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Stability of geogrid-reinforced landfill liners over sinkholes

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ABSTRACT: Plain-strain finite element analyses were conducted to estimate the stresses and deformation induced in landfill liners constructed on existing or potential sinkholes. The analyses demonstrated the significance of using structural geogrids to reduce the magnitude of deformation and possibly the potential for collapse of landfill liners. Variations of analysis key parameters included depth of overburden (D) and width of the sinkhole (B). For a D/B ratio of 2 and a D of 40 feet, the tensile strain is reduced from 18%, for the case of no reinforcement, to 5% when reinforcement is included.

1 INTRODUCTION

Landfill siting is a complicated process that must satisfy several environmental, socioeconomic, regulatory, and engineering criteria. It is not uncommon that selected landfill sites that satisfy most of these requirements are located on challenging geologic formations such as the mantled Krast terrane zones. These zones, usually overlain by unconsolidated sediments, are susceptible to solution weathering and therefore to the development of sinkholes.

According to the size and depth of a given sinkhole and type and strength of the overburden, a varying degree of subsidence will take place as the imposed loads exceed the bearing strength of the soil profile. A failure could take place during construction, operation or after the closure of the landfill.

If conditions are such that unacceptable levels of deformation may lead to collapse of landfill liners, synthetic geogrids can be used to reinforce the liners and bridge over the weak zones.

2 SINKHOLE FORMATION

Sinkholes are generally found in Karst terrane areas characterized by soluble rock, such as limestone. The soluble rock is usually overlain by a varying thickness of unconsolidated sediments.

Typically, soluble rock deposits of the Karst terrane contain vertical joints with internal drainage fissures. Water seeps into the vertical joints and erosion occurs causing overburden to fall into the cavity and generates a depression at the ground surface. As presented by Handfelt and Attwool (1987) types of sinkholes could be classified as either collapse or ravelled. Figure 1 schematically illustrates the formation of the two types. A collapse-type sink hole will cause a depression at the ground surface due to the collapse of the cavity roof. A ravelled-type sinkhole will cause a depression at the ground surface due to erosion and ravelling of the soil above the cavity. As stated by Ruth et al (1985), most sinkholes are 5 to 20 feet in diameter. However, sinkholes as large as 350 feet in diameter have been reported. Manmade activities such as modifications to the ground surface and construction activities can promote the development of sinkholes. Goehring and Sayed (1989) described a case study where, during construction activities, an adjacent sinkhole expanded from 30 feet to 50 feet in diameter during a period of three weeks.

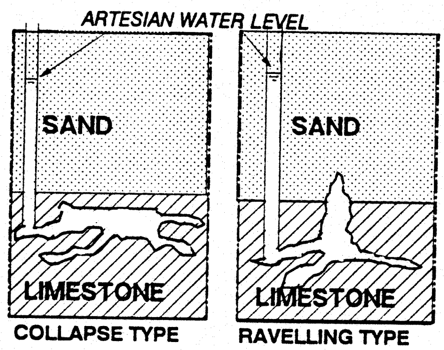

Figure 1. Types of Sinkholes, After Handfelt et al. 1987

3 EXISTING ANALYSIS METHODS

Few attempts have been made to develop an analysis methodology for the design of landfill liners in areas of sinkhole activities. A finite element analysis was conducted by Drumm et al (1987) to investigate the effect of bedrock cavities on deformational and stability characteristics of a residual soil profile. Axisymmetric conditions were assumed in the analysis. In a follow up study by Drumm et al (1990) the stability of Karst sites in eastern Tennessee was evaluated and analyzed using plain-strain finite element analysis. The feasibility of using geogrids to improve the stability conditions was not within the scope of either studies.

Bonaparte and Berg (1987) presented a procedure for supporting roadways over depressions using geogrids. Application of this method to a landfill situation was also adopted. Giroud et al (1990) presented design charts, equations and tables for the design of soil-geosynthetics systems to span voids. The development of this design methodology was based on the assumption that the geosynthetics will be placed directly on top of the void.

Drumm et al (1990) presented a summary of empirical methods that have been used to predict ground surface subsidence in mining engineering. These methods are mainly based on establishing profile functions. Two profile functions that are commonly used in the Appalachian coalfields of the eastern United States are summarized as follow:

1. Hyperbolic Tangent Function by Brauner (1973):

$$S(x) = \frac{1}{2} So \,\{1-\tanh(\frac{CX}{B})\} \qquad (1)$$

where S(x) = vertical displacement at location X, So = maximum vertical displacement, and C = an empirical parameter.

2. Negative Exponential Function by Chen and Peng (1981):

$$S(x) = So \, \exp^{-\alpha(\frac{x}{L})^{\beta}} \qquad (2)$$

where L = half-width of the subsidence basin, and α and β are empirical

Profile functions are empirical in nature and do not directly account for the engineering properties of the subsurface soil profiles. Moreover, as stated by Drumm et al (1990) an estimate of the maximum vertical subsidence is required which in a landfill case, unlike mining, cannot readily be evaluated.

Because of the variable nature of the problem, incremental non-linear finite element analysis is used in this study. The analyses include the study of two cases in which the thickness of the overburden and the size of the sinkholes were varied. The effects of subsurface cavities on potential deformation and subsidence strains of landfill liners is evaluated.

4 FINITE ELEMENT MODEL

The plain-strain computer program SSCOMPPC by Boulanger et al (1991) was utilized in this study. Using this program, stress-strain behavior of the soil is approximated using a hyperbolic model as presented by Duncan et al (1984). This model requires the user to input material constants describing the soil behavior. Based on review of available subsurface conditions of Karst terrane areas, a representative profile was idealized along with probable sinkhole sizes. As shown in Table 1, two cases were investigated. For each case a sand layer is overlying a limestone bedrock. The engineering properties of the subsurface strata along with the input analysis parameters are summarized in Table 2.

Table 1: Size of Sinkholes and Depth of Overburden for the Analysis Cases

	CASE 1	CASE 2
Overburden (D), feet	20	40
Sinkhole Size (B), feet	10,20,30	10,20,30

Table 2: Engineering Properties of Idealized Subsurface Layers

	SAND	LIMESTONE
Φ (degree)	32	40
qu (psf)	0	32,000
K	1150	11,900
Kb	960	1,000
n	0.6	0.5
m	0.5	0.5
Rf	0.9	0.7

where: Φ= angle of internal friction, qu= compressive strength, K= modulus number, n= modulus exponent, Rf= failure ratio, Kb= bulk modulus number, and m= bulk modulus exponent, as defined by Duncan el al (1984).

For the sake of this study it is assumed that a collapse-type sinkhole is to be developed. In addition it is assumed that no roof collapse, erosion, or ravelling has taken place. The landfill cell is assumed to be 100 feet wide and the sinkhole depth is 15 feet.

The idealized analysis profile and the finite element mesh are shown in Figure 2. Half of the domain geometry is modeled in the analysis.

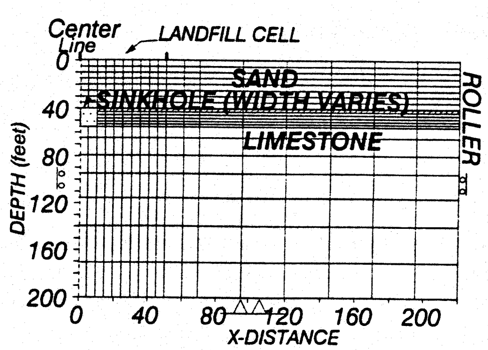

Figure 2. Finite Element Mesh and Boundary Conditions

The boundary conditions are chosen, as shown in Figure 2.

5. NO-REINFORCEMENT ANALYSIS

The finite element analysis was conducted for the cases described in Table 1 assuming no geogrid reinforcement. The effect of soil arching on redistribution of the imposed waste loads was investigated. Figure 3 presents the vertical stress distribution at the sand/limestone interface for an applied surface pressure of 3000 psf.

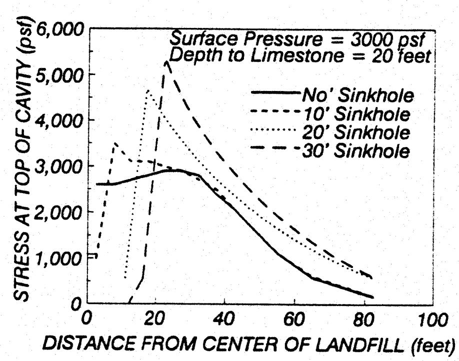

Figure 3. Vertical Stress Distribution at the Sand-Limestone Interface

The distribution of the stresses is significantly influenced by the presence of the sinkhole. As the size of the void increases more of the surface pressure is carried by the soil around the void and the arching ratio (R), defined as the stresses "immediately" above the roof of the void divided by the applied surface pressure, becomes less. For the 10 feet wide sinkhole, R is approximately equal to 1/3. For the extreme case of a 30 feet sinkhole, R approaches a value of zero which indicates a complete arching of stresses around the sinkhole. However, the arching mechanism results in redistribution of the stresses to the soil around the void. Depending on the void size, the magnitude of the resulting stresses outside the void limits is higher compared to the case of no void, as shown in Figure 3. Consequently, the maximum surface deformation would occur in the vicinity of the voids rather than within the voids limit.

Deformation evaluated from the finite element analysis was used to estimate the tensile surface strains. These tensile strains represent strain to be imposed on liner systems constructed on subsurface profiles containing the idealized voids. For a given liner section, the piece-wise tensile strain is estimated using horizontal and vertical surface settlements as shown schematically in Figure 4.

$$e = \frac{\sqrt{\Delta s^2 + (L1 + \Delta H)^2} - L1}{L1}$$

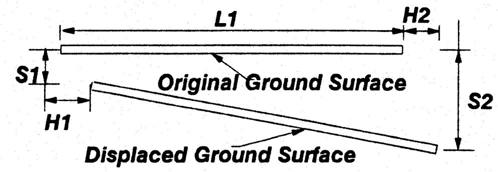

Figure 4. Scheme for Computing Tensile Strain in a Section of a Liner System.

Tensile strain contours are plotted in Figure 5 (a,b) as a function of the applied surface pressure for overburden thicknesses of 20 and 40 feet. As the depth of overburden to the diameter of sinkhole (D/B) ratio is increased the surface tensile strain is decreased. For an applied surface pressure of 10,000 psf and overburden thickness of 40 feet, the tensile strain ranged from 17% for D/B of 2 to approximately 6% for D/B ratio of 4. For the severe case of 20 feet of overburden, a tensile strain of 20% was estimated for a D/B of 1 (20-foot sinkhole) under an applied surface pressure of 7000 psf.

The strain contours are dependent upon the size of the sinkhole, the depth of the overburden, and the applied pressure. For the same applied pressure and D/B ratio, the magnitude of tensile strain could be different for different overburden depths. For a D/B ratio of 2 and applied pressure of 10,000 psf the tensile strain is approximately 5% for 20-foot overburden and 17% for a 40-foot overburden. Such a behavior is due to the fact that the size of the sinkhole for the former is 10 feet and for the latter is 20 feet. Therefore, the magnitude of strain can not be uniquely defined in terms of D/B ratio without specifying the value of D.

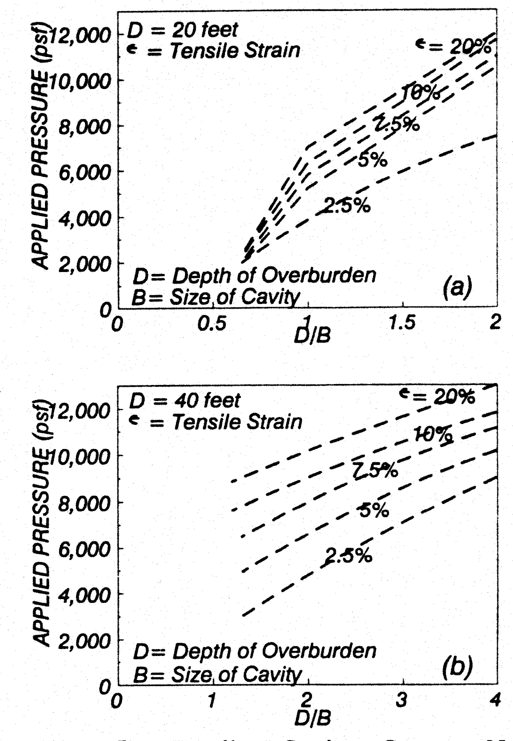

Figure 5. Tensile Strain Contours-No Reinforcement

6. GEOGRID-REINFORCEMENT ANALYSIS

A geogrid support system was used to reduce the amount of tensile strain on the landfill liner. The polymer geogrids were modeled as interface elements with separate interfaces above and below the layer of reinforcement. Values for reinforcement interface parameters are given in Table 3 and were adopted from data presented by Collin (1986), Human et al (1986), and Schmertmann et al (1989).

Table 3. Soil-Reinforcement Interface Properties

$\delta°$	$\Delta\delta°$	kn	ks	n	Rf
25	3	$1x10^6$	$1x10^3$	1	0.9

kn= normal spring stiffness constant, ks= shear spring stiffness constant, n= modulus exponent, Rf= failure ratio, and δ= interface friction angle, as defined by Boulanger el al (1991).

The analysis was conducted for the Case of 40-foot overburden. As shown in Figure 6, the inclusion of the geogrid-reinforcement directly below the liner resulted in lowering the magnitude of the tensile strain on the liner system.

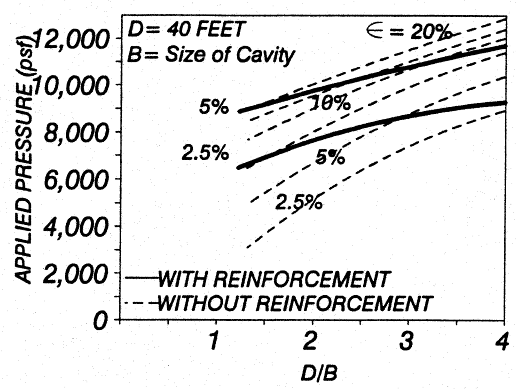

Figure 6. Tensile Strain Contours for Geogrid-Reinforced Liners

For the case of D/B of 2 and applied pressure of 10,000 psf, the magnitude of tensile strain was reduced from 17% to approximately 5%. The confinement effects due to the geogrids assisted in increasing the arching mechanism and mitigating a portion of the applied pressure through its mobilized shear resistance. As D/B ratio increases the surface deformation decreases and less of the shear strength due to reinforcement is mobilized. It can be speculated that for an overburden depth of 40 and D/B ratio greater than 4, the amount of strain induced at the ground surface is not sufficient to mobilize the shear strength of the geogrids. Therefore, as D/B ratio increases the contour lines with and without reinforcement tend to converge.

7. DISCUSSION AND CONCLUSIONS

While it is not necessarily the case that sinkholes may exist at the time of constructing a given landfill, it is also not necessarily guaranteed that such geologic hazards will not develop during the lifetime of a landfill. While it would always be preferable to avoid the construction of landfills in such zones, it has been the case in the past that several landfills are located on mantled karst terrane areas. Until an alternative to landfilling is found, such a trend is expected to continue in the future as land suitable for the construction of landfills becomes less and less available.

Research presented herein is aimed at increasing the state of the knowledge regarding the feasibility of using synthetic geogrids to improve the performance of landfill liners over subsurface voids. The analysis results are based on assuming that plain-strain conditions are representative of site conditions. Such an assumption implicitly models the sinkhole as a tunnel rather than as a void with a limited extent. Figure 7 presents the results of a linear finite element analysis using the general purpose computer program ALGOR (1990). As expected, the magnitude of settlement is less if the sinkhole was modelled as a void with a spherical or cylindrical shape.

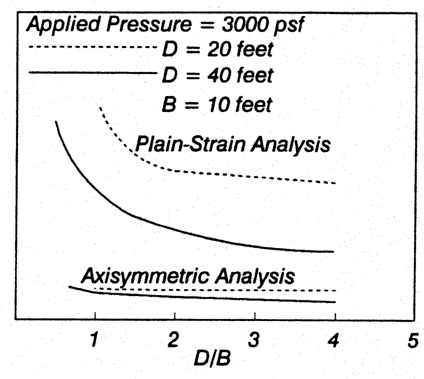

Figure 7. Deformation From Axisymmetric and Plain Strain Linear Analyses

However, in a site prone to subsurface void formation, rock fractures or lineaments could be continuous in nature and more than one sinkhole can form within the area. Accordingly, a plain-strain analysis may be more applicable and provide results that are conservative. Based on the analyses conducted herein the following conclusions can be drawn:

1. Arching contributes to the redistribution of stresses within the soil mass adjacent to the void. Because of this mechanism, the maximum stresses occur outside the limits of the void rather than directly above it.

2. Accordingly, assuming no roof collapse or ravelling, the maximum deformation and therefore tensile strain in the liner system will occur in the vicinity of the void.
3. The tensile strain cannot be defined only in terms of the D/B ratio and the applied pressures. The depth of overburden should also be specified.
4. The use of structural geogrids to reinforce the landfill liner system reduced the tensile strain from 17% to approximately 5% for the case of a D/B of 2, a D of 40 feet, and an applied pressure of 10,000 psf.
5. This analysis did not address the strain compatibility of typical liner components nor the long-term stability of the geosynthetic material. Strain distribution in the different components (flexible membranes, geotextile, clay barriers, etc..) should be considered.

REFERENCES

Algor Interactive Systems, Inc. 1990. AccuPak Finite Element Analysis System, Pittsburgh.

Bonaparte R. and R. R. Berg. 1987. The use of Polymer Geosynthetics to Support Roadways Over Sinkhole Prone Areas Proceedings, 2nd Multi-disc. Conference on Sinkholes and the Environmental Impact of Karst, Orlando, Florida: 437-445.

Boulanger R. W., J. D. Bray, S. H. Chew, R. B. Seed, J. K. Mitchell, and J. M. Duncan. 1991. SSCOMP: A Finite Element Analysis Program for Evaluation of Soil-Structure Interaction And Compaction Effects. Report No. UCB/GT/91-02. University of California, Berkeley.

Brauner G. 1973. Subsidence Due to Underground Mining. Information Circular No. 8571, U.S. Bureau of Mines.

Chen, D., and Peng S. S. 1981. Analysis of Surface Subsidence Parameters Due to Underground Longwall Mining in the Northern Appalachian Coalfield. Tech. Report 81-1, Department of Mining Engineering, West Virginia University.

Collin, J. G. 1986. Earth Wall Design. Ph.D. Thesis, University of Cailfornia, Berkeley.

Drumm E. C. , Ketelle R. H., Manrod W. E., and Ben-Hassine J. 1987. Analysis of Plastic Soil in Contact with Cavitose Bedrock Proceedings of the ASCE conference on Geotechnical Practice for Waste Disposal, Ann Arbor, Michigan.

Drumm E. C. , Kane, W. F., Ketelle R. H., Ben-Hassine J., and Scarborough J. A. 1990. Subsidence of Residual Soils in a Karst Terrain. Rep. No. ORNL/TM-11525, Oak Ridge National Laboratory, Tennessee.

Duncan J. M., R. B. Seed, K. S. Wong, and Y. Ozawa. 1984. FEADAM84: A Computer Program For Finite Element Analysis of Dams. Geotechnical Engineering Research Report No. SU/GT/84-03, Department of Civil Engineering, Stanford University.

Giroud, J. P., R. Bonaparte, J. F. Beech, and B. A. Gross. 1990. Design of Soil Layer-Geosynthetic Systems Overlying Voids. Geotextiles and Geomembranes, Vol. 9, No. 1:11-20.

Goegrinh R. L., and S. M. Sayed. 1989. Stabilizing A 'Mann'-Sized Sinkhole. Civil Engineering (12), ASCE: 52-53.

Handfelt, L. D., and J. A. William. 1988. Exploration of Krast Conditions in Central Florida, Geotechnical Special Publication No. 14, ASCE National Convention:40-52.

Human C. A., R. B. Seed, J. K. Mitchell, and Borja, R. I.1986. Predicted Behavior of the Stanstead Abbotts Trial Reinforcement. Proceedings of the Prediction Symposium on a Reinforced Embankment on Soft Ground, King's College, London.

Ruth, B. E., T. F. Beggs, and J. D. Degner. 1985. Prediction Sinkhole Collapse. Civil Engineering (11), ASCE: 58-60.

Schmertmann, G. R., S. H. Chew, and J. K. Mitchell. 1989. Finite Element Modeling of Reinforced Soil Wall Behavior. Report No. UCB/GT/891-01. University of California, Berkeley.

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