and immediately after construction, as waste is placed into the lateral expansion, and over the long-term when subjected to seismic loadings. The analyses presented in this section establish the minimum interface friction that must be developed between the layered components forming the piggy back liner system. Prior to construction, each critical interface must be tested in a direct shear test (ASTM D3080 or ASTM D5321) to verify the interface friction angle for the actual components to be constructed.

Post-Construction Stability ---- The general equation for the factor of safety, FS, of an infinite slope veneer can be assessed using the following general equation (Matasovic, 1991):

$$FS = \frac{c/(\gamma \cdot z \cdot \cos^2\beta) + \tan\phi\,[1 - \gamma_w(z-d_w)/(\gamma \cdot z)] - k_s \cdot \tan\beta}{k_s + \tan\beta}$$

where k_s = seismic coefficient, γ = unit weight of slope material(s), γ_w = unit weight of water, c = cohesion, ϕ = angle of internal friction of the assumed failure interface of surface, z = depth to the assumed failure interface or surface, and d_w = depth to the water table (assumed parallel to the slope). For the short-term post-construction stability analysis, the seismic coefficient can be set equal to zero and the minimum FS acceptable is 1.5.

The factor of safety against sliding for the cohesionless leachate collection removal (LCR) system (C=0) can be expressed as follows:

$$FS = \frac{\tan\phi\,[1 - \gamma_w\,(z-d_w)/(\gamma \cdot z)]}{\tan\beta}$$

If we assume that the LCR system is 50% saturated (d_w = 0.5z), e.g. exposed to a design storm, then the above equation can be written as follows:

$$FS = \frac{\tan\phi\left[\;1 = 0.5\,\frac{\gamma_w}{\gamma}\right]}{\tan\beta}$$

Given a design slope of 3H:1V (β = 18.4°) and an estimated unit weight of cohesionless soil of 100 pcf, the minimum interface friction, ϕ, required is 36°. Neglecting seepage forces, the minimum interface friction angle is 26.5°, this will require the use of a textured geomembrane liner.

Operational Stability Considerations — Waste will be placed within the lined lateral expansion in the sequence shown on Figure 20. As a result of this sequence, the last volume to be filled will be the valley defined by the slopes of waste placed in the non-piggy back portion of the lateral expansion and the piggy back liner itself. Waste placed within this valley will be stable if placed in continuous horizontal lifts not exceeding 10 ft. in height. Each lift must be completed full width across the valley prior to beginning a new lift. Each lift must begin at the slope formed by waste previously placed in the non-piggy back portion of the lined landfill and progress toward the piggy back portion of the liner.

Long-Term stability Considerations ---- The post-closure stability of the lined lateral expansion must be evaluated to ensure that it is stable in the event of a future seismic event. As required by 40 CFR 258, the completed lined lateral expansion must have a minimum factor of safety of 1.0 when subjected to the maximum credible seismic event. The long-term seismic stability can be evaluated using the procedure recommended by EPA (Richardson,et. al., 1995).

SUMMARY

The design of piggy back lined landfills and on sites having large post-operational settlement potential can be successfully performed using basic analysis procedures commonly used by geotechnical engineers on conventional structures. Piggy back landfills offer their owners the potential to reclaim airspace that was lost due to regulatory changes. In most instances, the use of a piggy backed lined landfill will be politically more expedient that siting a new lined landfill.

References

Belfiore, F., Manassero, M. and Viola, C. (1990), "Geotechnical Analysis of Some Industrial Sludges," Proc of Geotechnics of Waste Fills - Theory and Practice, ASTM STP 1070, Philadelphia.

Bjarngard, A.B. and Edgers, L. (1990), "Settlement of Municipal Solid Waste Landfills," Thirteenth Annual Madison Waste Conference, Sept 19-20, University of Wisconsin, Dept. of Engineering.

Carey, P.J., Nagesh, K. and Gurda, J.J. (1993), "Renewal of Old Sites - Overliner System Design, A Case Study of the Brookhaven Landfill, Long Island, New York," Waste Tech '93, National Solid Waste Management Association, January 14-15, Marina Del Rey, CA.

Deustch, W.L., Easterly, O.R., and Vitale, J. (1994), "Modeling Settlements of an Existing Municipal Solid Waste Landfill Sideslope Using an Earthen Surcharge Pile," Specialty Conference on Vertical and Horizontal Deformations of Foundations and Embankments, ASCE, Texas A&M University.

Dodt, M.E., Sweatman, M.B. and Bergstrom, W.R. (1987), "Field Measurements of Landfill Surface Settlements," Proc. of the Geotechnical Practice for Waste Disposal, ASCE, Geot. No. 13, June.

Giroud, J.P., Bonaparte, R., Beech, J., and B. Gross (1990). "Design of Soil Layer-Geosynthetic Systems Overlying Voids," Geotextile and Geomembranes, Vol. 9, No. 1, pp. 11-50.

Guerriero, J.R. and Druback, G.W. (1991), "Implementation of Vertical Landfill Expansion," Proc. of Environmental Engineering '91'.

Landva, A.O. and Clark, J.I. (1990), "Geotechnics of Waste Fill,"Proc of Geotechnics of Waste Fills - Theory and Practice, ASTM STP 1070, Philadelphia.

Matasovic, N. (1991). "Selection of Method for Seismic Slope Stability Analysis," Proc. 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri.

Morris, D.V. and Woods, C.E. (1990), "Settlement and Engineering Considerations in Landfill and Final Cover Design," Proc of Geotechnics of Waste Fills - Theory and Practice, ASTM STP 1070, Philadelphia.

Richardson, G.N., Kauazanjian, E., and N. Matasovic (1995). RCRA Subtitle D(258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities, EPA/600/R-95/051, U.S. Environmental Protection Agency, Washington, D.C.

Simon, R.S., Ladd, C.C., and Christian, J.T. (1972), "FEECON - A Computer Program to Predict Performance of Embankments on Soft Soil," MIT Department of Civil Engineering, Cambridge, MA.

Tieman, G.E., Druback, G.W., Davis, K.A. and Weidner, C.H. (1990), "Stability Consideration of Vertical Landfill Expansions," Proc of Geotechnics of Waste Fills - Theory and Practice, ASTM STP 1070, Philadelphia.

Terzaghi, 1943