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Effect of cement treatment on soil non-woven geotextile interface

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Numerical modeling.

Abstract. It is imperative that any soil improvement technique can considerably affect the soil media mechanical behavior. However, considering the recent researches, it can be concluded that a combined study on the effect of cement treatment on soil-geotextile interfacial shear strength parameters has almost been neglected. Thus, the main objective of this study is to fill this research gap with the main focus on a selected site in Shiraz city, Iran. In this regard, shear strength parameters of untreated and cement treated soil samples have been acquired by traditional and modified direct shear tests apparatuses. The results indicate that at high cement contents, the soil-geotextile interfacial shear strength increases with an observed behavior similar to over consolidated soils. While up to 1% of cement treatment did not improve the properties, adding 5% and 10% of cement increased both friction angle and cohesion of the soil considerably. Complementary microscopic evaluation of the interface indicated that the interfacial soil-geotextile shear strength is highly dependent upon the soil particles size distribution and specially its fines content. Finally, the numerical modeling of an illustrative reinforced soil structure revealed that knowing the state of stress is a prerequisite to any selection of the soil improvement zone.

1. Introduction

As far as safety is concerned, geotechnical engineers have always sought to include major governing factors in the geomechanical analysis of soil structures. Generally speaking, any significant soil treatment or inhomogeneity in the soil media would impose considerable effects on the design process. As an instance for soil treatment, cement has been used to improve soil characteristics by many researchers including Lasisi et al. [1], Walker [2], Kolias et al. [3], Consoli et al. [4], Sariosseiri et al. [5] and Taheri et al. [6]. In general, geosynthetics are excellent choices for soil reinforcement and in particular geotextiles are the most favorite type for fine grained soil improvement cases. On the other hand, the behavior of geosynthetics and especially geotextiles is a clear-cut example for inhomogeneity in the soil media which has been widely studied following theoretical and experimental approaches (e.g. [7-14]).

In most applications, geotextiles are sandwiched between soils on both sides introducing new planar interfacial surfaces with almost unknown frictional characteristics. According to Krieger and Thamm [15], the soil-geotextile interfacial frictional characteristics (i.e. interface friction angle and adhesion) are key parameters in reasonable modeling of geotextile reinforced soil walls. Karpurapu and Bathurst [16], Rowe and Ho [17], Desai and El-Hoseiny[18], El-Sawwaf [19] and many other investigators have incorporated the interfacial friction parameters in reinforced soil numerical modeling including Mechanically Stabi-

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lized Earth (MSE) walls and slope stability applications.

Several methods have been introduced by researchers for determination of frictional parameters of the soil-geotextile interface. Inclined Plane Test (IPT) adopted by Wasti and Ozduzgun [20], Palmeira et al. [21], Narejo [22], Chung et al. [23], Wu et al.[24] and Pitanga et al. [25] and direct shear test employed by Fourie and Fabian [26], Garbulewski [27], Bouazza [28], Lee and Manjunath [29], Palmeira [30], Anubhav[31] and Bacas [32] are good illustrations of such methods.

Having considered the recent researches on soil-geotextile interface and soil-cement treatment, it can be concluded that a combined comprehensive study on the effect of cement treatment on soil-geotextile interfacial shear strength parameters has been almost neglected. Hence, the main objective of this paper is to evaluate the effectiveness of cement treatment on the geomechanical properties of geotextile reinforced soil structures with the main focus on a selected site in Shiraz city, located in South-West of Iran. From the geological point of view and according to local engineering practices and records, Shiraz plain is mostly composed of calcareous limestone and marlstone formations as well as alluvial deposits which are mostly found at North and North-East of Shiraz. Overall map, terrain view and locality of the sampling zone for current study have been demonstrated in Figure 1 (from Google Maps). In fact, high weathering potential of the local marlstones significantly degrades their strength parameters mostly along with extensive physical decomposition. Hence, concerning construction costs and material availability, in large projects, it is mostly preferred to modify and utilize in-situ weathered deposits instead of replacing them. Consequently, in this study, representative samples of local soil have been selected and experimentally treated by different percentages of cement modification to assess their level of strength parameter improvement.

Figure 1. Terrain view and locality of the sampling zone.

As opposed to inclined plane test, the direct shear test provides the possibility to control the amount of normal stress on the sliding surface. Hence, this method is more flexible for applying desired normal stresses. Hence, among previously discussed testing approaches, direct shear test as a simple method with wide domain of use among geotechnical engineers has been adopted for the current study. Different types of woven and non-woven geotextiles have been introduced at this stage. In Iranian geotechnical engineering practice, the non-woven geotextiles are more popular as they are cheaper while having wide domain of technical use (e.g. reinforcement, cushion, separation, drainage, etc.). Hence, samples of non-woven geotextile have chosen. Moreover, considering numerous commercially available non-woven geotextiles and regarding their unique material standards, a single representative non-woven geotextile has been used in current study. Subsequently, inherent and interfacial strength parameters of previously introduced cement treated soil samples in contact with the geotextile layer were acquired using traditional and modified direct shear test apparatuses, respectively. These results were then employed for numerical modeling of an illustrative reinforced soil structure to evaluate cement treatment influence on soil strength improvement.

In addition, the utilized non-woven geotextile samples have been thoroughly investigated by microscopic photography both before and after relevant direct shear tests to investigate their physical structure and modifications during tests.

The results of this analysis can be extensively used for the general soil categories presented over Shiraz city plain, including some detailed soil-geotextile interfacial characteristics which have not been reported yet.

2. Material modeling

Adopting the Mohr-Coulomb failure envelope, the shear strength in the soil sliding plane can be expressed as:

$$\tau_f = c + \sigma_n \tan\varphi, \tag{1}$$

where τ_f is the failure shear stress, c is the soil cohesion, σ_n is the normal stress, and φ is the soil friction angle. In the cases where one of the sliding sides is not composed of soil material (e.g. geotextile), the traditional Mohr-Coulomb failure criteria can be reformulated as below:

$$\tau_{\text{inter}} = c_{\text{inter}} + \sigma_n \tan \varphi_{\text{inter}}, \tag{2}$$

in which τ_{inter} is the interfacial failure shear strength, c_{inter} is the interfacial soil cohesion, σ_n is the normal stress and φ_{inter} is the interfacial soil friction angle. The contact efficiency ratio between the untreated soil and interfacial properties can be defined both for cohesion and friction angle:

$$R_c = \frac{c_{\text{inter}}}{c}, \tag{3}$$

$$R_{\tan\varphi} = \frac{\tan \varphi_{\text{inter}}}{\tan \varphi}, \tag{4}$$

where R_c and $R_{\tan\varphi}$ are the interfacial strength efficiency ratios for cohesion and friction angle, respectively.

3. Experimental studies

During the current research, different stages of experimental studies have been followed including test specimen preparation and characterization as well as performing traditional and modified direct shear tests. Properties presented hereafter have been obtained based on the individual test procedures introduced by American Society for Testing and Material (ASTM) standard for acquiring each value.

3.1. Soil materials

The basic properties of soil samples, from the sampling zone shown in Figure 1 and used in the current study, are presented in Table 1.

Grain size distribution of this soil is shown in Figure 2, demonstrating a fine grained particle size distribution with 68% passing #200 sieve.

Furthermore, standard type II Portland cement was selected for the subsequent soil-cement mixture preparation.

3.2. Sample preparation

Four specimen categories including an untreated and three types of cement treated samples were prepared for the current study. The untreated sample was remolded to provide its in-situ natural properties as shown in Table 1. The other samples were prepared in accordance with the details illustrated in Table 2. These specimens were prepared by static compaction at the natural soil water content. The unit weight was set to be the same as the natural soil to exclude weight effects in the subsequent numerical modeling and analysis.

Untreated soil samples were prepared by thoroughly mixing soil with sufficient water to produce the desired water content and allowing the soil to stand 18 hours prior to the test in accordance to the guides, illustrated in Table 3.

3.3. Geotextile samples

Samples of commercially available non-woven geotextile were selected for the current study. The specifications of these materials are presented in Table 4. The geotextile tensile strength values presented herein were individually verified through relevant laboratory

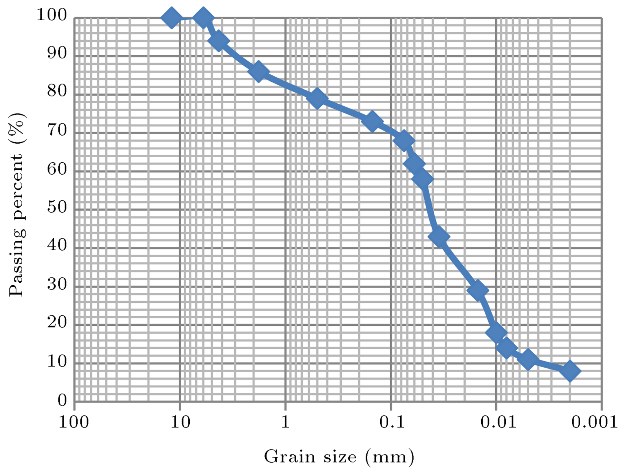


Figure 2. Grain size distribution of soil samples.

Table 2. Specifications of cement treated specimens.

Cement content	γ (kN/m^3)	w (%)	Curing period (days)
1%	19.8	16	3
5%	19.8	16	3
10%	19.8	16	3

Table 3. Soil stand time prior to test (adopted from ASTMD5321-12).

Classification (by ASTM D2478)	Minimum standing time (Hrs)
SW, SP	No requirement
SM	3
SC, ML, CL	18
MH, CH	36

Table 1. Soil material specifications.

Classification	γ_m (kN/m^3)	w_n (%)	c (kPa)	φ (deg)	LL	PL
CL	19.8	16	28	27	46	32

Table 4. Geotextile specifications incorporated for current study.

Type	Material	Tensile strength[1] (kN/m)	Thickness[2] (mm)
Non-woven	Polypropylene	22.5	5

[1]In accordance with EN ISO 10319;
[2]In accordance with EN ISO 9863-1.

tests to confirm the nominal values presented by the manufacturer.

3.4. Test program

Direct shear tests were carried out on specimens following ASTM D5321 (2012) recommendation in both square and rectangular shear boxes. The box dimensions suggested had to be greater than 300 mm (12 in), 15 times d_{85} of the soil used in the test, or a minimum of five times the maximum opening size (in plan) of the geosynthetic tested. The mentioned minimum container dimensions are guidelines based on requirements for testing most combinations of geosynthetics and soils. Containers smaller than those specified above can be used if it can be shown that data generated by the smaller devices contain no bias when compared to the minimum size devices specified. According to Jewell and Wroth [33], Palmeira [34], O'Rourke et al. [35], Takasumi et al. [36] and more recently Anubhav [31] and Tuna et al. [37], the shear box dimensions even for sands do not alter the results significantly. Other researches on fine-grained soils exhibit similar results for 300 mm and 60 mm shear boxes (e.g. [26,27]). In the current research the d_{85} and geotextile opening size are less than 1 mm and 0.1 mm, respectively. Consequently, the small shear box (6×6 cm) adopted for the current study satisfies the minimum requirements.

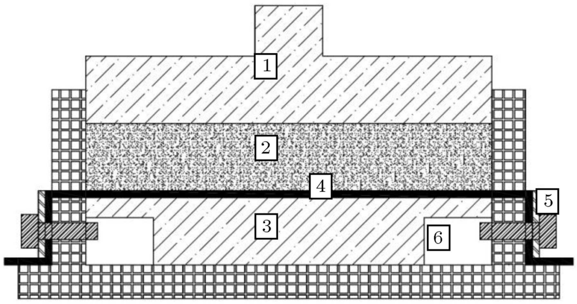
To begin with, tests were carried out in laboratory under controlled condition with air temperature at $21 \pm 2°$C and relative humidity between 50 and 70% utilizing an air conditioning unit.

The assembly of the modified shear box incorporated for the current study is shown in Figure 3. It can be seen that the soil container is located at the top of the geotextile layer which is tightly clamped to the lower half.

The test preparation initiates with fixing the geotextile layer to the lower half without any wrinkles or folds at the surface. Subsequently, the upper container was filled with soil material.

Before starting the tests, the upper half of the interface shear box was lifted 1 mm from the lower half to avoid particles from trapping between the mid gap and the geotextile, which could lead to false values of friction angle.

The assembly was then placed in the apparatus.



1	Loading cap	4	Geotextile sample
2	Soil sample	5	Clamping jaw
3	Dummy block	6	Void

Figure 3. Assembly of the modified shear box used in the current study.

The direct shear test apparatus was then checked and calibrated. Subsequently, force transducer, vertical and horizontal displacement Linear Variable Differential Transformers (LVDTs) were also connected to the computer via Autonomous Data acquisition Unit (ADU). All utilized transducers were calibrated individually and provided in accordance with the specifications and precision defined in ASTM D5321 (2012). Defining the experiment schedule in the controlling computer software, test was ready to commence.

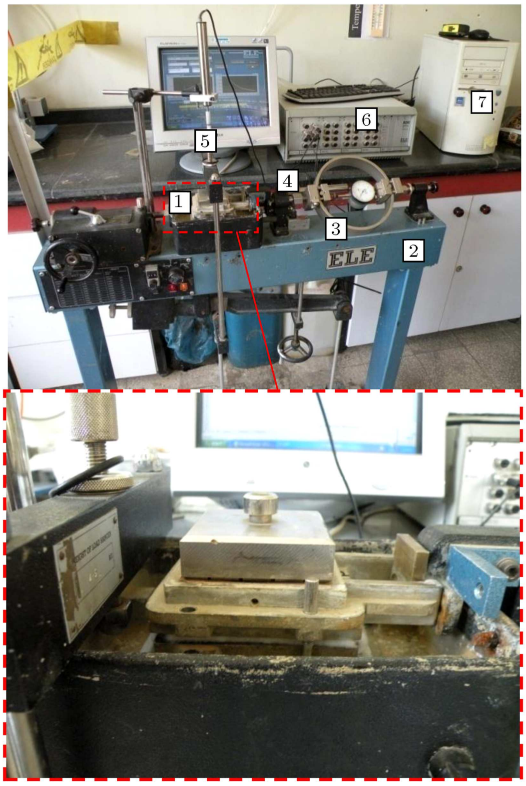
As the soil samples were at their natural unsaturated condition, no excess pore pressure was anticipated and hence the displacement rate of 1 mm/min was adopted. Test durations were also limited by the large deformation occurrence. Since the lower half was fully covered with geotextiles, there was no need for any area correction. Final assembly including the modified shear box and the relevant instrumentations is shown in Figure 4.

Three sets of conventional direct shear tests were conducted on the cement treated samples having the specifications presented in Table 2. Subsequently, soil-geotextile interfacial shear strength parameters and behavior were investigated utilizing the previously described modified direct shear test apparatus and according to ASTM D5321-12.

3.5. Test results

Results of soil-geotextile interfacial shear stress tests are shown in Figure 5(a), considering 3 different normal stress levels. The same curves are presented in Figure 5(b), (c) and (d) for 1%, 5% and 10% cement contents, respectively.

The results indicate that as the cement content of the soil increases, the shear strength of the interfacial layer increases as well with a behavior similar to over consolidated soils at high cement contents. In other words, cement treatment changes the soil-geotextile interfacial shear strength behavior from ductile and flexible to brittle and stiff state.



1	Modified soil-geotextile shear box	4	Horizontal displacement LVDT
2	Traditional shear test apparatus	5	Vertical displacement LVDT
3	Load measurement instrument	6	Data Acquisition Unit (ADU)

Figure 4. Modified shear box assembly and measuring instrumentations.

Table 5. Shear strength parameters for untreated and treated soil both for traditional and geotextile interfacial tests.

	C (kPa)	C_{inter} (kPa)	Φ (deg)	φ_{inter} (deg)
Untreated soil	28	13	27	18
Soil + 1% cement	27	13	27	17
Soil + 5% cement	89	51	31	22
Soil + 10% cement	221	165	36	30

Strength parameters obtained from untreated and cement treated soil tests both for conventional and geotextile interfacial tests are presented in Table 5.

3.6. Discussion on experimental results

The laboratory test results presented in Figure 5 and Table 5 can be expressed in terms of the strength improvement percentage in comparison to the initial properties. The improvement percentages for cohesion and friction angle are defined as below:

$$\text{imp}_c(\%) = \frac{c_{\text{treated}}}{c_{\text{untreated}}} \times 100, \tag{5}$$

$$\text{imp}_{\tan\varphi}(\%) = \frac{\tan\varphi_{\text{treated}}}{\tan\varphi_{\text{untreated}}} \times 100, \tag{6}$$

where imp_c and $\text{imp}_{\tan\varphi}$ are improvement percentages for cohesion and friction, respectively. The subscript "treated" stands for the cement treated parameter and the "untreated" subscript represents the untreated natural soil parameter.

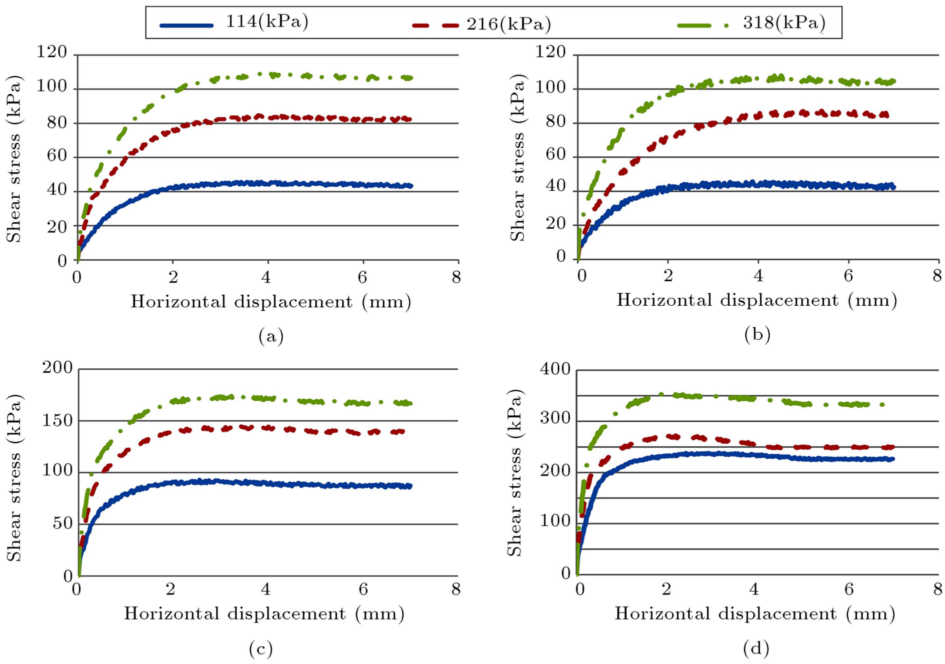


Figure 5. Shear stress vs. horizontal displacement for interfacial properties between geotextile and (a) untreated soil, (b) 1% cement content, (c) 5% cement content, and (d) 10% cement content.

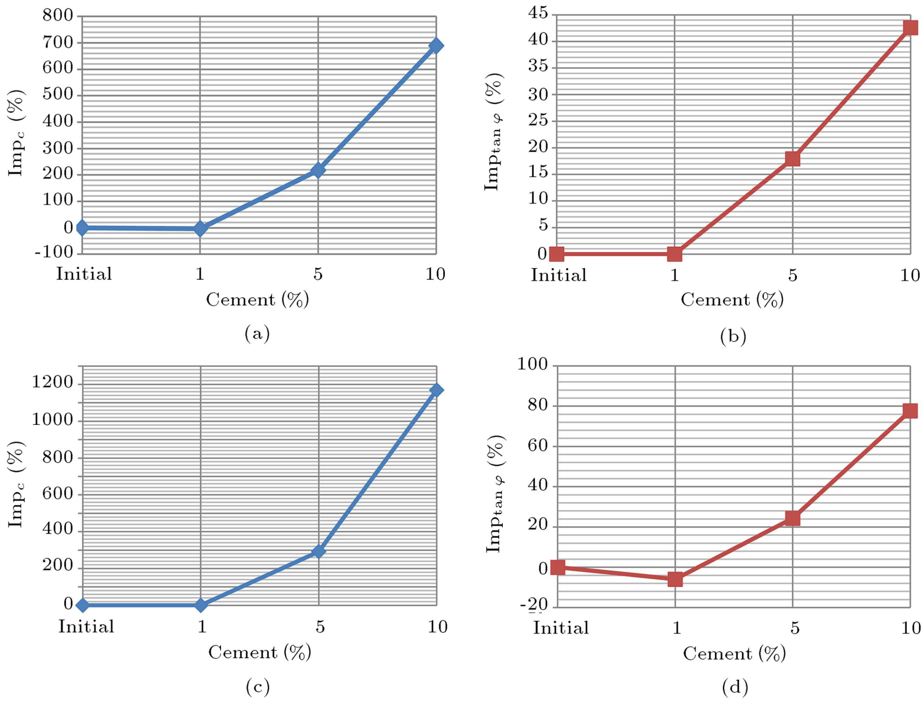


Figure 6. Cement induced shear strength parameter alterations: (a) Cohesion improvement percentage in soil media; (b) improvement of $\tan\varphi$ in soil media; (c) cohesion improvement in soil-geotextile interface; and (d) improvement of $\tan\varphi$ in soil-geotextile interface.

Considering traditional direct shear test results on untreated initial state and different levels of cement treatment, Figure 6(a) and (b) indicate soil strength improvement percentages for c and $\tan\varphi$, respectively. It can be seen that up to 1% of cement treatment does not improve the properties, but 5% and 10% of cement addition increases the shear strength parameters substantially. From Figure 6(a) and (b), it can be concluded that contribution of cement content to cohesion is more significant compared to the additional strength obtained from increase in friction angle.

The results obtained from the modified direct shear tests presenting strength improvement percentage are indicated in Figure 6(c) and (d) for c_{int} and $\tan\varphi_{\text{int}}$ in soil-geotextile interface, respectively. These results indicate that 1% cement addition does not alter the shear strength appreciably. However, as the cement content increase to 5%, there is considerable increase in both friction angle and cohesion. Moreover, it can be seen that samples with 10% cement content experience highest rate of strength improvement. Similar to the results of traditional direct shear test, cohesion is the shear strength parameter most influenced by the cement content.

Figure 7 shows contact efficiency ratios defined by Eqs. (3) and (4) for cohesion (R_c) and friction angle ($R_{\tan\varphi}$), respectively. The results show that cement addition generally increases efficiency ratios, but has much more significant improvement effects for cohesion. Hence, it can be concluded that the bonds

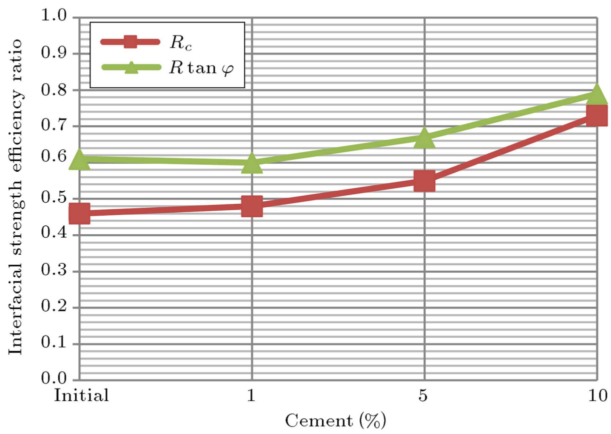
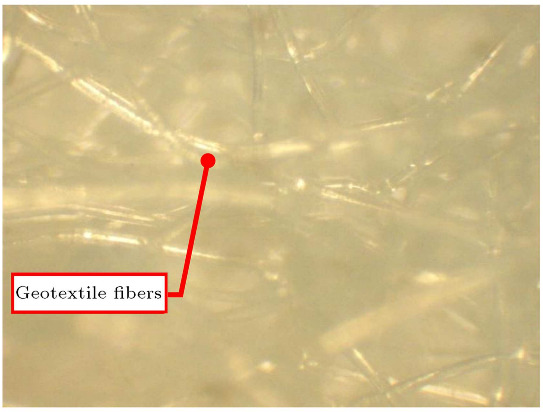


Figure 7. Efficiency ratio improvements for cohesion and friction angle.

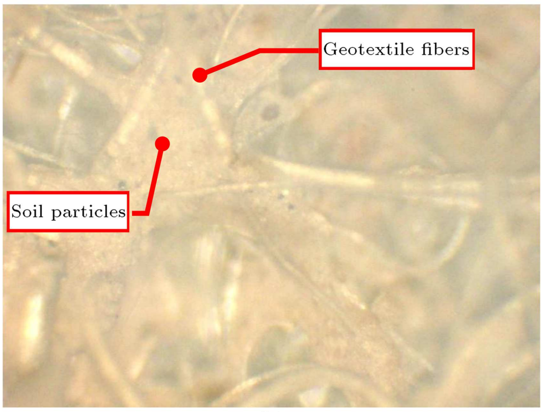
provided by cement treatment provide strong adhesion between the soil media and geotextile layer, but do not equally change the grains geometry or interfacial roughness.

3.7. Microscopic observation

Light microscope images of the geotextile sample, used for the current study before and after direct shear tests, are presented in Figure 8(a) and (b), respectively. Figure 8(a) indicates non-woven polymer fibers randomly intertwined in all directions. It can also be seen that the surface of these fibers are glossy and vivid. After performing the soil-geotextile direct shear tests, fine grained soil particles treated with cement,



(a)



(b)

Figure 8. Light microscope (400× zoomed) image of non-woven geotextile fibers (a) before, and (b) after test program.

cover and adjoin these thin polymer fibers as indicated in Figure 8(b) for 10% cement content. In fact, these connections are deduced to be of soil-cement-fiber bindings and capillary based soil-fiber unsaturated bonds. Indeed, the soil particles can only be penetrated into the available void porosity between geotextile fibers during the construction process. Hence, in the current study, the connectivity potential and therefore the interfacial soil-geotextile shear strength is highly affected by the soil particles size distribution and in particular its fines content.

Considering the structural arrangement of soil particles in contact with geotextile fibers, it can be concluded that a rather porous surface layer for a non-woven geotextile provides better conditions for interfacial bindings and adhesions. This high porosity seems to be unnecessary in inner structure of these geotextile layers.

4. Numerical modeling

4.1. Scope of usage

Sensitivity of the real soil media to the strength parameter modifications obtained from laboratory tests

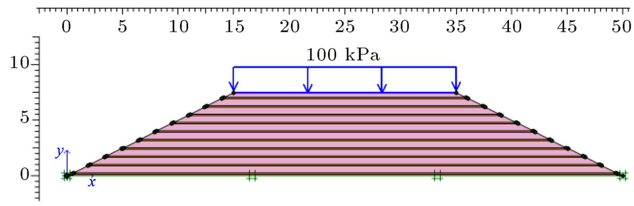


Figure 9. Geometry of the MSE embankment utilized for current modeling.

for soil, geotextile and their interface were thoroughly evaluated by numerical modeling of an illustrative MSE embankment. The geometry of this arbitrary model is presented in Figure 9 demonstrating a typical 7.5 m high embankment reinforced with 10 layers of geotextile, and subjected to a distributed surcharge of 100 kPa. The geometry of the media is an isosceles trapezoid which can be virtually divided into a rectangular area under the surcharge and two side triangular 1 : 2 slopes.

Many numerical codes are commercially available for geotechnical load-deformation study, commonly utilizing finite element or finite difference methods. In this regard, Plaxis 2D was employed for the current study.

4.2. Modeling specifications

The geometry modeled for this analysis is similar to the predefined section shown in Figure 9. The mesh consists of 15 nodded triangular elements with plane strain formulation regarding the real field stress-strain conditions for such cases. Compatible linear interface elements were used to simulate the soil-geotextile interaction. Reduced shear strength parameters were defined for these elements which model the strength characteristics of the soil-geotextile interface. Regarding the thinness of the interfacial layer, meshing of this model was refined near the interfacial elements (see Figure 10) to overcome the potential aspect ratio errors induced by disproportionate dimensions.

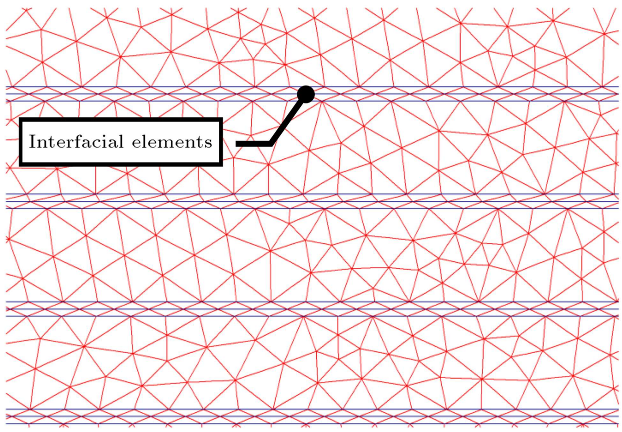


Figure 10. Refined mesh near the soil-geotextile interface.

In this study, the safety margin of the modeled condition was determined by the Shear Strength Reduction (SSR) method. In this approach adopting the perfect elastoplasticity framework, the shear strength parameters namely c and $\tan\varphi$ are sequentially reduced from their initial value to reach the limit failure state.

Hence, the safety factor can be defined as:

$$\mathrm{SF}=\frac{\text{available strength}}{\text{strength at failure}}=\frac{\tan\varphi_{\text{initial}}}{\tan\varphi_{\text{reduced}}}=\frac{c_{\text{initial}}}{c_{\text{reduced}}}, \quad (7)$$

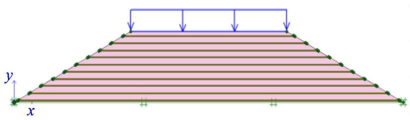
where SF is the safety factor, $\tan\varphi_{\text{initial}}$ and c_{initial} are the initial shear strength parameters, and $\tan\varphi_{\text{reduced}}$ and c_{reduced} are the reduced shear strength parameters at the limit failure state.

4.3. Investigated cases

The predefined embankment model was analyzed under 10 different conditions to evaluate its sensitivity to the material and interfacial properties. These conditions are presented in Table 6 and defined in Figure 11. As the material specifications obtained for the untreated and 1% cement treated soils are almost the same (see Table 5), 1% cement treated material was applied only in case 2 (see Table 6).

4.4. Numerical results and discussion

For the first case (see Table 6) which is an embankment made of homogeneous untreated soil layers, the state of the vertical stress distribution before and after loading has been indicated in Figure 12(a) and (b), respectively. According to the results, it can be seen that the loading process increases the stress level in a rectangular area below the surcharge from the bottom to the top of the embankment, but does not significantly change the stress values in the triangular side slope zones. In other words, the rectangular area under the surcharge is experiencing vertical stress increase without lateral stress amplification which provides higher potentials of plastic point generation in the rectangular zone. This condition is much more likely in the upper and middle third of the embankment which has almost identical side vertical stress levels before and after loading.



Upper third	4.75 m-7.5 m
Middle third	2.5 m-4.75
Lower third	0-2.5 m

Figure 11. Definition of material zoning.

Table 6. Material considered for different zones in the embankment.

	Material zoning		
	Upper third	**Middle third**	**Lower third**
Case 1	Untreated soil[1]	Untreated soil	Untreated soil
Case 2	+1% cement[2]	+1% cement	+1% cement
Case 3	Untreated soil	Untreated soil	+5% cement[3]
Case 4	Untreated soil	+5% cement	+5% cement
Case 5	+5% cement	+5% cement	+5% cement
Case 6	Untreated soil	+5% cement	+10% cement[4]
Case 7	+10% cement	+5% cement	+5% cement
Case 8	+5% cement	+5% cement	+10% cement
Case 9	+5% cement	+10% cement	+10% cement
Case 10	+10% cement	+10% cement	+10% cement

[1]Untreated soil: Untreated natural soil;

[2]+1% cement: Soil treated with 1% cement content;

[3]+5% cement: Soil treated with 5% cement content;

[4]+10% cement: Soil treated with 10% cement content.

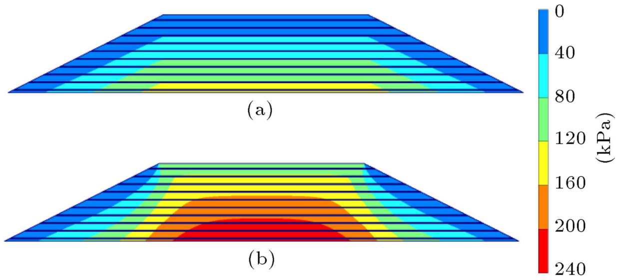


Figure 12. Vertical stress state in case 1: (a) Before, and (b) after loading.

The model has been analyzed for the cases defined in Table 6 to assess the strength performance modifications induced by different levels of cement content. Figure 13 indicates the extent of plastic point distribution for the cases studied.

It can be seen from Figure 13 that the plastic points are mainly focused in the interfacial layers and not in the soil media. Also, it is obvious that the extent of plastic point distribution follows a descending trend from case 1 to 4. Comparing the specifications presented in Table 6 for cases 1 to 4, it can be seen that their main difference is only in the material properties of the middle and lower third zones. Hence, it can be concluded that the material designation in the middle and lower third zones not only affects the plastic point distribution in their own zones but also influences the upper third zone plasticity conditions. This effect can be interpreted as the result of stress continuity in the whole media where plasticity of the lower zones imposes excessive stress redistribution in the upper zone.

Moreover, cases 4 and 6 show similar plastic point distribution extent including two distinct side zones which have become plastic due to low lateral confinement and high stress level induced by the surcharge loading. Hence, it can be concluded that loading near the slope margins will highly endanger total safety of the soil structures. Besides, it can be seen that soil improvement, if not controlled, does not necessarily enhance the performance of a soil structure and obviously, unreasonable levels of soil improvement would

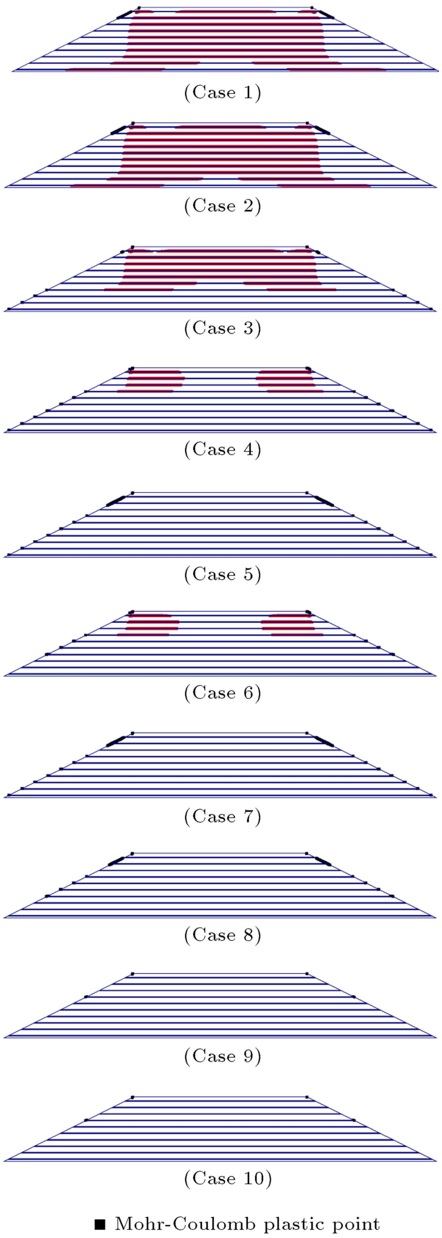


Figure 13. Plastic point distribution for cases 1 to 9.

unintentionally induce undue costs to any project. In addition, having no plastic point in their interfacial layers, cases 5, 7, 8 and 9 seem to be all in a safe state. To conclude, plastic point distribution extent is not necessarily depth dependent and does not strictly follow the layers pattern. For that reason it seems to be more efficient to perform soil improvements mostly in the selective zones with high vertical stress and low side confinements.

Furthermore, the safety factors obtained from the SSR method are shown in Figure 14 to provide a quantitative comparison between the cases as well.

The results indicate that cases 1, 2, 3, 4 and 6 exhibit almost the same safety factors. Considering the results obtained for cases 1, 2, 3, 4 and 6, it can be

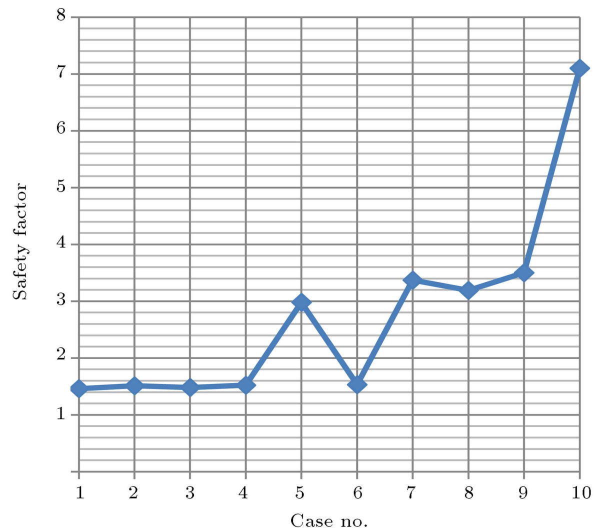


Figure 14. Safety factors obtained for predefined cases.

concluded that regarding the relatively high value of surcharge (100 kPa) and its induced stresses, material specification assigned to the upper third zone of the MSE embankment has a significant effect on the safety factor of the media. Thus, as discussed before the almost similar material properties in the upper third zone of the embankment govern the safety factor in these cases. Consequently, the soil treatments in the upper third zone are the main reasons for safety factor increase in cases 5, 7, 8, 9 and 10. Moreover, for cases 5, 7, 8 and 9, the obtained safety factor is around 3 with case 5 as the most economical case among the alternatives. In addition, while the only enhancement of case 10 with respect to 9 is the material properties improvement in the upper third zone, its safety factor is double fold.

To sum up, it was noted that the stress state of the media has the most important impact on the selection of the soil improvement zones. For the presented special case of a MSE embankment and for the relatively large surcharge considered, the upper and middle zones are the key zones. Therefore, to obtain the optimum safety factor, the media needs a relatively identical improvement level in all layers. However, if the surcharge is relatively low, improvement should start from the upper layer and increase toward the lower zone. In order to assess the severity of loading effects and subsequent proper judgment, the initial case for untreated soil media can be numerically modeled to estimate the distribution pattern and extent of the plastic points, as followed in the current study.

5. Conclusion

In this study, samples of clayey soil from Shiraz city located in the Fars province of Iran were used to assess cement treatment effects on the characteristics of soil-geotextile interface. Results of direct shear tests, microscopic observations and numerical modeling have led to the following conclusions:

- Cement treatment increases soil-geotextile interfacial shear strength with a behavior similar to over consolidated soils at high cement contents.
- For the tested clayey sample, up to 1% of cement treatment did not improve the properties but 5% and 10% of cement addition considerably increases both friction angle and cohesion.
- The bonds provided by cement treatment provide strong adhesion between the soil media and geotextile layer but do not equally change the grains geometry or interfacial roughness.
- The interfacial soil-geotextile shear strength is highly dependent upon the soil particles size distribution and in particular its fines content.
- Non-compacted rather porous surface layer for a non-woven geotextile provides better conditions for interfacial bindings and adhesions.
- Analysis of the soil media stress state is prerequisite to any appropriate selection of the soil improvement zone.

As is the case with any research, this study had its own limitations. Most of all, considering the fine grained soil used in this study, soil and cement mixture specifications is highly dependent upon the mixing method and instruments. Furthermore, although the test was performed in the standard temperature and humidity, for practical usage the in-situ conditions can be far away from these tests which might alter current conclusions.

In the course of this study, various areas where identified for further researches including the following:

- The test procedure can be repeated using different soil samples including silts and fine grained sands to assess the dependency of the results on the mineralogy and grain size distribution.
- Samples of woven geotextile can be used to compare the interfacial strength parameters between woven and non-woven geotextiles.
- Considering the drainage application of non-woven geotextiles, permeability tests can be performed on samples before and after soil treatments.
- Effect of the cement curing period on the soil-cement and interfacial shear strength parameters can be investigated for different soil types.
- Different samples with 1% cement content increment (i.e. 1%, 2%, 3%, 4%, 5% cement contents) can be investigated to clarify the improvement trend observed between 1% and 5% cement contents.

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References

1. Lasisi, F. and Ogunjide, A.M. "Effect of grain size on the strength characteristics of cement-stabilized lateritic soils", *Building and Environment*, **19**(1), pp. 49-54 (1984).
2. Walker, P.J. "Strength, durability and shrinkage characteristics of cement stabilised soil blocks", *Cement and Concrete Composites*, **17**(4), pp. 301-310 (1995).
3. Kolias, S., Kasselouri-Rigopoulou, V. and Karahalios, A. "Stabilisation of clayey soils with high calcium fly ash and cement", *Cement and Concrete Composites*, **27**(2), pp. 301-313 (2005).
4. Consoli, N.C., Foppa, D., Festugato, L. and Heineck, K.S. "Key parameters for strength control of artificially cemented soils", *Journal of Geotechnical and Geoenvironmental Engineering*, **133**(2), pp. 197-205 (2007).
5. Sariosseiri, F. and Muhunthan, B. "Effect of cement treatment on geotechnical properties of some Washington State soils", *Engineering Geology*, **104**(1), pp. 119-125 (2009).
6. Taheri, A., Sasaki, Y., Tatsuoka, F. and Watanabe, K. "Strength and deformation characteristics of cement-mixed gravelly soil in multiple-step triaxial compression", *Soils and Foundations*, **52**(1), pp. 126-145 (2012).
7. Profillidis, V. and Poniridis, P. "Visco-elastic behaviour of the soil-geotextile interface: A theoretical approach", *Geotextiles and Geomembranes*, **4**(2), pp. 145-153 (1986).
8. Fabian, K.J. and Fourie, A.B. "Clay-geotextile interaction in large retaining wall models", *Geotextiles and Geomembranes*, **7**(3), pp. 179-201 (1988).
9. Mlynarek, J., Bogumil Lewandowski, J., Rollin, A. and Bolduc, G. "Soil-geotextile system interaction", *Geotextiles and Geomembranes*, **10**(2), pp. 161-176 (1991).
10. Tan, S.A., Chew, S.H. and Wong, W.K. "Sand-geotextile interface shear strength by torsional ring shear tests", *Geotextiles and Geomembranes*, **16**(3), pp. 161-174 (1998).
11. Lee, K.M. and Manjunath, V.R. "Soil-geotextile interface friction by direct shear tests", *Canadian Geotechnical Journal*, **37**(1), pp. 238-252 (2000).
12. Racana, N., Grediac, M. and Gourves, R. "Pull-out response of corrugated geotextile strips", *Geotextiles and Geomembranes*, **21**(5), pp. 265-288 (2003).

13. Latha, G.M. and Murthy, V.S. "Effects of reinforcement form on the behavior of geosynthetic reinforced sand", *Geotextiles and Geomembranes*, **25**(1), pp. 23-32 (2007).

14. Vashi, J.M., Desai, A.K. and Solanki, C.H. "Evaluation of PET and PP geotextile reinforced embankment on soft soil", *Procedia Engineering*, **51**, pp. 19-24 (2013).

15. Thamm, B.R., Krieger, B. and Krieger, J. "Full scale test on a geotextile reinforced retaining structure: Proc 4th international conference on geotextiles, geomembranes and related products, The Hague, 28 May-1 June 1990 V1 P3-8. Publ Rotterdam: AA Balkema, 1990", In *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, **28**(6), pp. A384. Pergamon (1991).

16. Karpurapu, R. and Bathurst, R.J. "Behaviour of geosynthetic reinforced soil retaining walls using the finite element method", *Computers and Geotechnics*, **17**(3), pp. 279-299 (1995).

17. Ho, S.K. and Rowe, R.K. "Effect of wall geometry on the behaviour of reinforced soil walls", *Geotextiles and Geomembranes*, **14**(10), pp. 521-541(1996).

18. Desai, C.S. and El-Hoseiny, K.E. "Prediction of field behavior of geosynthetic-reinforced soil wall", *J. of Geotech. and Geoenv. Eng., ASCE*, **131**(6), pp. 209-239 (2005).

19. El Sawwaf, M.A. "Behavior of strip footing on geogrid-reinforced sand over a soft clay slope", *Geotextiles and Geomembranes*, **25**(1), pp. 50-60 (2007).

20. Wasti, Y. and Bahadıir Özdüzgün, Z. "Geomembrane-geotextile interface shear properties as determined by inclined board and direct shear box tests", *Geotextiles and Geomembranes*, **19**(1), pp. 45-57 (2001).

21. Palmeira, E.M., Lima Jr., N.R. and Mello, L.G.R. "Interaction between soils and geosynthetic layers in large-scale ramp tests", *Geosynthetics International*, **9**(2), pp. 149-187 (2002).

22. Narejo, D.B. "A simple tilt table device to measure index friction angle of geosynthetics", *Geotextiles and Geomembranes*, **21**(1), pp. 49-57 (2003).

23. Chung, M., Seo, M.W., Kim, K.S. and Park, J.B. "Protective effect of overlying geosynthetic on geomembrane liner observed from landfill field tests and inclined board laboratory experiments", *Waste management & research*, **24**(3), pp. 250-259 (2006).

24. Wu, W., Wick, H., Ferstl, F. and Aschauer, F. "A tilt table device for testing geosynthetic interfaces in centrifuge", *Geotextiles and Geomembranes*, **26**(1), pp. 31-38 (2008).

25. Pitanga, H.N., Gourc, J.P. and Vilar, O.M. "Interface shear strength of geosynthetics: Evaluation and analysis of inclined plane tests", *Geotextiles and Geomembranes*, **27**(6), pp. 435-446 (2009).

26. Fourie, A.B. and Fabian, K.J. "Laboratory determination of clay-geotextile interaction", *Geotextiles and Geomembranes*, **6**(4), pp. 275-294 (1987).

27. Garbulewski, K. "A laboratory assessment of the interaction between peat and geosynthetics", *Geotextiles and Geomembranes*, **10**(3), pp. 269-284 (1991).

28. Bouazza, A. and Djafer-Khodja, S. "Friction characteristics of a nonwoven geotextile and peat", *Geotextiles and Geomembranes*, **13**(12), pp. 807-812 (1994).

29. Lee, K.M. and Manjunath, V.R. "Soil-geotextile interface friction by direct shear tests", *Canadian Geotechnical Journal*, **37**(1), pp. 238-252 (2000).

30. Palmeira, E.M. "Soil-geosynthetic interaction: Modelling and analysis", *Geotextiles and Geomembranes*, **27**(5), pp. 368-390 (2009).

31. Anubhav, S. and Basudhar, P.K. "Modeling of soil-woven geotextile interface behavior from direct shear test results", *Geotextiles and Geomembranes*, **28**(4), pp. 403-408 (2010).

32. Bacas, B.M., Konietzky, H., Berini, J.C. and Sagaseta, C. "A new constitutive model for textured geomembrane/geotextile interfaces", *Geotextiles and Geomembranes*, **29**(2), pp. 137-148 (2011).

33. Jewell, R.A. and Wroth. C.P. "Direct shear tests on reinforced sand", *Geotechnique*, **37**(1), pp. 53-68 (1987).

34. Palmeira, E.M. and Milligan, G.W.E. "Scale and other factors affecting the results of pull-out tests of grids buried in sand", *Geotechnique*, **39**(3), pp. 511-542 (1989).

35. O'Rourke, T.D., Druschel, S.J. and Netravali, A.N. "Shear strength characteristics of sand-polymer interfaces", *Journal of Geotechnical Engineering*, **116**(3), pp. 451-469 (1990).

36. Takasumi, D.L., Green, K.R. and Holtz, R.D. "Soil-geosynthetics interface strength characteristics: A review of state-of-the-art testing procedures", In *Geosynthetics*, **91**, pp. 87-100 (1991).

37. Tuna, S.C. and Altun, S. "Mechanical behaviour of sand-geotextile interface", *Scientia Iranica*, **19**(4), pp. 1044-1051 (2012).

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