Use of residual shear strength parameters for slope stability analysis against surficial failures



Venkata S. Dronamraju, Anand J. Puppala & Sarwenaj Ashraf The University of Texas at Arlington, Arlington, Texas, USA

ABSTRACT

Surficial slope failures occur on earth dams due to infiltration of rainfall after prolonged dry spells. Drained and residual conditions prevail in the desiccated zones of the slopes. Torsion ring shear tests were conducted on soil samples of earth dam. These results were used to perform stability analyses of surficial failures. The results depict that importance of mode of failure surface and the effect of depth of desiccation zone on the extent of failure. Minimum drained cohesion and friction angles needed to prevent surficial failures are determined and presented. Need for surficial stabilization to mitigate these failures are discussed.

RÉSUMÉ

Les échecs de pente de Surficial se produisent sur les digues de terre en raison de l'infiltration de chute de pluie après les périodes sèches prolongées. Les conditions égouttées et restantes prédominent dans les zones desséchées des pentes. Les épreuves de tondage d'anneau de torsion ont été accomplies sur les échantillons de sol de digue de terre. Ces résultats ont été utilisés pour exécuter des analyses de stabilité d'échecs surficial. Les résultats représentent cette importance de mode de surface d'échec et de l'effet de profondeur de zone de dessiccation sur la mesure d'échec. Cohésion égouttée minimale et friction.

1 INTRODUCTION

Stability of slopes has been a growing concern to Geotechnical engineers in the backdrop of landslides and slope failures of earth dams. Failures of natural slope are often associated with loss of life and property. Failures of earth dams due to seepage have proved to be occasionally catastrophic in nature. Earth dams and highway embankments are also susceptible for another kind of slope failure known as surficial failure or shallow skin slides. These failures are classified as shallow slope failures as the average depth of failure varies from 0.3 m to 1.2 m (Day, 1996). In many cases the failure surface is parallel to the slope face (Day, 1996). However, it was observed that many of the slope failures of earth dams have circular arc or curvilinear failure surfaces.

Surficial failures of earth dams constructed of cohesive soils are often triggered by rainfall. Long periods of rainfall can cause deep seated global failures and one or two days of intense rainfall can cause shallow failures (Duncan et al. 2005). The surficial failures may not lead to a catastrophic failure, if they are identified and repaired quickly. The ideal way of preventing surficial failures is to exercise proper care at the time of design and construction. The slopes can also be strengthened or stabilized during maintenance to prevent such failures wherever necessitated. There is a need to prevent these failures as they weaken the dam besides involving huge repair costs which can run into millions of dollars (McClesky, 2005).

During design of an earth dam, stability of dam is checked for safety at the end of construction, long term stability, sudden drawdown and earthquake conditions (Duncan et al. 2005). However, no specific analysis is often carried out in respect of surficial failures of earth dams at the time of design. This paper illustrates a methodology to analyse the surficial failures of man made slopes.

A typical surficial failure that occurred at Joe Pool Dam in Texas, USA is shown in Figure 1.



Figure 1. Surficial Failure of Joe Pool Dam, USA

Joe Pool Dam is located 15 miles southwest of Dallas in the state of Texas in USA. The dam was constructed between 1979 and 1986 and impounding started in January 1986. The length of dam is about 7.4 km and maximum height of dam is 33 m. The dam has a down stream side slope of 2.8H:1V (20°). The soil of the dam is classified as CH soil and it has a Plasticity Index of 37. The activity of the soil is around 3.2.

A research investigation was undertaken at the University of Texas at Arlington with a main objective of mitigating the surficial failures by stabilizing the slopes with various surficial soil treatment methods to mitigate shallow slope failures. As a part of the research, comprehensive laboratory studies were first conducted on the core soil and treated core soil. A large database of various shear strength properties is thus determined. These properties are used in various hypothetical slope stability studies using a commercial slope stability software program, GSTABL.

As a part of the analysis, the minimum amount of friction angle required to ensure safety of the dam against surficial failures is determined. The analysis is used to explain how these methodologies at the time of design will help in preventing the surficial slope failures. Few guidelines are framed to carryout this analysis as a supplement to complete slope stability analysis.

2 MECHANISM OF SURFICIAL FAILURE

Clayey soils are subjected to seepage, swelling, shrinkage, creep, formation of cracks, and development of slickensides that influence their shear strength (Duncan et al. 2005). Shear strength of soil of earth dam also predominantly depends on local weather conditions. During dry season, desiccation cracks form on the dam slope. When the soil is dry, shear strength of a soil of slope is generally high due to matric suction as the soil is in unsaturated condition.

During intense rainfall, Infiltration of rainwater results in formation of wetted zone near the slope surface and this may lead to failure during prolonged rainfall (Cho et al. 2002). The desiccation cracks accelerate the infiltration process and the soil gets saturated resulting in increase of pore pressure and decrease of soil suction. This phenomenon results in an overall reduction of effective normal stress, thus reducing the shear strength of soil (Chen et al. 2004, Brand, 1981). Alternate wetting and drying cycles also result in creep of clay and thereby a reduction of shear strength. Apart from reduction of shear strength, the shear stresses increase when cracks are filled with infiltrated water.

With continued rainfall, seepage develops parallel to the slope (Day, 1996). The rainwater induced moisture results in swelling of clay which results in increase of void ratio and permeability.

Under these circumstances, drained conditions typically prevail in the desiccation zone parallel to slope. As the drained cohesion parameter tends to approach zero, the shear strength of a soil decreases and resisting shear strength is only dependent on drained friction angle.

The problem aggravates in case of overtopping levees as the buoyant weight of saturated soil in the desiccation zone reduces the drained shear strength increasing the vulnerability to surficial failure. A combination of one or more of these conditions leads to surficial failures of earth dams and embankments.

3 RESIDUAL SHEAR STRENGTH CONDITIONS

Residual shear strength of a soil represents the lowest possible shear strength that a soil exhibits at large shear strains and this mobilized strength plays a significant role in the stability analyses. A slope does not fail if the applied shear stress is less than the residual shear strength (Gilbert et al. 2005). Residual shear strength of a cohesive soil is applicable to slopes that have already undergone a shear failure (Stark et al. 2005). Once sliding has occurred in a clay, the clay particles become reoriented parallel to the slip surface, and the strength decreases progressively reaching a low residual value (Duncan. 1996).

Slickensides develop in clays due to shear on distinct planes of slip from the realignment of clay particles on the same planes (Meehan et al. 2008). Friction angle on the slickensided surfaces is termed as the residual friction angle (Duncan et al. 2005). Skempton showed that once a failure has occurred and a slickensided failure surface has developed, only residual shear strength is available to resist failure.

The drained fully softened shear strength is another parameter used for evaluating first time slope failures (Stark et al. 2005). Strain softening of soil may lead to progressive failures. In case of a progressive failure, the soil particles along the failure surface remain in residual state and they will not attain its peak shear strength value (Duncan et al. 2005). The fully softened strength is measured by remoulding the clay in the laboratory at a water content equal to the liquid limit of the soil and then measuring its strength in a normally consolidated condition.

Mesri and Shagein (2003) showed that slopes in non homogeneous stiff clay and clay shale exhibit a residual strength along at least a portion of slip surface for first time slides. Many studies have further shown that during the first time slope failure, part of the slip surface is in residual condition. As such, the residual shear strength parameters are considered to be pertinent for carrying out surficial slope failure analysis.

3.1 Drained Conditions and Effective Stress Analysis

A slope failure may occur under drained or undrained conditions of soil (Duncan, 2005). Undrained strengths are important for short term loading and drained strengths are vital for long term loading conditions (Duncan et al. 1996).

Considering the mechanism of surficial failures, measurement of residual strength parameters under drained conditions in the desiccation zone is considered to be vital for study of surficial failures. The shear strength of a soil is obtained by equation 1 where c' and \emptyset ' are effective drained cohesion and effective drained friction angle and σ ' is the effective stress.

$$\tau = c' + \sigma' \tan \varphi'$$
[1]

The effective stress analysis is used to calculate the stability of the slope against surficial failure.

4 MEASUREMENT OF DRAINED RESIDUAL SHEAR STRENGTH PARAMETERS

Mobilization of residual strengths is generally associated with large displacements or shearing times. Therefore, laboratory measurement of residual strengths is difficult using conventional direct shear apparatus. Ring shear tests are preferable for measuring residual shear strengths as unlimited displacement is possible in ring shear apparatus (Stark et al. 1993, Duncan et al. 2005). This will allow soil particles to be oriented parallel to the direction of shear due to which residual shear strength is developed. Additional advantage is that the cross sectional area of shear surface is kept constant during shear.

In this research, Bromhead ring shear test apparatus was used to obtain the drained shear strength parameters of soil and soil treated with stabilizers. Test procedure prescribed by ASTM D 6467-99 was followed.

4.1 Description of Bromhead Ring Shear Apparatus

The equipment contains a shear device which holds the specimen securely between two porous inserts and provides a means of applying normal stress to the faces of the specimen, permitting drainage of water through the top and bottom boundaries of specimen. The device is capable of applying a torque to the specimen along a shear plane parallel to the faces of specimen. At the inner and outer walls of the specimen container friction is developed during shear.

The device is capable of shearing the specimen at a uniform rate of displacement. The rate of displacement can be selected using a combination of gear wheels from 44.52 mm/min. travel to 0.018 mm/min. travel. The specimen container is annular in shape with an inner diameter of 70 mm and outer diameter of 100 mm. the container radially confines the 5 mm thick soil specimen. Due to this confinement wall friction is developed at the inner and outer circumference of the specimen. The magnitude of the wall friction is the least at the top porous stone and the soil interface and increases with the depth of the specimen. Thus the failure plane occurs at the surface of the top porous stone where the wall friction is the least. This type of failure condition is referred to as *smear* condition (Bromhead, 1986).

4.2 Test Procedure

The equipment contains a Bromhead torsion ring shear device. The preparation of soil specimen and the set up for preconsolidation is shown in Figure 2. Soil specimen is prepared at a water content equal to the liquid limit and placed in the annular space of the bottom platen and the top platen is placed over it.

The specimen is pre-consolidated under a water bath at a load increment ratio of one at applied normal stresses of 25 kPa, 50 kPa, 100 kPa, 200 kPa and 400 kPa. For each load increment, it was ensured that primary consolidation was complete. In order to reduce the amount of horizontal displacement required to reach a residual condition, the specimen was pre-sheared at a constant rate of displacement of 18 mm/min.



Figure 2. Description of ring shear test procedure

After completing pre-shearing, identical soil specimens were sheared at various normal stresses and at a very slow rate of displacement. A rate of displacement of 0.58 mm/min travel was selected for the present tests. Slow rate of displacement allows dissipation of pore pressures and helps obtain realistic drained shear strength values. As the analysis was focussed on shallow slope failures, the results obtained at lower normal stresses is discussed as it simulates the field condition. For example, a depth of desiccation zone of 1 m undergoes a normal stress of 20 kPa to 25 kPa and the shear strength obtained directly represents the drained residual shear strength of soil in the field.

5 DISCUSSION OF RESULTS

Torsion ring shear tests were performed on the Joe Pool Dam soil. A typical test result showing the peak and residual values of soil is presented in Figure 3.

Test results indicated that the cohesion is approaching zero at low normal stresses and the drained friction angle is close to 20[°] as shown in Figure 4.



Figure 3. Displacement vs. Shear stress plot for soil at a normal stress of 100 kPa

The slope is found to be unstable at a drained cohesion of zero and drained friction angle of 20° . Slope stability studies yielded a failure in desiccated zone with a

factor of safety less than 1. As such, it was planned to strengthen the soil of Joe Pool Dam by adding admixtures to soil. Tests were hence performed on soils mixed with various proportions of lime and polypropylene fibers. Typical results for soil with admixtures of 8% lime and 4% Lime with 0.30% polypropylene fibers is also shown in Figure 4.

Addition of admixtures has improved the drained residual shear strength parameters. A drained residual cohesion of 10 kPa for 8% Lime and 15 kPa for 4% Lime with 0.30% fibers was obtained. The drained friction angle was observed to be 36[°] for 8% Lime and 40[°] for 4% Lime with 0.30% fibers.

It was noticed that the shear strength of soil obtained using direct shear test under drained conditions is higher than the results obtained using torsion ring shear test due to variations in shear strain levels imparted to the soil. Since surficial failures in the field associated with large shear strains, torsion ring shear test results were used in the present analysis.



Figure 4. Drained residual shear strength parameters

The results obtained for the Joe Pool Dam soil are compared with various previous studies. Previous studies based on stability analyses carried out for various types of soils reported that the drained friction angle varied from 5[°] to 40[°] (Mesri et al, 2003). It was also recommended to use effective stress cohesion of zero for residual and fully softened conditions (Stark et al. 2005). University of Missouri, Columbia conducted tests for Missouri Department of Transportation, USA on soil obtained from surficial failure sites of highway embankments and reported an effective cohesion of zero.

6 MODELLING OF JOE POOL DAM SLOPE

In the case of landslides, the shear over the narrow zone of failure is often termed as discrete shear zone which ranges from a few centimetres to about 1.2 m to 2.4 m thick (Cornforth, 2005). In large landslides, major portion of slide mass moves parallel to ground surface. As such, the infinite slope analysis is carried out. When the landslide is very long with respect to depth, the soil resistance contributed by the entry and exit slip surface may be minor (Cornforth, 2005). Landslides are also modeled as a double wedge or triple wedge to carry out the analysis. However, in the case of present earth dams, failure surface is modelled as a circular arc based on the field observations of various surficial failures of earth dams noticed in North Texas area. The circular failure surface is found to be more suitable for earth dams since the ratio of length to depth of surficial failure is small. The assumption of circular failure and use of bishop's failure circle gave a better insight in to the various intricacies involved with stability analysis of surficial failures.

Joe Pool Dam site with a slope angle of 20° (1V:2.8H) was modelled here using GSTABL program. Both the geometry of slope and soil parameters were given as input parameters. Bishop slip circle method was used for the analysis. The program considers points within the range specified for initiation and termination of slip circle and calculates the factor of safety for all slip circles developed by specifying number of points within the range and number of circles specified to be initiated from each point. The ten slip circles having least factor of safety are plotted besides indicating them in the order of increasing factor of safety. The circle having the least factor of safety is displayed in red.

For the initial analysis, drained cohesion was taken as zero and drained friction angle was taken as 15[°] for the desiccated zone. An undrained cohesion parameter of 80 kPa and friction angle of 20[°] was considered for the unsaturated zone below the desiccation zone. As the analysis was considered for surficial failures which are shallow and local in nature, the effect of saturated zone below phreatic line was ignored. Based on the pattern of previous failures, it was assumed that the depth of desiccation zone to vary from 0.30 m to 2 m. The actual depth of desiccation zone however was dependent on a combination of factors including type of soil, permeability, angle of slope, intensity of rainfall and temperature fluctuations and more research is needed in this area.

6.1 Maximum Length of Failure

Usually, the program is used for analysis of global slope failures and local deep seated failures by specifying a wide range of initiation points. In order to obtain the failure surfaces within the shallow desiccation zone, a trial and error method was followed. An interval of 1 m was used to input for left and right initiation point. Similarly an interval of 1 m was used for left and right termination limits. The number of initiation circles was taken as 10 and the number of failure surfaces from each initiation point was taken as 5 and the length of failure surface line segment was taken as 1 m.

To obtain the maximum length of failure surface, the interval between initiation point and termination point was increased. Figures 5 and 6 show the output of stability analyses for a slope of $2.8H:1V (20^{\circ})$ with a desiccation depth of 1 m and embankment height of 15 m.

It can be seen from the results that for any slope for a specified depth of desiccation zone, there is a maximum critical length beyond which the factor of safety increases as the failure surface intersects with the unsaturated zone that exhibit high strength parameters. Failure can occur for any length less than the critical length at any location on the slope.



Figure 5. Failure circle for 20° slope with 1 m desiccation zone and length of failure less than 18 m

For the above example, the maximum length of failure was around 18 m. For a length of failure surface less than 18 m, the factor of safety was observed to be 0.74 as shown in Figure 5. When the failure length increases, the failure circle intersected the unsaturated zone and the resulting failure surface was a deep seated failure with higher factor of safety of 6.72 as shown in Figure 6.





Figure 7. Variation of maximum length of failure for 20° slope

In the similar manner, the maximum lengths of failure for different depths of desiccation zones ranging from 0.3 m to 2 m are computed and these results are plotted in Figure 7.

Maximum length of slope prone for failure is 18 m for 20[°] slopes and this length is reduced to 13 m for 30[°] slopes and around 7 m for 40[°] slopes for a desiccation depth of 1 m. From the stability analyses results, it appears that this maximum length of failure is constant for a given depth of desiccation zone. This is valid for dams whose lengths of slopes are more than the maximum length of failure. In cases where length of slope was shorter, full length of slope is prone to failure. For example, for 6 m or higher embankments with a slope of 20[°] and a depth of 1 m desiccation zone, the maximum length of failure for circular arc type failure is around 18 m. If the height of embankment is less than 6 m, the failure can occur for the entire length of slope.

Analysis is also performed to assess whether the surficial failures extend into the crest of dam or embankment. It was noticed that the factor of safety increased when the failure surface extends to the crest. This is attributed to the fact that slip surface intersected the unsaturated zone below desiccation zone. As such, it could be concluded that the surficial slope failures do not extend much into the crest of dam.

6.2 Back-calculation of shear parameters to ensure safety using computer software

The computer software is further utilised to determine the minimum value of drained friction angle and drained cohesion of soil to ensure that the slope is safe against surficial failures. Slope analysis was performed for the maximum length of failure surface of 18 m and for a desiccation depth of 1m. The analysis is performed for normal condition when the slope is not submerged and for submerged condition. For submerged condition, the water table is aligned along the slope surface for the dam having a slope angle of 20°.



Figure 8. Effect of friction angle on factor of safety

For a 20[°] slope in normal conditions, it is noticed that the minimum drained friction angle (with zero drained cohesion) required to achieve a factor of safety of 1.3 is 25[°]. This factor of safety is reduced when the soil is under submerged condition.



Figure 9. Variation of factor of safety (a) c' = 2 kPa and (b) c' = 4 kPa

Similar analyses were repeated by including drained cohesion parameters. Figures 9 (a) and (b) show the variation of factor of safety with respect to drained friction angle when the drained cohesion is varied from 2 kPa to 4 kPa. In order to achieve the same factor of safety, for the soils having higher cohesion, the required drained friction angle can be small. For a 20° slope, it is noticed that for a minimum cohesion of 5 kPa, a factor of safety of more than 1.3 is possible even with a drained friction angle of zero. As such it can be concluded that under drained conditions failure does not occur as long as soil possesses either drained cohesion of 5 kPa or a drained friction angle of 25° or a combination of both. It is noted that this requirement of minimum drained parameters increases with an increase in slope angle.

In general, drained cohesion value for most of the soils approaches zero or in some cases it may attain a minimum value. The safety of slope of dam against surficial failure also depends on drained cohesion parameter. Since the effective stress is small at shallow depths, the influence of an increase in frictional angle on drained shear strength is minimal. However, any enhancement of drained cohesion will lead to small, but sufficient shear strength increase that can resist surficial failures. As such the experiments were also performed on soils treated with lime and polypropylene fibers which have shown improvement in both drained cohesion and friction angle. These treatments are expected to reduce surficial failures in the dam slopes.

7 NON-CIRCULAR FAILURE SURFACES PARALLEL TO SLOPE

The slope is also modelled using a non-circular slip surface assuming the failure surface to be parallel to slope. Field observations reveal that the failures parallel to slope for the entire length of slope are occurring in the case of low height embankments. Figure 10 shows a typical failure photograph where a failure occurred parallel to slope almost for entire length of a highway embankment of an exit ramp in Dallas, Texas, USA.



Figure 10. Failure of highway embankment in Texas, USA

Using the same computer program, the dam slope is modelled with a failure surface parallel to slope both for a part of the slope (as shown in Figure 11) and for the entire surface (as shown in Figure 12).



Figure 11. Failure circle for 20[°] slope with failure surface parallel to slope for part length



Figure 12. Failure circle for 20[°] slope with failure surface parallel to slope for entire length

The factor of safety in both cases was close to 0.77 and this value was close to factor of safety value of 0.74 which was obtained when circular arc failure analysis was used. This indicated that both wedge type and circular type analyses for surficial failures will yield similar factor of safety values as long as the failure is confined to desiccation zone.

8 VALIDATION OF RESULTS

A detailed analysis of surficial failures occurred in Joe Pool Dam and Grapevine Dam in the state of Texas maintained by the United States Army Corps of Engineers was carried out. As per the available data almost all the failures were preceded by different spells of rainfall events.

Majority of the failures were occurred on the down stream side slope nearer to crest. Torsion ring shear tests were performed on soil samples obtained from the dam location near a shallow slope failure. The results have shown a drained friction angle of 20[°] with a zero cohesion value. The extent of failures occurred on Grapevine dam are compared with the results of circular failure analysis. The observed length of failures along the slope is 10 m for 0.6 m deep failure, 14 m for 1.2 m failure and 21 m for 1.8 m deep failure (McClesky, 2005). This indicates that the results shown in Figure 7 closely match with the field observations.

University of Missouri, Columbia have conducted another study for Department of Transportation. They studied surficial failures occurred at highway embankments having slopes ranging from 2.5H:1V to 2.2H:1V at various locations (Loehr and Bowders, 2007). Failure for entire length of slope was reported at I-435 site where the height of bank was about 4.6 m. Failure occurred at Emma site was only for a part length of slope where the height of bank was 6.7 m and slope was ranging from 2.5H:1V to 2.2H:1V. The extent of failures could not be verified for steeper slopes as most of the failures studied are having slopes flatter than 24[°] for earth dams and highway embankments.

Considering the above analysis, and based on the field observations it is recommended that the circular type failure surface analysis can be used for the earth dams and high embankments while assessing the safety with respect to surficial failures.

9 SUMMARY AND CONCLUSIONS

Surficial slope failures occur due to infiltration of surface runoff from rainfall events, which can lead to drained conditions in the desiccated zones. Due to large strains in these zones, drained and residual conditions often prevail and hence drained residual soil parameters are vital to carryout the stability analysis. Important findings from this study are listed here:

- Analyses with both circular and wedge type failure surfaces yielded similar factor of safety values for surficial failures. Hence, conventional circular arc type failure analysis is recommended for earth dams and high embankments. This type of analysis is also in agreement with the field observations of surficial failures of a few dams and highway embankments in USA. The factor of safety increases once the failure circle intersects the unsaturated zone where the strength parameters are higher than those of the saturated desiccation zone. The failure circle may not extend much into the crest and most of the failures occur on slope portion of earth dams and embankments.
- Residual shear strength parameters of soil using torsion ring shear device are needed to carryout surficial slope failure analysis of earth dams and embankments.
- Drained friction angle controls the computed factor of safety of surficial failures as drained cohesion of untreated embankment fills are often close to zero.
- Small increase in drained cohesion of the surficial desiccated zone can significantly enhance the factor of safety values. Hence surficial treatments with additives such as lime and fibers are highly recommended to mitigate surficial failures.
- The maximum length of failure is constant for a given depth of desiccation zone. This is valid for dams whose lengths of slopes are more than the maximum length of failure. In cases where lengths of slopes are shorter, full length of slope is prone to failure.

ACKNOWLEDGEMENTS

The writers would like to acknowledge Les Perrin, PE, and Ken McCleskey of the United States Army Corps of Engineers, Fort Worth, Texas, USA for their assistance and supervision of the on-going research on mitigation of surficial failures of earth dams.

REFERENCES

- Bell, F.G. 1996. Lime Stabilization of Clay Minerals and Soils, *Engineering Geology*, 42:1996, 223-237.
- Cho E.S. and Lee R.S. Evaluation of Surficial Stability for Homogeneous Slopes Considering Rainfall Characteristics, *Journal of Geotechnical and Geoenvironmental Engineering*, September 2002: 756-763.
- Cornforth, H.D. 2005. Landslides in Practice Investigation, Analysis and Remedial/Preventive Options in Soils, Hoboken, N.J., John Wiley & Sons, 2005.
- Esrig, I.M., McKenna, E.P. 2001. Lime Cement Column Ground Stabilization for I-15 in Salt Lake City, *Practice Periodical on Structural Design and Construction*, 6(3), 104-115.
- Day, W.R. 1996. Design and Repair for Surficial Slope Failures, *Practice Periodical on Structural Design and Construction*, 83-87.
- Duncan, J.M. and Wright, G.S. 2005. *Soil Strength and Slope Stability*, Hoboken, N.J., John Wiley & Sons, 2005
- Gilbert, R.B., Najjar, S.S. and Shields, M.K. 2005. Importance of Residual Strengths in Factors of Safety and Reliability, *GRI- 18 Geosynthetics Research and Development in Progress.*
- Gilbert, R.B. and Byrne, R.J. 1996. Strain-Softening Behaviour of Waste Containment System Interfaces, *Geosynthetics International*, 3(2), 181-203.
- Gnanendran, C.T. and Selvadurai, A.P.S. 2001. Strain measurement and interpretation of stabilizing force in geogrid reinforcement, Journal of *Geotextiles and Geomembranes* 19: 177-194.
- Ingold, T.S. and Miller, K.S. 1983. Drained axisymmetric loading of reinforced clay, *Journal of Geotechnical Engineering*, ASCE, 109: 883-898.
- Leshchinsky, D. and Perry, E.B. 1987. A design procedure for geotextile reinforced walls, *Geosynthetics '87*, IFAI, New Orleans, LA, USA, 1: 95-107.
- Loehr, E.J. and Bowders, J.J. 2007. Slope stabilization using recycled plastic pins – Phase III, Final Report R198-007D, Prepared by *Missouri Transportation Institute and Missouri Department of Transportation*, January 2007.
- McCleskey, K.L. 2005. Experimental Investigations to Select Stabilization Methods to Mitigate Desiccation Cracks in order to Reduce Slope Failures, University of Texas at Arlington, 1-138.
- Meehan, L.C., Brandon, L.T. and Duncan J.M. 2008. Measuring "Fast" shear strengths along slickensided surfaces in the bromhead ring

Shear, *Geotechnical Testing Journal*, 31: 239-242.

- Mesri, G. and Shahien, M. 2003. Discussion and Closure of Residual Shear Strength Mobilized in First-Time Slope Failures, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, May 2004, 544-549.
- Rogers, C.D.F. and Glendinning, S. 1997. Improvement of Clay Soils in situ using Lime Piles in the UK, *Engineering Geology*, 47: 243-257.
- Stark, D.T. and Contreras, A.I. 1998. Fourth Avenue Landslide during 1964 Alaskan Earthquake, *Journal of Geotechnical and Geoenvironmental Engineering*, 124(2): 99-109.
- Stark, D.T., Choi, H. and McCone, S. 2005. Drained Shear Strength Parameters for Analysis of Landslides. *Journal of Geotechnical and Geoenvironmental Engineering*, 131(5): 575-588.
- Skempton, A.W. 1970. First-time Slides in Over Consolidated Clays, *Geotechnique*, 20(3): 320-324
- Skempton, A.W. 1985. Residual Strength of Clays in Landslides, Folded Strata and the Laboratory, *Geotechnique*, 35(1): 3-18.
- Skempton, A.W. 1997. Slope Stability of Cuttings in Brown London Clay, *Proceedings of 9th Int. Conference on Soil Mechanics and Foundation. Engineering*, Tokya, 3: 261-270
- Tiwari, B., Brandon, L.T., Marui, H.T. and Gyanu R. 2005. Comparison of Residual Shear Strengths from Back Analysis and Ring Shear Tests on Undisturbed and Remoulded Specimens, *Journal of Geotechnical and Geoenvironmental Engineering*, 131(9): 1071-1079.
- Tiwari, B., Marui, H.T., Gyanu, R. and Marui, H.T. 2005. Variation in Residual Shear Strength of the Soil with the Salinity of Pore Fluid, *Journal of Geotechnical and Geoenvironmental Engineering.* 131(12): 1445-1456.
- Terzaghi, K., Peck, R.B. and Mesri, G.1996. Soil mechanics in engineering practice, 3rd ed., N.Y, John Wiley & Sons, USA, 1996