It seems highly unlikely that all 32 of these measurements were incorrect. Thus, the data for low-plasticity soils appear not only to be valid, but to constitute some of the most thoroughly documented data that are available.

## AREAS OF AGREEMENT

The writers do agree with the discussers in one aspect: quality control is perhaps the most important factor affecting successful construction of a soil liner. In fact, it was stated in the paper that "the improvement of performance obtained by high-quality construction far outweighs the benefits of simply increasing the thickness."

## CONCLUSION

Nothing in the discussion has caused the writers to change their conclusion that a reasonable minimum thickness for compacted soil liners is 0.6–0.9 m, or four to six lifts. Nevertheless, the writers do add a note of caution: the analyses, database, and conclusions are based on test pads and liners constructed with compacted, native soils. Furthermore, as noted in the paper, simply using the minimum recommended number of lifts does not itself guarantee low hydraulic conductivity. The materials used in building the liner must be appropriate, the construction procedures proper, and quality-assurance practices comprehensive.

## APPENDIX. REFERENCES


Benson, C., and Charbeneau, R. (1991). "Reliability analysis of time of travel in earthen landfill liners." *Proc., Geotech. Congr.*, ASCE, New York, N.Y., 456–467.

Benson, C., and Boutwell, G. (1992). "Compaction control and scale-dependent hydraulic conductivity of clay liners." *Proc., 15th Annu. Madison Waste Conf.*, University of Wisconsin, Madison, Wis., 62–83.

Benson, C. (1993). "Probability distributions for hydraulic conductivity of compacted clay." *J. Geotech. Engrg.*, ASCE, 119(3), 471–486.

Benson, C., Hardianto, F., and Motan, E. (1994a). "Representative specimen size for hydraulic conductivity assessment of compacted soil liners." *Hydraulic conductivity and waste contaminant transport in soils, ASTM STP 1142*, D. Daniel and S. Trautwein, eds., ASTM, Philadelphia, Pa., 3–29.

Benson, C., Zhai, H., and Rashad, S. (1994b). "Statistical sample size for construction of soil liners." *J. Geotech. Engrg.*, ASCE, 120(10), 1704–1724.

Benson, C., Zhai, H., and Wang, X. (1994c). "Estimating hydraulic conductivity of compacted clay liners." *J. Geotech. Engrg.*, ASCE, 120(2), 366–387.

Boutwell, G., and Rauser, C. (1990). "Clay liner construction." *Proc., Geotech. Engr. in Today's Envir.*, ASCE, New York, N.Y., 1–7.

Dagan, G. (1982). "Stochastic modeling of groundwater flow by unconditional and conditional probabilities: 1. conditional simulation and the direct problem." *Water Resour. Res.*, 18(4), 813–833.

Donald, S. (1990). "Stochastic analysis of compacted clay landfill liners," MSc thesis, University of Waterloo, at Waterloo, Ontario, Canada.

Draper, N., and Smith, H. (1981). *Applied regression analysis*. John Wiley and Sons, New York, N.Y.

Fenton, G., and Griffiths, D. (1993). "Statistics of block conductivity through a simple bounded stochastic medium." *Water Resour. Res.*, 29(6), 1825–1830.

Gutjhar, A., Gelhar, L., Bakr, A., and MacMillan, J. (1978). "Stochastic analysis of spatial variability in subsurface flows, 2. evaluation and application." *Water Resour. Res.*, 14(5), 953–959.

Kmet, P., Quinn, K., and Slavik, C. (1981). "Analysis of design parameters affecting the collection efficiency of clay lined landfills." *Proc., Fourth Annu. Madison Waste Conf.*, University of Wisconsin, Madison, Wis., 204–227.

Warren, J., and Price, H. (1961). "Flow in heterogeneous porous media." *Soc. of Pet. Engrs. J.*, 1(9), 153–169.


# Landfill Liner Interface Strengths from Torsional-Ring-Shear Tests[a]

## Discussion by N. Dixon[3] and D. R. V. Jones[4]

The authors have demonstrated clearly that the Bromhead ring-shear apparatus can be used to provide information on the residual shear strength of geosynthetic/soil interfaces. In addition, their findings on the nonlinearity of the failure envelope and recommendations on the use of either peak or residual interface strengths depending on the strain history of individual sectors of a potential failure surface are significant contributions. They have updated landfill-liner design considerations in line with accepted good practice for natural soil slope assessment [e.g., Bishop (1971)]. However, a number of factors relating to the ring-shear test procedure and the authors general conclusions require further discussion.

The Bromhead ring-shear apparatus was designed specifically to obtain the drained residual shear strength of remolded cohesive soils. Under normal test conditions the degree of compaction and initial moisture content of the soil are not critical: Drained residual shear strength being a fundamental property related to the particle-size distribution of the soil, the clay mineralogy, and pore fluid chemistry. The discussers' experience indicates that considerable difficulties can be encountered forming a uniform sample of specified density in the ring-shear sample container. The authors intention to develop a large ring-shear specimen container may reduce this problem.

Of particular concern is the use of the ring-shear test to obtain peak shear strengths. For some interfaces the values obtained may be in good agreement with the correct peak shear strengths; however, there is no guarantee that errors will be insignificant. Bromhead (1986) has stated that ring-shear devices are of little use for measuring the peak strength of soils, with errors in the order of 10% not being untypical. This is due to the nonuniformity of shear displacement across the sample that results in progressive failure, as discussed by the authors. There appears to be little validity in using a method of obtaining peak strengths that is known to be incorrect. For example, in the United Kingdom, where there is considerable experience with the Bromhead ring-shear apparatus, British Standard *BS 1377: Part 7* (1990) only covers the procedure for obtaining residual shear strength parameters from the ring-shear test. The authors adopted a high displacement rate of 44 mm/min in order to provide undrained conditions. It would be interesting to know what steps were taken to ensure that rheological or viscous shear strength effects were not present (Bromhead 1979).

The justification for the authors carrying out undrained ring-shear tests was the Kettleman Hills waste-repository failure. However, the causes and mechanisms of failure (i.e., the rapid loading of the lining system, and, hence, the limited time for the pore pressures in the clay to dissipate) are unusual, and may not be the worst scenario for design purposes in a large number of cases. There is also uncertainty over the role that the geosynthetic/clay interface has as a drainage path.


[a]March, 1994, Vol. 120, No. 3, by Timothy D. Stark and Alan R. Poeppel (Paper 4992).



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Interface shear strength tests carried out by Orman (1994), and presently being undertaken by the discussers, demonstrate that the shear displacement at interfaces between geosynthetic and soil are confined to a shear zone only a few millimeters deep. The close proximity of the shear surface to the soil/geosynthetic interface leads to the drainage status at the interface being critical. Undrained conditions will only exist if this interface is a no-flow boundary. Given the practical problems involved in placing large areas of geosynthetic, it seems unlikely that there will be intimate contact throughout. Preferential flow paths could be formed at the contact, thus providing drained, or partially drained, conditions during shearing. Large-scale direct-shear tests on textured geosynthetic/clay interfaces are being carried out at present by the writers. A number of these tests are incorporating pore-water–pressure measurements within the clay layer and at the interface. Monitoring of pore-pressure changes during the consolidation phases of the tests, using a normal stress of 100 kPa, has shown that the interface between the geosynthetic and clay can provide a drainage path.

The discussers agree with the authors' conclusion that the most important aspect of landfill stability analysis is the measurement and selection of the interface shear strength. Caution should be exercised when selecting drained or undrained parameters. Uncertainties about drainage conditions at the interface, and differences between operational phasing of sites relating to rate and type of loading, mean that considerable care should be taken in selecting design parameters. It may be necessary to use both drained and undrained, and peak and residual strength parameters in order to obtain the critical stability conditions, and, hence, ensure integrity of the design.

## APPENDIX. REFERENCES

Bishop, A. W. (1971). "Technical note: the influence of progressive failure on the choice of the method of stability analysis." *Géotechnique*, 21, 168–172.

"British standard methods of test for soils for civil engineering purposes Part 7. Shear strength tests (total stress)." (1990). *BS 1377: Part 7*, British Standards Institution, London, England.

Bromhead, E. N. (1986). *Stability of slopes*. Surrey University Press, 1st Ed., Surrey, England.

Orman, M. E. (1994). "Interface shear-strength properties of roughened HDPE." *J. Geotech. Engrg.*, ASCE, 120(4), 758–761.

## Closure by Timothy D. Stark[5] and Alan R. Poeppel[6]

The writers appreciate the comments of the discussers, who suggest that the peak strength measured in a torsional-ring-shear apparatus may not be in agreement with direct-shear values. A uniform shear stress distribution across the shear surface is assumed to compute the peak and residual shear stress. This assumption leads to the use of (3) for computing the average shear stress $\tau_a$ across the annular specimen

$$\tau_a = \frac{3(F_1 + F_2)L}{4\pi(R_2^3 - R_1^3)} \quad (3)$$

where $F_1$ and $F_2$ = loads on the proving rings; $R_1$ and $R_2$ = inner and outer radii of the specimen, respectively; and $L$ = distance between proving ring points of application. It is recognized that a nonuniform shear displacement occurs across the shear surface at the peak strength condition. This is caused by the difference in shear displacement between the inner and outer edges of the specimen. Hvorslev (1937, 1939) [reproduced in La Gatta (1970)] showed mathematically and experimentally that the maximum shear stress for cohesive soils computed from (3) is in agreement with the peak shear strength measured in direct-shear tests when an annular specimen with a ratio of $R_1/R_2$ greater than or equal to 0.5 is used. The ratio of $R_1/R_2$ is 0.7 in the Bromhead ring-shear apparatus, and thus the peak interface shear strength is usually in agreement with direct-shear values. If there is a difference in the peak strength values, the ring-shear apparatus will probably yield a slightly lower value because of the difference in shear displacement. In summary, the ring-shear apparatus appears to be a viable, cost-effective alternative for the measurement of peak and residual geosynthetic/geosynthetic and geosynthetic/soil interface strengths.

An unconsolidated-undrained ring-shear-test procedure was used for the clay/geomembrane interface to represent the field conditions at the Kettleman Hills waste repository. Mitchell et al. (1990) and Byrne et al. (1992) concluded that an unconsolidated-undrained condition is representative of the field conditions at Kettleman Hills. As a result, a shear displacement rate of 44 mm/min was used to simulate undrained shear conditions. No special steps were taken to ensure that rheological or viscous shear strength effects were not present. It should be noted that the ring-shear apparatus also is being used to measure drained interface strengths. If a soil/geosynthetic interface is involved, the procedure developed by Gibson and Henkel (1954) is used to estimate the drained shear displacement rate. Shear displacement rates for geosynthetic/geosynthetic interfaces vary depending on field conditions.

Finally, the writers want to reemphasize that the most important aspect of a landfill stability analysis is measurement and selection of interface shear strengths. As a result, the determination of the interface test conditions is of paramount importance in obtaining the proper interface strength for design. Some of the factors influencing the measured interface strength are the normal stress, drainage condition, shear displacement rate, hydration time and normal stress, and specimen size.

## APPENDIX. REFERENCES

Hvorslev, M. J. (1937). "Physical properties of remolded cohesive soils." *Translation No. 69-5*, U.S. Army Waterways Experiment Station, Vicksburg, Miss.

Hvorslev, M. J. (1939). "Torsion ring shear tests and their place in the determination of the shearing resistance of soils." *Proc., ASTM Symp. of Shear Testing of Soils*, Vol. 39, ASTM, Philadelphia, Pa., 999–1022.

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# IDENTIFICATION AND CHARACTERIZATION OF COLLAPSIBLE GRAVELS[a]

## Discussion by Robert W. Day,[5] Fellow, ASCE

The authors present an important paper on the collapse of natural deposits of gravel. The six case histories are inter-

[a]March, 1994, Vol. 120, No. 3, by Kyle M. Rollins, Ralph L. Rollins, Trevor D. Smith, and George H. Beckwith (Paper 5563).

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