Int. J. of Geosynth. and Ground Eng. (2016) 2:23
DOI 10.1007/s40891-016-0063-5

CrossMark

ORIGINAL ARTICLE

Frictional and Interface Frictional Characteristics of Multi-layer Cover System Materials and Its Impact on Overall Stability

S. K. Yamsani[1] · S. Sreedeep[1] · R. R. Rakesh[2]

Received: 17 May 2016 / Accepted: 16 July 2016 / Published online: 6 August 2016

Abstract The municipal and hazardous landfills nearing its design capacity need to be isolated from the atmosphere using multi-layered cover system (MLCS). These MLCS constitutes different layers of soil and geosynthetics with widely varying properties. Each of these layers fulfil specific requirements by acting as a surface protection, drainage, separation, filtration and hydraulic barrier layers of MLCS. Failure of these MLCS leads to waste–atmosphere interaction and results in extremely hazardous situation to biosphere. The stability of MLCS significantly depends on the shear strength characteristics of materials used viz., the internal frictional characteristics of individual soil materials and interface frictional characteristics of soil–soil or soil–geotextile combination. In view of this, frictional characteristics of four type of soils, four soil–geotextile interfaces and one soil–soil interface was determined using direct shear and modified direct shear testing methods. The modification of geomembrane used in barrier layer was also attempted, for improving its interface shear characteristics. The usefulness of the above parameters and the influence of its variability on the slope stability of MLCS of a near surface low level radio-active waste disposal facility (NSDF) is demonstrated in this study.

Keywords Interface shear · Soil · Geotextile · Modified direct shear test · Multi layered cover system (MLCS) · Stability

✉ S. Sreedeep
srees@iitg.ernet.in

S. K. Yamsani
y.sudheer@iitg.ernet.in

R. R. Rakesh
ravirr@barc.gov.in

[1] Department of Civil Engineering, Indian Institute of Technology Guwahati, Guwahati, India

[2] CE & QAS/Nuclear Recycle Group, Bhabha Atomic Research Centre, Mumbai, India

Introduction

Engineered landfills are mandatory for the safe disposal of hazardous and non-hazardous municipal wastes [1]. Liners and covers form the main components of landfill system, which isolates the waste components from the surrounding subsurface and atmosphere. The engineered covers are mandatory for isolating the harmful effects of non-engineered and indiscriminate surface waste dump yards which are quite common in developing countries like India [2, 3]. In general, the purpose of the cover is to minimize the impact of variations in surface atmospheric conditions like high temperatures and heavy rainfall on contained wastes. Due to these cyclic variations in weather a lot of fatigue load is transferred to the cover materials. For this reason, a combination of various soil and geosynthetics layers are used as multi layered cover system (MLCS) [4]. The MLCS is constructed with an appropriate slope to drain out rain water easily and quickly. The choice of slope angle is governed by the workability requirement and available land area.

Frictional and interface frictional characteristics are the important parameters governing the stability of sloped MLCS [5–8]. Moreover, it is presumed that interface layers (the plane between two different types of materials) are the weakest planes more prone to slip or failure due to the lack of proper interaction [9–11]. Failure at Kettleman hill landfill and others signifies the importance of interface frictional characteristics in interpreting stability of MLCS [12–14]. This makes it important to investigate the frictional and interface frictional characteristics of various materials used in MLCS construction and to determine its effect on slope stability.

Site reconnaissance explains that most of the MLCS failed at their initial stages immediately after the construction or sometimes during construction phase [15, 16], mainly due to the weak friction and interface friction characteristics of materials used in construction of MLCS. Therefore, the present study attempts to understand the role of shear and interfacial shear strength of materials on the overall stability of sloped MLCS. The effect of climatic variations with time and impact of rainfall on the stability of MLCS has not been considered in this study.

MLCS Configuration

MLCS essentially consists of a surface layer for protecting the hydraulic barrier below from the undesirable climatic changes and other anthropogenic factors. Immediately below the surface layer is the drainage layer for collecting the infiltrated rain water and routing it outside the MLCS for minimizing hydraulic head over the hydraulic barrier. A nonwoven or a composite geotextile is provided as a separator to resist internal erosion (piping of fines) at the interface of surface layer and drainage layer. Below the drainage layer a low permeable thick clay barrier is provided to resist further infiltration. In some cases, an additional geomembrane or geocomposite clay liner is provided for further safety. In such scenario, a thin layer of sand is laid over geomembrane to protect from puncture due to sharp particles in drainage layer. Finally, a foundation layer is made at the bottom of all these layers using well compacted local soils to take care of the differential settlements of MLCS (due to waste degradation). There are numerous alternate configurations of MLCS adopted depending on the type of waste isolated and the local site scenario [17, 18]. A typical configuration of MLCS considered in this study is presented in Fig. 1 with detailed description of different layers and their interfaces.

Materials

As shown in Fig. 1, a locally available moderately permeable red soil was proposed as surface protection layer with its permeability (k) less than 10^{-7} m/s. A high permeable ($k \approx 10^{-5}$ m/s) fine gravel was used as drainage layer, a well permeable composite geotextile material was used for filtration and protection of surface layer from piping, a composite geomembrane material having low permeability ($k < 10^{-12}$ m/s) is used as an additional barrier, a fine–medium sand was provided below drainage layer material for protecting geomembrane from gravels, a thick layer of low permeable ($k < 10^{-12}$ m/s) bentonite clay is used as a barrier to restrict water flow and locally available fill material is included as the foundation layer. The foundation material in this study is made of same soil used in surface layer. The materials used in this study were conforming to the recommendations of USEPA [19] as MLCS. For providing adequate shear resistance, low permeable smooth high density polyethylene (HDPE) geomembranes were sandwiched between two thin geo-nets to form composite geomembrane [20]. Fine sand and geo-net layers above geomembrane also helps in transmitting surface infiltration, thereby reducing hydraulic head [4].

The geotechnical characterisation of selected materials are summarised in Table 1. The grab tensile strength of various geotextiles, HDPE geomembrane and geonet collected from local sources were tested as per ASTM D 4632 [21] recommendations and corresponding results are depicted in Fig. 2. The composite geotextile and composite geomembrane (geomembrane sandwiched between two geo-nets) with maximum tensile strength satisfying the strength recommendations of AASHTO M288 are selected for their use in filter and barrier layers of MLCS, respectively [4]. The basic properties of selected geosynthetics are summarised in Table 2.

Methodology

For determining the shear characteristics of soil materials, a 6 cm × 6 cm small direct shear box was employed while a 30 cm × 30 cm large direct shear box was used to determine the shear characteristics of gravels and interface shear characteristics as recommended by IS 2720 Part-39 [22] and ASTM D 3080 [23], respectively. The strain rates of shear tests were varied over a wide range based on the permeability of materials for ensuring proper dissipation of excess pore pressures as per the recommendations of [23, 24]. Direct shear test is a robust device for quickly evaluating the shear characteristics of given soil material [25–27]. Also direct shear test can be easily modified to determine interface shear characteristics, replacing one half with different material. It helps in better representation of field condition, with its predefined failure surface. Several studies reported in the past have used modified direct shear test to determine interface shear tests of various soil–soil or soil–geotextile or geotextile–geotextile interface shear characteristics [28–30].

All the shear tests were conducted after prior saturation of the soil compacted at optimum moisture content and

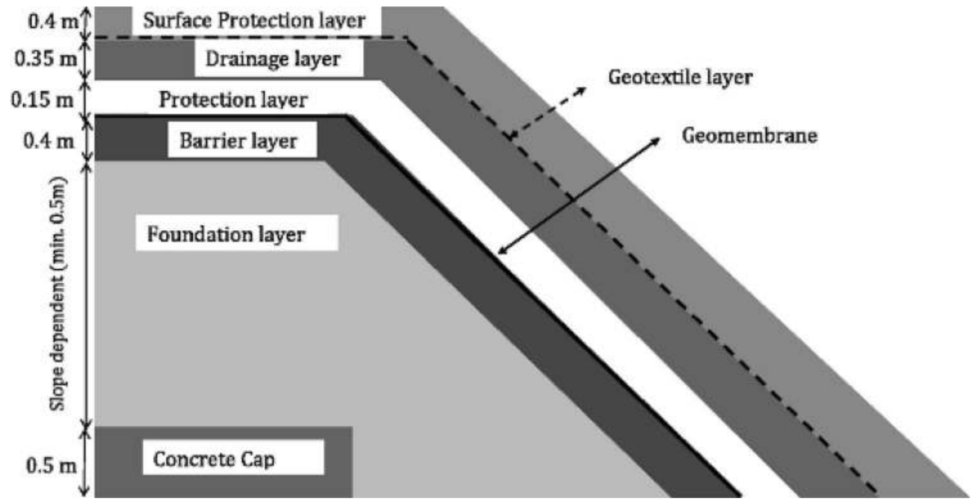


Fig. 1 Typical configuration of MLCS

Table 1 Geotechnical characterisation of selected soil materials

Material properties	Red soil (RS)	Bentonite (BS)	Gravel (GS)	Sand (SS)
Atterberg limits (%)				
Specific gravity	2.69	2.72	2.57	2.67
Hygroscopic water content	5.45	11.57	1.5	2.4
Liquid limit	44	395	–	–
Plastic limit	22	39	–	–
Shrinkage limit	19	11	–	–
Plasticity index	22	356	–	–
Particle size distribution (%)				
Coarse sand (2.00–4.75 mm)	22.00	–	Passing 12.5 mm and retained on 10 mm	1.30
Medium sand (0.425–2.00 mm)	33.64	–		68.7
Fine sand (0.075–0.425 mm)	28.04	4.4		29.72
Silt (0.002–0.075 mm)	9.83	38.77		0.28
Clay (<0.002 mm)	6.65	56.83		–
Compaction characteristics				
USCS classification (ASTM D2487-98)	SM	CH	GP	SP
Maximum dry density (g/cm^3)	1.7	1.33	Cohesion less soils	
Optimum moisture content (%)	18.44	32		
Minimum density (g/cm^3)	Cohesive soils		1.341	1.486
Maximum density (g/cm^3)			1.4	1.67
Saturated hydraulic conductivity (m/s)	3×10^{-8}	1×10^{-12}	1×10^{-4}	3×10^{-6}

maximum dry density. The SS and GS are compacted at their corresponding maximum relative density. For SS and GS the shearing started after 1 h of loading (as they don't under go much consolidation) while in soils it was sheared after loading for 24 h. In the case of BS, wetting beyond 3 days caused uncontrolled swelling, leading to protrusion of swollen BS from predefined failure plane. Hence BS was sheared after 3-day saturation considering it to be worst possible field scenario resulting in undrained shear characteristics. Duncan [31] suggested to consider the soils having permeability more than 10^{-6} m/s (e.g., for SS and GS) as drained materials while soil having permeability less than 10^{-9} m/s (e.g., BS) as undrained.

In determining interface shear characteristics of soil–soil interface, one half of the direct shear box was filled with one type of soil and the other half with another. While

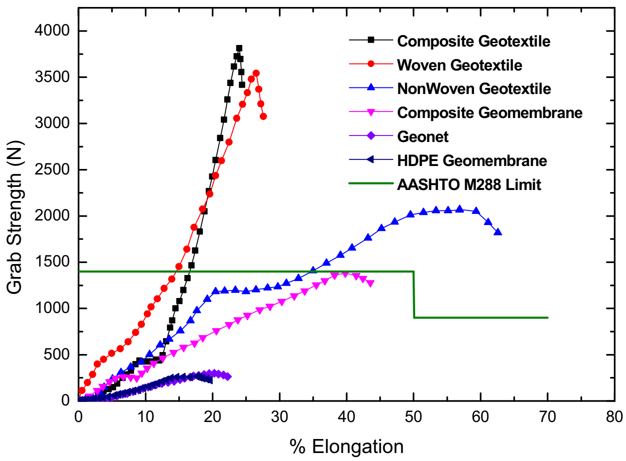


Fig. 2 Grab tensile strength properties of various geosynthetics

determining interface shear characteristics of soil–geotextile interface, geotextile was rigidly fixed to lower half of the direct shear box and in the other half soil was filled at desired compaction state. Geotextile was wrapped to a wooden box which fits perfectly into the modified direct shear [7, 8].

Limit equilibrium method (LEM) and finite element method (FEM) are commonly adopted in evaluating stability of earthen structures [32]. Due to the lack of stress–strain consideration in LEM, researchers suggest to adopt FEM for stability analysis [31, 33]. Nevertheless, FEM also have some limitations like convergence issue in using nonlinear soil models, defining appropriate Poisson's ratio and hence researchers suggest to use LEM stability analysis along with FEM stresses for attaining reliable results [33]. The stresses calculated by FEM is used for the determination of factor of safety (FOS) using LEM. For this purpose, the numerical modelling was performed using Geostudio, one of the most common tool used in geotechnical practice [34–37]. In this study, Sigma-W module of Geostudio was used to determine stresses. The materials are specified at the density and water content values listed in Table 1, the elasticity characteristics of soils are defined according to the soil classification described in Table 1 [38–40]. These stresses were incorporated in Morgenstern–Price LEM for attaining FOS in

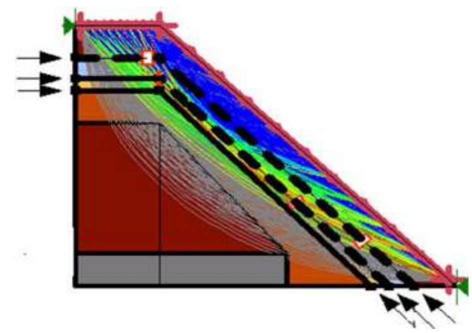


Fig. 3 Typical configuration of circular slips encompassing critical slope section of MLCS

Slope-W module of Geostudio [41]. For additional comparison and gaining more confidence of final FOS, stability analysis using stresses calculated by LEM is also adopted. The effect of rainfall and seepage was not considered in this analysis. In both the methods, the entry–exit slip definition was used to define about 500 slip surfaces covering entire slope section as shown in Fig. 3. The arrows represent different reinforcements applied at the interfaces as already described in the Fig. 1.

Results and Discussion

Shear Characteristics of Soil Materials

The selected samples were sheared under three different normal stresses (59, 108 and 157 kPa) and the corresponding peak shear stresses were obtained. The variation of shear stress versus normal stress are plotted as shown in Fig. 4 and the corresponding shear characteristics are summarised in Table 3. The effect of particle size is clearly visible with increasing strength for soils with relatively large size particles [7]. However, the variation in particle size in this study is in a broad range. In case of sand, at lower normal stress of 59 kPa a small amount of shear stress increment is observed indicating the dilation under lower normal stresses, which gets suppressed at higher normal stresses [42]. No significant dilation can be seen in case of gravels, attributed to the crushing of particles associated with shearing [43].

Table 2 Basic characteristics of selected geosynthetic materials

Material properties	Composite geotextile (CGT)	Composite geomembrane (CGM)
Mass per unit area (g/m^2)	658.34	219.638
Apparent opening size (mm)	0.133	No pores
Tensile strength (N)	3813.67	3542.25
% Elongation at peak	23.98	26.52
Permeability (m/s)	1.50×10^{-6}	5.51×10^{-15}

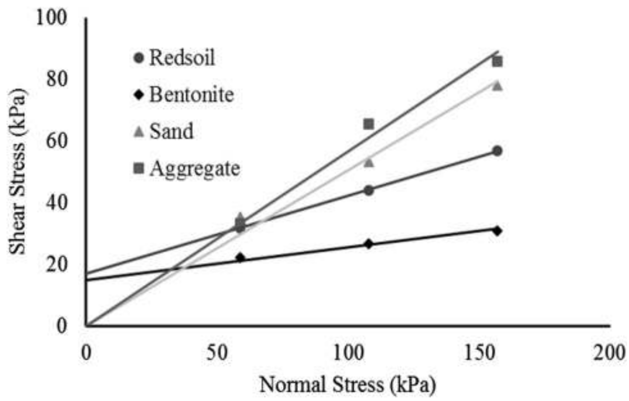


Fig. 4 Variation of shear stress with normal stress for various materials

Table 3 Shear parameters of various materials

Material types	Cohesion, c (kPa)	Angle of frictional resistance, φ (°)
Red soil	16.96	14.2
Bentonite	14.34	6.2
Gravel	0	29.5
Sand	0	26.9

Interface Shear Characteristics

The interpretation of interface shear testing results are similar to that of conventional shear testing. The shear stress versus normal stress variation of different interfaces were plotted in Fig. 5 and the corresponding interface shear characteristics are summarised in Table 4.

Koutsourais et al. [5] has observed a wide range of interface friction angles varying for various soil–geosynthetic combinations. The interface friction values of any particular soil–geosynthetic interface depends on numerous factors like the soil internal friction angle, geosynthetic tensile capacity, sample conditions, loading rates, etc. The cohesive soil interaction with geosynthetics as observed in red soil–geotextile and bentonite–geomembrane interactions, has resulted in adhesive resistance along the interface similar to Athanasopoulos [44]. The interface shear resistance observe in case of red soil–geotextile interaction are observed to be in the range of cohesive soil (of similar frictional resistance) interactions with different geotextiles observed by Athanasopoulos [44]. The geomembrane–bentonite interface friction is observed to be same as textured geomembrane–clay interaction observed by Fishman and Pal [45]. However, the adhesion value is observed to be lower than their case. This can be attributed to the difference in clay contents of the soils and also to the change in geomembrane textures. Also, the modification of relatively smooth HDPE geomembrane surface using thermally bonded geonet has shown better interface shear resistance similar to Stark and Newman [20]. The only soil–soil interaction observed in the current situation is the gravel–sand interface underneath drainage layer. The visual examination of gravel–sand interface after shear testing revealed that about two gravel particle thickness (approximately 20–25 mm) penetrated into the sand layer. This observation recommends minimum thickness of twice the average particle size of gravels as sand layer above geomembrane. Such a sand layer would effectively minimize the possibility of drainage layer gravels puncturing the geomembrane.

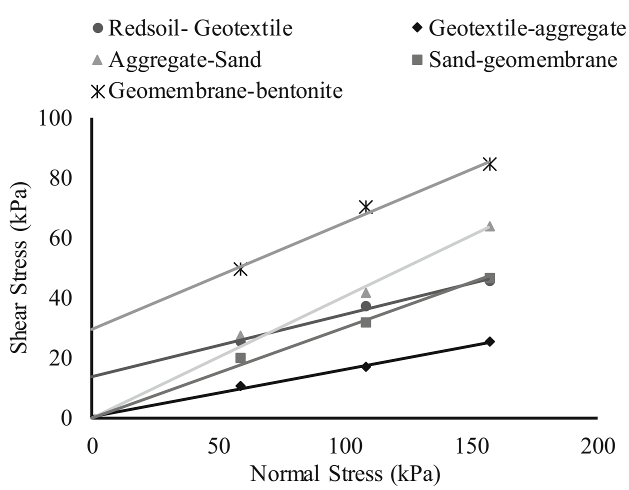


Fig. 5 Variation of shear stress with normal stress for various interface components of MLCS

Stability Analysis

Near surface disposal facilities containing low level hazardous waste are in general sealed in concrete caps. Ideally, the MLCS is provided over this concrete cap for additional and long-term safety as illustrated in Fig. 1. However, numerous such concrete caps are installed adjacent to each other and sometimes may have space constraints. To aid practicing engineers with different alternatives, three slope configurations with slopes (a) 1V:1H, (b) 2V:3H and (c) 1V:2H were considered in this study. For present study a concrete cap of 5 m × 10 m area is considered for determining the stability of MLCS provided over it. The stability analysis will be symmetric about the centre of the two dimensional slope system and hence only half of section is analysed to reduce the computational efforts. The stability of all the three slope configurations are evaluated using Geostudio 2012 considering the material characteristics detailed in Tables 1, 2, 3 and 4. Due concern is made to maintain the slope angle and the minimum thickness of every layer as described in Fig. 1. This is achieved by

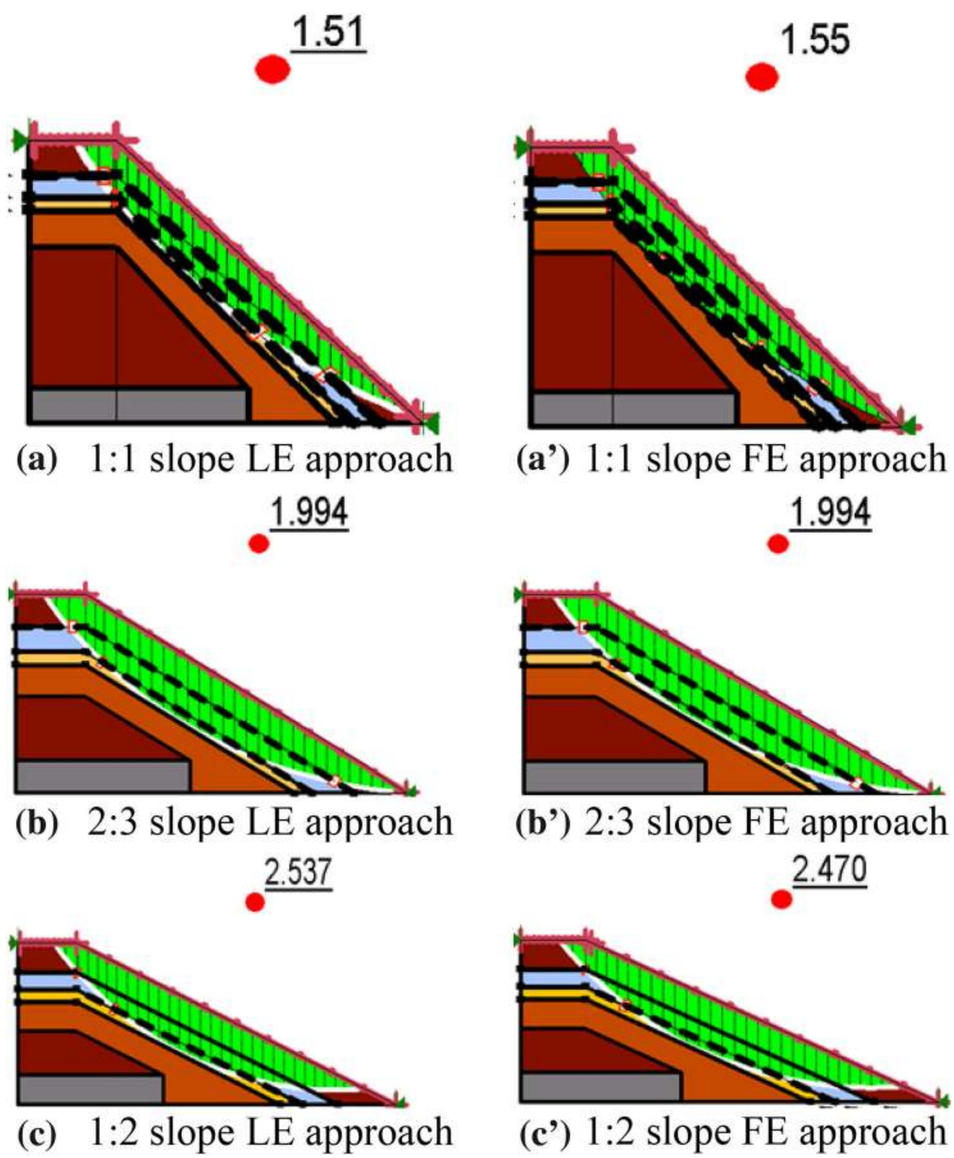
Table 4 Interface shear characteristics of various cover components

Interface	Adhesion, a (kPa)	Interface friction angle, δ (°)
Red soil–geotextile	13.7	11.7
Geotextile–gravel	0	9.1
Gravel–sand	0	22.1
Sand–geomembrane	0	16.7
Geomembrane–bentonite	29.4	19.6

considering tapered layers with increasing width in the base for lower slope angles. Moreover, increase in base width of individual layers of MLCS would increase the overall stability of the system [4].

The results of all the analysis are summarised in Fig. 6 where the critical slip surface at failure, for different trials are depicted. From results it can be clearly observed that there is no significant difference in FOS between LE and FE methods, similar to [46]. The FOS varied from 1.51 to 2.537 with change in slope from 1V:1H to 1V:2H. The general phenomenon of increasing FOS with decreasing slope can be observed along with the shift of critical slip from toe to face. All the cases considered in this study gives satisfactory FOS of more than 1.5. As the MLCS is

Fig. 6 Critical slip surface of various slope sections analysed in the study



expected to safely protect low level hazardous waste disposal facility for long duration of time, a higher FOS of 2.5 as attained in 1V:2H slope can be considered the best alternative.

Detail observation of critical slip surface in all the slope configurations, depicts that the sand–geomembrane interface and the thin sand layer appears to be the weak zones through which the base of critical slip surface traverses. For understanding the effect of the sand material on FOS, a sensitivity analysis is adopted using LE approach for all the slope configurations. The results of sensitivity analysis are summarised in Fig. 7. As seen in the figure, the FOS is observed to increase with increasing strength of the sand present in the weak zone. For the highest slope of 1V:1H the MLCS slope is observed to be safer for friction angle of sand greater than 26.9°. Within the variation of friction angle from 20.9° to 32.9° (26.9° ± 6° for understanding sensitivity of sand properties) the FOS of MLCS varied from 1.35 to 1.91 for 1V:1H slope, 1.76 to 2.51 for 2V:3H slope and from 2.44 to 3.44 for 1V:2H slope, respectively, i.e., relatively the smaller slopes were observed to be more sensitive to the variation in strength of material in weak zone.

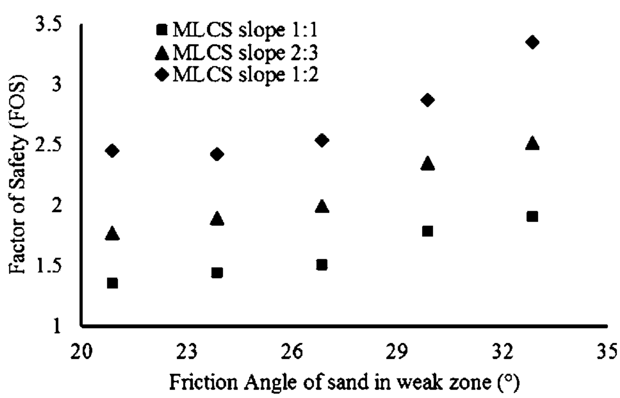
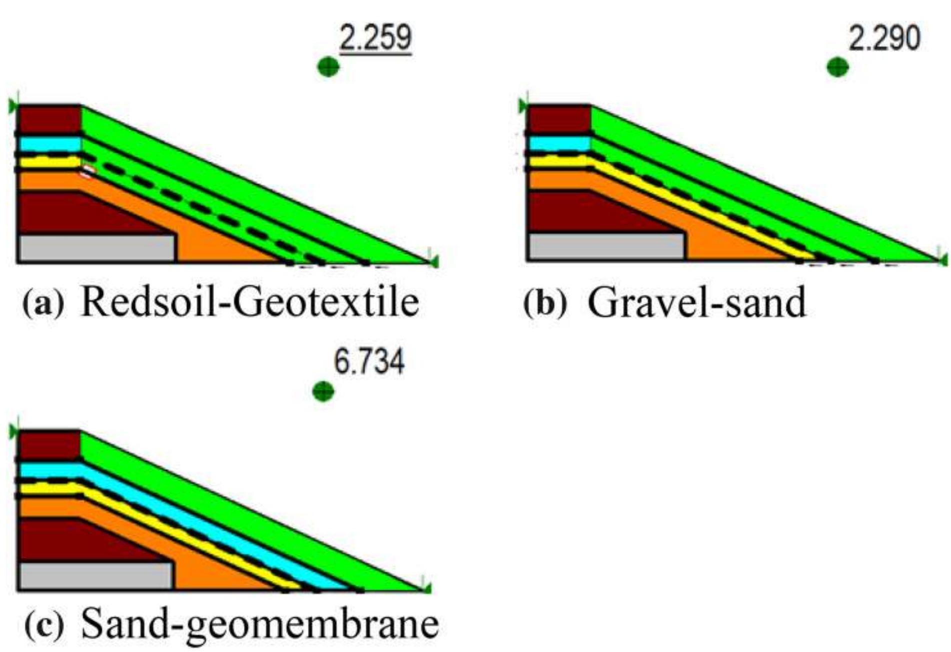


Fig. 7 Sensitivity analysis at the base of critical slip surface

Further, to emphasise the effect of interface shear characteristics on the stability of MLCS three fully specified slip surfaces are defined along the interfaces of the different materials. The properties of the materials thickness, tensile strength, interface shear characteristics are defined as determined in Tables 2 and 4. However, there is no specific option available in Geostudio to define the interface shear characteristics of soil–soil interface, this was therefore described as reinforcement material with zero tensile strength and interface shear characteristics as described in Table 4. The FOS of three interfaces red soil–geotextile, gravel–sand and sand–geomembrane interfaces are found to be 6.734, 2.290 and 2.259, respectively as shown in Fig. 8. The slope was observed to fail at sand–geomembrane interface similar to earlier observations, and the critical FOS was observed to decrease from 2.537 to 2.259. From this it is reaffirmed that the sand–geomembrane interface is the weakest plane amongst the MLCS considered in this study. It is quite apparent that this FOS will reduce when the climatic factors such as precipitation is taken into account for slope stability. Such a study need to be performed in detail by considering the effect of different rainfall events on the stability of sloped MLCS.

Fig. 8 Fully specified slip surfaces along the interface between different layers



Conclusions

This study evaluated the stability of sloped MLCS attributed to the shear strength and interface shear strength characteristics of different materials used in its construction. The materials used in this study satisfy USEPA recommendations. Shear and interface shear characteristics of various cover components were systematically evaluated and summarised. The recommendation for safe thickness of sand layer over geomembrane is made. The materials used in this study can be used safely for MLCS configuration with a slope of 1V:2H. The shear and interface shear characteristics of materials used in this study satisfy the stability requirement of selected MLCS with a FOS greater than 1.5. The sensitivity analysis performed at the base of critical slip surface has shown an increase in FOS with increase in the frictional resistance of material present in weak zone. Relatively the smaller slopes were observed to be more sensitive to variation in strength of material in weak zone. The redefined slip configuration along the interfaces has shown a decrease in FOS and proven that sand–geomembrane interface is the weakest amongst all.

Acknowledgments The authors are grateful to the Board of Research in Nuclear Sciences (BRNS), Department of Atomic Energy (DAE), India (Project No. 2013/36/06-BRNS) for funding the research presented in this paper.

References

1. Eid HT (2011) Shear strength of geosynthetic composite systems for design of landfill liner and cover slopes. Geotext Geomembr 29(3):335–344
2. Sharholy M, Ahmad K, Mahmood G, Trivedi RC (2008) Municipal solid waste management in Indian cities—a review. Waste Manag 28(2):459–467
3. Chakrabarti M, Dubey A (2015) Remediation techniques, for open dump sites, used for the disposal of municipal solid waste in India. J Basic Appl Eng Res 2(17):1510–1513
4. Koerner RM, Daniel DE (1997) Final covers for solid waste landfills and abandoned dumps. ASCE Press, Reston
5. Koutsourais MM, Sprague CJ, Pucetas RC (1991) Interfacial friction study of cap and liner components for landfill design. Geotext Geomembr 10(5):531–548
6. Stark TD, Poeppel AR (1994) Landfill liner interface strengths from torsional-ring-shear tests. J Geotech Eng 120(3):597–615
7. Vangla P, Latha GM (2015) Influence of particle size on the friction and interfacial shear strength of sands of similar morphology. Int J Geosynth Ground Eng 1(1):1–12
8. Choudhary AK, Krishna AM (2016) Experimental investigation of interface behaviour of different types of granular soil/geosynthetics. Int J Geosynth Ground Eng 2(1):1–11
9. Koerner RM, Hwu BL (1991) Stability and tension considerations regarding cover soils on geomembrane lined slopes. Geotext Geomembr 10(4):335–355
10. Dixon N, Jones DRV, Fowmes GJ (2006) Interface shear strength variability and its use in reliability-based landfill stability analysis. Geosynth Int 13(1):1–14
11. Bergado DT, Ramana GV, Sia HI (2006) Evaluation of interface shear strength of composite liner system and stability analysis for a landfill lining system in Thailand. Geotext Geomembr 24(6):371–393
12. Mitchell JK, Seed RB, Seed HB (1990) Kettleman Hills waste landfill slope failure. I: Liner-system properties. J Geotech Eng 116(4):647–668
13. Ling HI, Leshchinsky D (1997) Seismic stability and permanent displacement of landfill cover systems. J Geotech Geoenviron Eng ASCE 123(2):113–122
14. Siebecker B (2005) When final caps fail. Waste 360 Mag. http://waste360.com/mag/waste_final_caps_fail. Accessed 7 March 2016
15. Daniel DE, Gross BA (1996) In: Rumer RR, Mitchell JK (eds) Caps, Section 6, assessment of barrier containment technologies, PB96-180583. National Technical Information Services (NTIS), Springfield, p 119–140
16. Paruvakat N, Richardson GN (1999) Landfill cover failure prompts standards upgrade. In: Designer's Forum. Geotechnical fabrics report, September, vol 17, no. 7, p 20–24
17. Cooper C, Mira R, Low T (2012) Draft record of decision for parcel E-2. Department of Navy, San Francisco
18. United States Department of Energy (2000) Innovation technology summary report on Alternative Landfill Cover. Sandia National Laboratories report, DOE/EM-0558
19. USEPA (1989) Final covers on hazardous waste landfills and surface impoundments, technical guidance document, EPA/530/SW-89/047. United States Environmental Protection Agency, Office of Solid Waste and Emergency Response, Washington, DC
20. Stark TD, Newman EJ (2010) Design of a landfill final cover system. Geosynth Int 17(3):124–131
21. ASTM D4632-08 (2008) Standard test method for grab breaking load and elongation of geotextiles. Annual book of ASTM standards, vol 04.08. ASTM International, West Conshohocken
22. IS 2720 Part 39 (1977) Methods of test for soils: direct shear test for soil containing gravel. Bureau of Indian Standards
23. American Society for Testing and Materials, ASTM D3080/3080M (2011) Standard test method for direct shear test of soils under consolidated drained conditions. Annual book of ASTM standards, vol 04.08. ASTM International, West Conshohocken
24. IS 2720 Part 13 (1986) Methods of test for soils: direct shear test for soils. Bureau of Indian Standards
25. Anubhav, Basudhar PK (2013) Interface behavior of woven geotextile with rounded and angular particle sand. J Mater Civ Eng ASCE 25(12):1970–1974
26. Nakao T, Fityus S (2008) Direct shear testing of a marginal material using a large shear box. Geotech Test J 31(5):561
27. Kim BS, Shibuya S, Park SW, Kato S (2012) Effect of opening on the shear behavior of granular materials in direct shear test. KSCE J Civ Eng 16(7):1132–1142
28. Christopher BR (1991) Geotextiles in landfill closures. Geotext Geomembr 10(5):459–470
29. Liu SH, Sun DA, Matsuoka H (2005) On the interface friction in direct shear test. Comput Geotech 32(5):317–325
30. Sayeed MMA, Ramaiah BJ, Rawal A (2014) Interface shear characteristics of jute/polypropylene hybrid nonwoven geotextiles and sand using large size direct shear test. Geotext Geomembr 42(1):63–68
31. Duncan JM (1996) State of the art: limit equilibrium and finite-element analysis of slopes. J Geotech Eng 122(7):577–596
32. Chakraborty R, Dey A (2016) Numerical investigation of slope instability induced by hydraulic and seismic actions. In: Proceedings of Northeast students geo-congress on advances in geotechnical engineering, NESGC 2016

33. Krahn J (2003) The 2001 RM Hardy Lecture: the limits of limit equilibrium analyses. Can Geotech J 40(3):643–660
34. Gharabaghi B, Singh MK, Inkratas C, Fleming IR, McBean E (2008) Comparison of slope stability in two Brazilian municipal landfills. Waste Manag 28(9):1509–1517
35. Collins BD, Sitar N (2010) Stability of steep slopes in cemented sands. J Geotech Geoenviron Eng ASCE 137(1):43–51
36. Turer D, Turer A (2010) A simplified approach for slope stability analysis of uncontrolled waste dumps. Waste Manag Res 29(2): 146–156
37. Harris SJ, Orense RP, Itoh K (2012) Back analyses of rainfall-induced slope failure in Northland Allochthon formation. Landslides 9(3):349–356
38. Arora KR (1987) Soil mechanics and foundation engineering. Standard Publishers Distributors, Delhi
39. Potts DM, Zdravkovic L, Zdravkovi L (2001) Finite element analysis in geotechnical engineering: application, vol 2. Thomas Telford Publications, London
40. Head KH, Epps R (1986) Manual of soil laboratory testing, vol 3. Pentech Press, London, pp 798–869
41. Geoslope International (2012) Geostudio: user's guide for Sigma-W and Slope-W, version 8.0. Geoslope International, Calgary
42. Houlsby GT (1991) How the dilatancy of soils affects their behaviour. University of Oxford, Department of Engineering Science, Oxford, pp 27–30
43. Arslan H, Baykal G, Sture S (2009) Analysis of the influence of crushing on the behavior of granular materials under shear. Granul Matter 11(2):87–97
44. Athanasopoulos GA (1996) Results of direct shear tests on geotextile reinforced cohesive soil. Geotext Geomembr 14(11):619–644
45. Fishman KL, Pal S (1994) Further study of geomembrane/cohesive soil interface shear behavior. Geotext Geomembr 13(9):571–590
46. Krishna AP (2006) Slope stability analysis by limit equilibrium and finite element methods. Doctoral Thesis, Norwegian University of Science and Technology (NTNU)