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Estimation of shear strength parameters of lime-cement stabilized granular soils from unconfined compressive tests

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Abstract. Analytical and numerical modeling of soft or problematic soils stabilized with lime and cement require a number of soil parameters which are usually obtained from expensive and time-consuming laboratory experiments. The high shear strength of lime and cement stabilized soils make it extremely difficult to obtain high quality laboratory data in some cases. In this study, an alternative method is proposed, which uses the unconfined compressive strength and estimating functions available in literature to evaluate the shear strength parameters of the treated materials. The estimated properties were applied in finite element model to determine which estimating function is more appropriate for lime and cement treated granular soils. The results show that at the mid-range strength of the stabilized soils, most of applied functions have a good compatibility with laboratory conditions. However, application of some functions at lower or higher strengths would lead to underestimation or overestimation of the unconfined compressive strength.

Keywords: lime and cement stabilization; finite element modeling; compressive strength; failure criterion; cohesion; internal friction angel

1. Introduction

Lime and lime-cement treatment or stabilization has been conventionally used in geotechnical engineering to improve the properties of soft or problematic soils. The effects of lime and cement stabilization on the properties (such as compressive strength, elasticity modulus and Atterberg's limits) of different types of soils have been investigated by many researchers (Hossain *et al*. 2007, Chiu *et al*. 2008, Sing *et al*. 2008, Yoon and Abu-Farsakh 2009, Al-Mukhtar *et al*. 2010, Nayak and Sarvade 2011, Senyur and Erer 1990, Yong and Ouhadi 2007, Okyay and Dias 2010, Gueddouda *et al*. 2011, Sharma *et al*. 2011, Calik and Sadoglu 2014).

To simulate the complex behaviour of the treated soil, comprehensive laboratory experiments on the stabilized soils are required in many cases to provide the input parameters for the numerical analyses and computer modelling. For example, Okyay and Dias (2010) investigated the properties

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of lime and/or cement stabilized soils which were used as pile to support load transfer platform. A series of laboratory experiments were carried out to obtain the mechanical characteristics of stabilized materials required for the numerical simulations. However, for small and medium scale infrastructure projects, conducting a full scale laboratory test is almost impossible because it is time consuming, labour-intensive and very costly. Therefore, a combination of simple laboratory experiments and computer simulation has been considered in recent years to overcome the high costs associated with full-scale testing (Okyay and Dias 2010, Arroyo *et al*. 2012).

The main objective of this study is to investigate if there is any easier way to estimate the shear strength parameters of stabilized soils out of unconfined compressive testing results. This would result in significant savings in time and money on small to medium projects since the procedure of evaluating the shear strength parameters of stabilized soils for accurate numerical simulations is an expensive and time consuming. If the easier method presents reliable results then real case problems can be solved by numerical simulation precisely by application of unconfined compressive test results which can be obtained faster and more economic.

This study comprises of two main parts: (1) laboratory experiments; (2) numerical simulation of unconfined compressive tests using commercial finite element software (PLAXIS^{2D} 8.2) to verify the accuracy of parameters' estimation. Two different types of granular soils were used in laboratory experiments and they were treated with various amounts of lime and cement. Unconfined compressive tests were performed on the cured test specimens. The failure strain, unconfined compressive strength, and elasticity modulus of treated materials were obtained from laboratory tests.

In numerical simulation, the strength parameters of the treated materials were estimated by application of estimating functions proposed by Mitchell (1976), Ahnberg (2006) and Sharma *et al*. (2010). The estimated cohesion and internal friction angle of stabilized materials were then applied in actual size axis-symmetrical numerical simulation of unconfined compressive tests to check the validity of the applied functions with the results of laboratory experiments.

After comparing the results estimated by using the functions proposed by Mitchell (1976), Ahnberg (2006) and Sharma *et al*. (2010), the results showed that Sharma's non-linear model was more reliable although it must be calibrated first.

2. Experiment program

2.1 Soil tested and sample preparation

A series of laboratory investigations have been conducted to study the mechanical properties of the stabilized granular soils. Since the treated materials were supposed to be used as bases or platforms' materials, well graded gravel and sand were selected according to the particle distribution boundaries proposed by ASTM C33 (2003) for concrete aggregates. The maximum particle size of the coarser materials has been restricted to 19 mm (i.e., less than 20% of the samples smallest dimension) while the maximum particle size of the applied sand was 9 mm. Fig. 1 displays the particle-size distribution curves of the granular materials which were stabilized with various amounts of lime and cement admixtures to form bounded materials with a variety of mechanical characteristics. Three samples were produced for each different design mixes (see Table 1) and unconfined compressive tests were performed on each sample after the curing time of seven weeks.

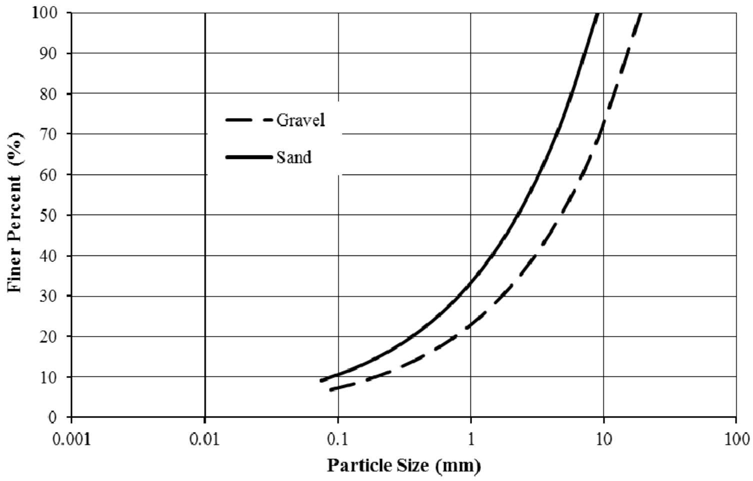


Fig. 1 The particle size distribution curves of gravel and sand used in the lab experiments

Type II Portland cement was selected as the basic stabilizing agent to produce the main bonding strength, because its resistance against acid attack has made this type of cement a favorable one in stabilizing projects. To make the stabilized material more resistant to harmful environmental effects and achieving more ductile behavior, High Calcium Hydrated Lime ($Ca(OH)_2$) was also mixed with the applied cement. This material is a practical lime type since its fine particle size makes the mixture procedure and the chemical reactions between the lime and clay less time consuming.

In order to check the validity of estimating functions, a wide range of mechanical properties of stabilized materials were required. Therefore, five different design mixes for gravel materials and three for treated sand specimens were applied. The properties of all mix designs are summarized in Table 1. The optimum water content, w_{opt}, used in sample production was obtained from the modified compaction tests according to ASTM D 698 (Budhu 2011). Various mechanical properties were obtained by altering the amount of cement and the lime/cement ratio for each soil type which is subsequently illustrated.

Table 1 The properties of different mix designs

Mix design	Cement ratio (%)	Lime ratio (%)	Moisture content (%)
S-1	4	6	10.4
S-2	5	5	9.5
S-3	6	4	9.1
G-1	4.5	6.8	9.4
G-2	4.5	5.6	8.9
G-3	4.5	4.5	8.2
G-4	5.6	6.8	9.6
G-5	5.6	4.5	8.9

Fig. 2 Samples cured at room temperature

Table 2 Summary of unconfined compressive test results

Mix design	Average compressive strength (MPa)	Average ultimate strain (%)	Average modulus of elasticity (MPa)
S-1	1.694	0.95	178.31
S-2	3.170	0.76	417.10
S-3	5.967	0.79	713.14
G-1	4.244	1.07	394.990
G-2	4.739	1.35	349.331
G-3	5.499	1.02	534.789
G-4	5.755	0.90	639.586
G-5	6.954	0.90	770.921

Three standard cylindrical samples (10 cm in diameter and 20 cm in height) were produced for unconfined compressive tests for each mix design. Samples were compacted in five layers by the maximum possible compaction energy to reach homogenous samples in elevation (Ladd 1978). The amount of applied energy for each sample's compaction was about 1124.2 N.m which was attained by numerous compaction tests (Azadegan *et al*. 2012). The compacted samples were submerged in water for seven weeks at room temperature prior to the unconfined compressive testing (see Fig. 2).

2.2 Unconfined compressive tests

The unconfined compressive tests were carried out on the cured samples according to ASTM C39-86. A curing time of seven weeks has been selected in such a way that no significant increment in samples' strength would be observed after this time (for more detailed information see Azadegan *et al*. (2012)). The unconfined compressive tests are summarized in Table 2. Ultimate compressive strength, failure strain and unconfined elasticity modulus were extracted for each sample from the unconfined compressive tests.

3. Estimating relations

Different failure criteria have been presented for cemented and brittle materials (Hegermier and Read 1985, Boswell and Chen 1987, Chiu *et al*. 2008, Jan and Van Mier 2008). In order to use the results of soil improvement in finite element (FE) simulations, the strength parameters (i.e., cohesion and friction angel) of stabilized soil must be firstly determined by application of estimating relations. Thus, three different estimating relations have been used in this study to evaluate the amounts of cohesion (C) and friction angle (ϕ) from compressive strength (parameters of shear strength) of the stabilized soils. Two of these three relations, presented by Mitchell (1976) and Ahnberg (2006), are based on the linear Mohr-Coulomb failure criterion and the other one, proposed by Sharma *et al*. (2010), is obtained from non-linear failure criterion. The estimating relations are described subsequently.

Mitchell (1976) adopted the following equation to estimate the cohesion of cement stabilized soil.

$$C = 7.0 + 0.225\sigma_c \tag{1}$$

where c and σ_c are cohesion and compressive strength (in psi) respectively.

The Mohr-Coulomb linear failure criterion (Eq. (2)) is assumed for this type of materials which in combination with Eq. (1) gives a constant value of internal friction angle, ϕ.

$$\sigma_1 = \sigma_{3.}\ \tan^2\left(45 + \frac{\phi}{2}\right) + 2C\tan\left(45 + \frac{\phi}{2}\right) \tag{2}$$

where, σ_1 and σ_3 are the main axial stresses in triaxial shear test, C is the cohesion and ϕ is the internal friction angle.

Ahnberg's relation (2006) presented in Eq. (3) shows the relationship between the cohesion and the compressive strength of cement stabilized soft soils such as clays and pits. Ahnberg has also adopted the linear Mohr-Coulomb failure criterion (Eq. (3)) to attain this relation.

$$C = 0.2482q_u \tag{3}$$

The Ahnberg's relation also presents constant value of friction angle and satisfies the strength changes by variations of the amount of cohesion.

Based on non-linear failure criterion, Sharma *et al*. (2010) obtained the following Eqs. (4)-(6) for weakly cemented sand by means of triaxial shear test

$$\tau = P_a\left(0.115\frac{q_u}{P_a} + 1.242\right)\left(\frac{\sigma + 0.035q_u}{P_a}\right)^n \tag{4}$$

where τ is the shear strength, q_u, P_a and σ are differential axial pressure, atmospheric pressure and confining pressure (in kPa) respectively. n varies from 0.5 to 1.0 and must be calibrated for different soil types.

When the confining pressure σ is reduced to zero (unconfined compressive test), the shear strength τ is equal to cohesion of materials, C_r

Table 3 Estimated cohesions and friction angels

Mix design	Sharma *et al*. 2010 (n = 0.5)		Sharma *et al*. 2010 (n = 0.6)		Ahnberg 2006	
	C_r (kPa)	ϕ_t	C_r (kPa)	ϕ_t	C (kPa)	ϕ_t
S-1	24.045	36.94	22.78	30.72	41.25	37.25
S-2	28.67	38.05	27.59	32.17	48.23	37.25
S-3	114.13	46.78	122.62	45.26	145.29	37.25
G-1	72.79	44.04	75.58	40.78	103.33	37.25
G-2	84.04	44.93	88.23	42.20	115.39	37.25
G-3	102.31	46.13	109.02	44.16	133.89	37.25
G-4	108.72	46.50	116.38	44.77	140.13	37.25
G-5	140.42	47.98	153.18	47.33	169.32	37.25

$$C_r = P_a\left(0.115\frac{q_u}{P_a}+1.242\right)\left(\frac{0.035q_u}{P_a}\right)^n \tag{5}$$

And the tangent friction angel (ϕ_t) at any arbitrary confining pressure of σ can be calculated by following relation

$$\tan(\phi_t) = \frac{\left(0.115\frac{q_u}{P_a}+1.242\right)^n}{\left(\frac{\sigma+0.035q_u}{P_a}\right)^{1-n}} \tag{6}$$

Table 3 shows the strength parameters for different mix designs which are calculated using Eqs. (5)-(6). These parameters were applied in finite element models of unconfined compressive tests to check which estimating relation produces more reliable results. Higher powers in Sharma *et al*.'s (2010) estimating relation led to great underestimation for the analysed soil type in preliminary studies; therefore, in this paper the results of two powers of 0.5 and 0.6 are cited for Sharma *et al*.'s (2010) relations and are compared with two other estimating models.

4. Finite element simulation

Finite element simulations of unconfined compressive tests were carried out using PLAXIS 8.2 for each mix design. The soil properties, required for numerical model, were obtained from laboratory experiments and estimating relations as well. The elasticity modulus and dry density were obtained from laboratory tests while strength parameters such as the cohesion and the internal friction angel were calculated using estimating relations (as shown in Table 3).

Plastic analysis on a soil cylinder, with dimensions of 20 cm and 10 cm in height and diameter respectively, was performed with the special characteristics of each mix design. A total of 96 finite element models for the final step of the study were run which contained approximately 10 analysis phases for each model (see Figs. 3 to 7). As shown in Fig. 4(b), a very fine finite element mesh

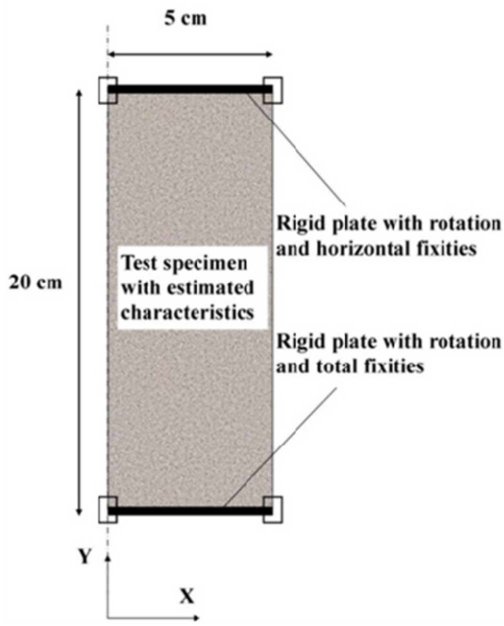
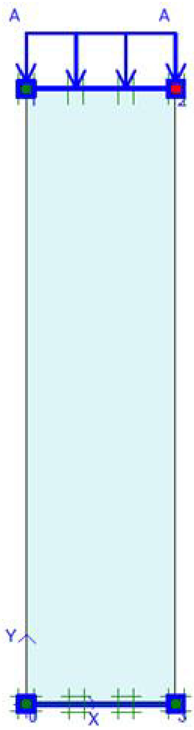
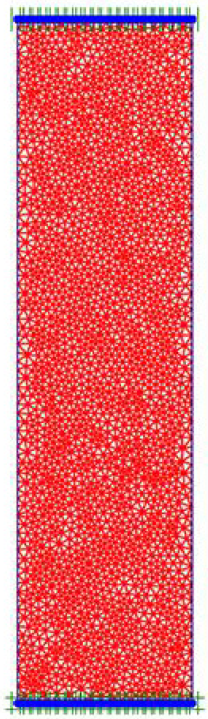


Fig. 3 Schematic representation of specimen



(a) Simulation



(b) Mesh generation (a very fine mesh was used)

Fig. 4 Finite element model employed for the analysis

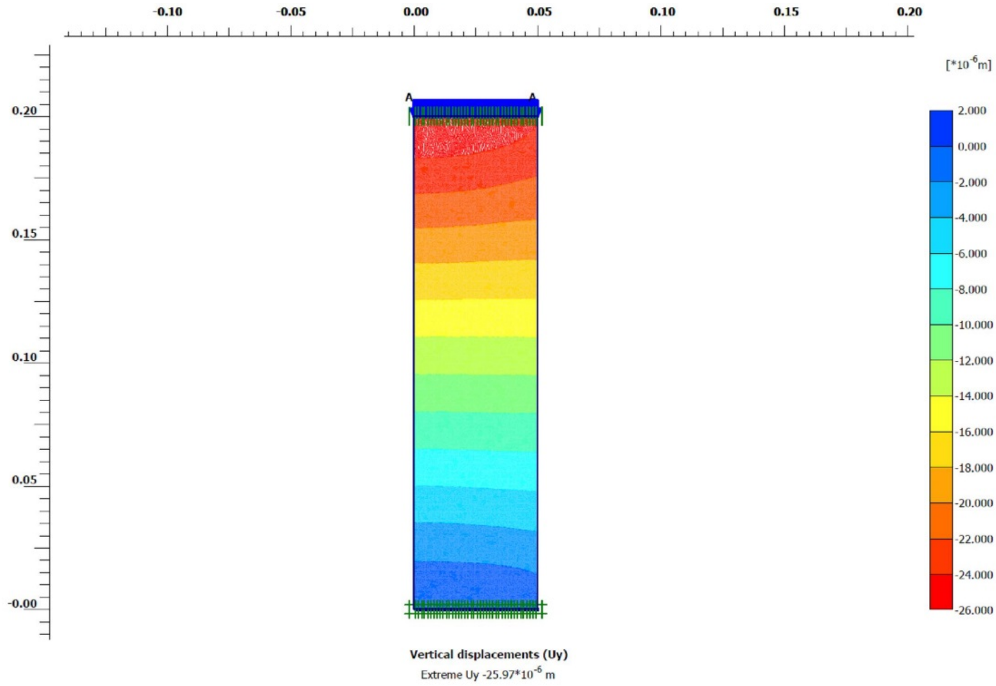


Fig. 5 Vertical displacement of mix design G-5 under 100 kPa load (mm)

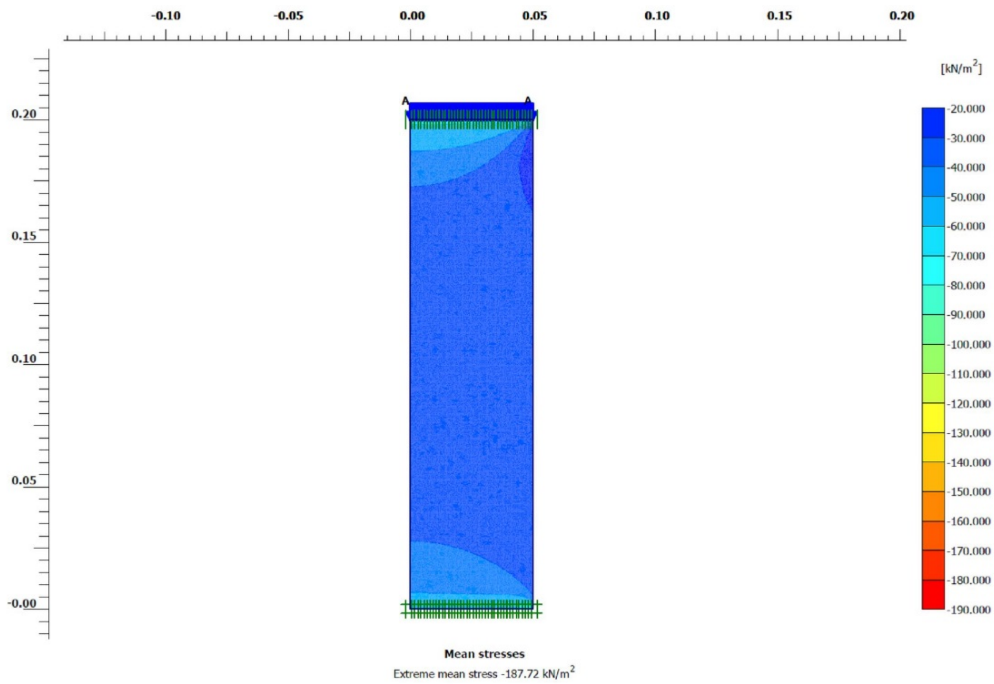


Fig. 6 Mean stresses of mix design G-5 under 100 kPa load (kPa)

was used to improve the accuracy of simulation results.

Pressure loading was applied on soil sample in FE simulations. In the first step of analysis a pressure load much lower than the failure load was applied to the sample. In second step a much higher load was selected to ensure that it is larger than the bearing capacity of soil sample and the software reported "soil body collapse error" as the calculation result. After that in order to achieve the appropriate failure result in F.E. analysis the bisection method of approximation was used, in which by dividing the band between loads of collapsed phase and analysed phase the failure point's band becomes smaller and smaller till the answers becomes clear with desired accuracy. For example in S-1 mix design for Sharma *et al.*'s (2010) relation with the power of 0.5 the exact answer of failure point might be 1694 kPa; therefore, the software cannot analyse the model because of the soil collapse error at the pressure amount of 2000 kPa and the model is accurately analysed at the pressure amount of 1000 kPa because this pressure is below the bearing capacity of

Table 4 Summary of indirect tension, unconfined compression, and single stage drained triaxial compression test results for varying densities and cement contents (Sharma *et al.* 2010)

Initial bulk density (g/cc)	% cement	σ'_{3f} [1,2] (kPa)	σ'_{1f} (kPa)	Initial bulk density (g/cc)	% cement	σ'_{3f} [1,2] (kPa)	σ'_{1f} (kPa)
1.8	0	0	42.4	2.1	1	100	1221
		20	64			300	2338
		50	155		2.5	-41.4	0
		100	308			0	1016
	1	-3.3	0			20	1129
		0	128			100	1964
		20	250			300	3081
		100	445	2.25	0	0	164
		300	1163			72	740
	2.5	-14.8	0			122	1120
		0	275			322	2251
		20	395		1	-23.9	0
		100	867			0	859
		300	1494			74	1559
2.1	0	0	141			124	1820
		20	200			322	2760
		100	675		2.5	-59.5	0
		300	1762			0	1807
	1	-15.3	0			72	3013
		0	411			123	3406
		20	533			322	4687

[1] Indirect tension test results are shown with negative values for σ'_{3f} and zero for σ'_{1f}

[2] Unconfined compression tests were performed under drained conditions with σ'_{3f} equal to zero

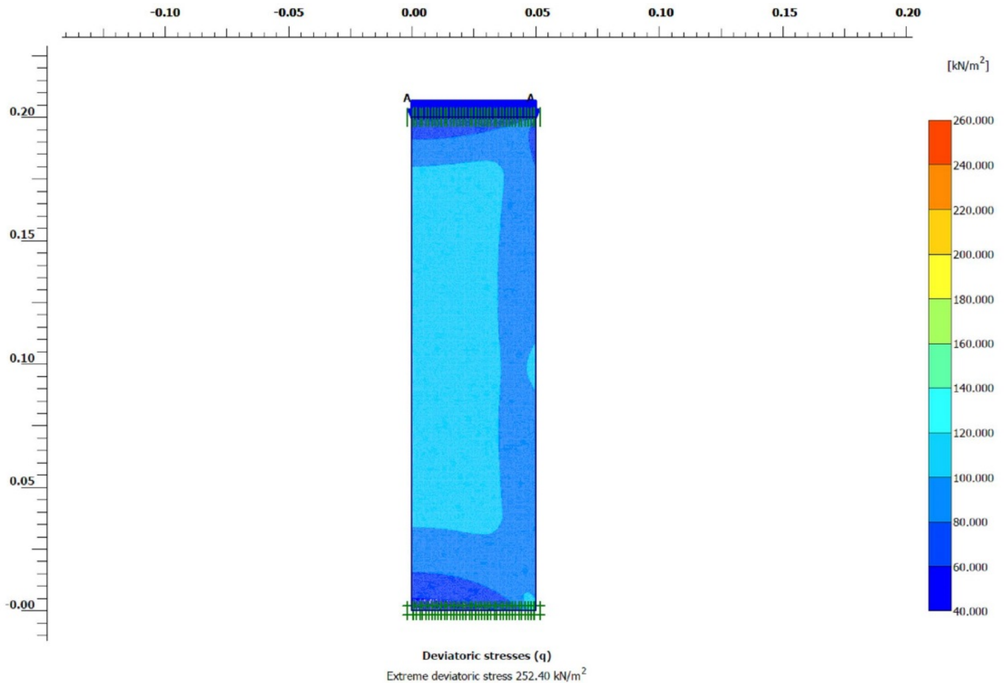


Fig. 7 Shear stresses of mix design G-5 under 100 kPa load (kPa)

sample. By dividing the band between 2000 and 1000 into two equal sections the next trial point becomes 1500. If the sample be stable at the load of 1500 kPa, the next trail point would be 1750 kPa; otherwise, the next point will be 1250 kPa. The procedure was repeated until the accuracy of the answer was equal to or less than 1% of the strength obtained from computer simulation.

Table 5 Estimated Parameters for Sharma *et al.*'s tests results (Sharma *et al.* (2010))

Mix design	Compres-sive streng-th (kPa)	n = 0.5		n = 0.6		n = 0.7		n = 0.8		n = 0.9		n = 1.0	
		C_r (kPa)	ϕ_t	C_r (kPa)	ϕ_t	C_r (kPa)	ϕ_t	C_r (kPa)	ϕ_t	C_r (kPa)	ϕ_t	C_r (kPa)	ϕ_t
1	42.4	31.38	34.19	33.98	35.29	36.58	36.39	39.18	37.49	41.78	38.59	31.38	34.19
2	128	42.49	34.97	45.09	36.07	47.69	37.17	50.29	38.27	52.89	39.37	42.49	34.97
3	275	60.62	36.19	63.22	37.29	65.82	38.39	68.42	39.49	71.02	40.59	60.62	36.19
4	141	44.12	35.08	46.72	36.18	49.32	37.28	51.92	38.38	54.52	39.48	44.12	35.08
5	411	77.38	37.21	79.98	38.31	82.58	39.41	85.18	40.51	87.78	41.61	77.38	37.21
6	1016	157.83	40.82	160.43	41.92	163.03	43.02	165.63	44.12	168.23	45.22	157.83	40.82
7	164	46.98	35.28	49.58	36.38	52.18	37.48	54.78	38.58	57.38	39.68	46.98	35.28
8	859	135.85	40.00	138.45	41.10	141.05	42.20	143.65	43.30	146.25	44.40	135.85	40.00
9	1807	280.64	44.08	283.24	44.11	285.84	44.23	288.44	44.61	291.04	44.75	280.64	44.08

Table 6 Summary of FE simulation of Sharma *et al.*'s (2010) laboratory experiment

Mix design	Compressive strength (kPa)	Approximated strength by FE simulation (kPa)						Exact solution for power of
		$n = 0.5$	$n = 0.6$	$n = 0.7$	$n = 0.8$	$n = 0.9$	$n = 1.0$	
1	42.4	358.32	154.76	65.48	31.46	17.58	10.17	0.765
3	275	432.43	308.423	192.77	145.34	109.75	85.36	0.635
6	1016	798.46	708.408	633.012	578.64	533.53	496.18	No exact
9	1807	1299.99	1328.35	1375.43	1431.32	1470.42	1516.66	No exact

First Sharma *et al.*'s (2010) laboratory experiments were used to validate the numerical model. The amount of pressures applied in final simulation phases were compared with the amount of total failure pressure imposed by the unconfined compressive test apparatus in the laboratory experiments. The strength parameters which were estimated using the parameters presented by Sharma *et al.* (2010) (Table 4) for different powers of "n" are given in Table 5. The ultimate compressive strength was accurately approximated by numerical simulations for two samples of Sharma *et al.*'s (2010) work which in Table 6 the power of exact solution is given in the last column. Although, the compressive strength of other simulated samples were underestimated but the result have an acceptable accuracy (see Table 6).

As it can be observed in Table 6, there exists a proper compatibility among the laboratory experiments' result in lower range of strengths and even the exact amount of strength could be found for a special power of "n" by finite element simulation; Although, in higher strengths the numerical calculation obviously underestimates the unconfined compressive strength of test specimens but the discrepancy is reduced as the compressive strength of specimens increases. This can be observed in Mix Design 9 from Sharma *et al.*'s (2010) work and in simulation of the laboratory tests conducted in this research.

The finite element analysis results, approximated compressive strength and failure strain, are given in Table 7. The amount of error in approximating the ultimate compressive strength for each simulation was calculated by the following relation

$$E(\sigma_u) = \frac{\sigma_{Mod} - \sigma_{Lab}}{\sigma_{Lab}} \times 100 \tag{7}$$

where $E(\sigma_u)$ is the amount of error in compressive strength, σ_{Lab} and σ_{Mod} are the compressive strength obtained from laboratory experiment and FE modeling respectively.

5. Discussion

This study has been carried out to investigate if it is possible to evaluate the required characteristics of stabilized soils for application in finite element modeling from unconfined compressive tests' results or not. Therefore, two different soil types were treated with various amounts of lime and cement in order to achieve a vast domain of mechanical characteristics and test specimens were prepared and compressive tests were performed on cured samples. Then by utilizing three different estimating relations the unknown mechanical characteristics such as

Table 7 Summarized results of finite element simulations of experimental program

Mix design	Sharma *et al*. (2010) (n = 0.5)			Sharma *et al*. (2010) (n = 0.6)			Ahnberg (2006)		
	σ_u (kPa)	ε_u (mm/mm)	Error in σ_u (%) Based on Table 2	σ_u (kPa)	ε_u (mm/mm)	Error in σ_u (%) Based on Table 2	σ_u (kPa)	ε_u (mm/mm)	Error in σ_u (%) Based on Table 2
S-1	1.65	0.0092	-2.8	1.58	0.0088	-7.2	1.86	0.0105	10.0
S-2	3.08	0.0074	-3.0	2.96	0.0071	-6.5	3.30	0.0079	4.1
G-1	4.05	0.0057	-4.6	3.99	0.0056	-6.0	4.29	0.0060	1.0
G-2	4.50	0.0114	-5.0	4.46	0.0113	-5.8	4.64	0.0118	-2.0
G-3	5.22	0.0149	-5.1	5.21	0.0149	-5.3	5.17	0.0148	-6.0
G-4	5.47	0.0102	-5.0	5.47	0.0102	-4.9	5.30	0.0099	-8.0
S-3	5.53	0.0086	-7.3	5.73	0.0090	-4.0	5.31	0.0083	-11.0
G-5	6.25	0.0081	-10.1	6.88	0.0089	-1.0	5.71	0.0074	-18.0

cohesion and internal friction angel were evaluated and applied in finite element simulation of unconfined compressive tests.

According to the results of finite element modeling three different regions can be assumed on the compressive strengths domain as low-range strengths (e.g., S-1 and S-2 design mixes), mid-range strengths (e.g., G-1 to G-4 design mixes) and high-ranged strengths (S-3 and G-5). The validity of estimating functions for computer simulation is discussed in the following sections.

Lower bond strength estimating model presented by Sharma *et al*. (2010) shows better compatibility with the laboratory experiments. Although, the higher powers of the model underestimate the compressive strengths but the power of 0.5 seems to be an appropriate approximation for cohesion and friction angel. Thus the approximating function for the low-ranged strengths can be expressed as following (Eqs. (8) and (9)) because of its least error in evaluating the strength in FE simulations

$$C_r = 100\left(0.115\frac{q_u}{100} + 1.242\right)\left(\frac{0.035q_u}{100}\right)^{0.5} \tag{8}$$

$$\tan(\phi_t) = \frac{\left(0.115\frac{q_u}{100} + 1.242\right)^{0.5}}{\left(\frac{0.035q_u}{100}\right)^{0.5}} \tag{9}$$

as P_a is assumed to be approximately 100 kPa.

At mid-ranged strengths it seems that all estimating functions act appropriately but to some extents Sharma *et al*.'s (2010) relation with the power of 0.5 seems to give better results. Ahnberg's relation overestimated the strength of specimens for the strengths which are below 4.3

MPa and this can cause some problems in future designs. Thus, Eqs. (8)-(9) can be suggested also for the same case.

When the strengths exceed 5.5 MPa, the estimating relation proposed by Sharma *et al*. (2010) with the power of 0.6 also presents more accurate answers in simulations since the results of the application of other functions in simulations lose their accuracy in comparison with laboratory results. Therefore, the estimating function for stabilized soils with higher strength can be written as

$$C_r = 100\left(0.115\frac{q_u}{100} + 1.242\right)\left(\frac{0.035q_u}{100}\right)^{0.6} \tag{10}$$

$$\tan(\phi_t) = \frac{\left(0.115\frac{q_u}{100} + 1.242\right)^{0.6}}{\left(\frac{0.035q_u}{100}\right)^{0.4}} \tag{11}$$

Estimating function presented by Mitchell (1976) for high compressive strengths of cemented materials, produces low cohesion and subsequently significantly high internal friction angels (more than even 80 degrees in this study). Therefore this relation was not taken into account in this study.

6. Conclusions

Performing comprehensive laboratory experiments on stabilized soils to get all parameters required for finite element simulation is very time consuming, expensive, and almost impossible for small projects. If the required characteristics of stabilized soils can be estimated from unconfined compressive test results, it will result in significant savings in time and money.

The main objective of this study is to find out which of the estimating relations available in literature is more reliable for finite element simulation of stabilized granular soils. Thus, two different granular soils, well graded gravel and well graded sand, were treated with various amounts of lime and cement to produce compressive test specimens that resulted a vast domain of compressive strength and properties. Three different estimating relations presented by Mitchel, Ahnberg and Sharma *et al*. (2010) were utilized to approximate the strength parameters of tested stabilized soil and the result were then applied in PLAXIS 8.2 models of unconfined compressive tests of treated materials.

The results indicate that the non-linear failure criterion presented by Sharma *et al*. (2010) has a good compatibility with the experimental results of this study. Moreover, the alteration of relation power (n) produces a wide range of cohesions and friction angles that give the model a more compatibility with various conditions. On the other hand, it shows that for any new research the relations must be calibrated at first.

Linear Mohr-Coulomb failure criterion and the depended estimating functions presented by Ahnberg led to almost close results in finite element simulations; however, the application of Ahnberg's model for estimating parameters would lead to overestimating the compressive strength of treated materials at low-ranged strengths. This would make the more complicated non-linear failure criterion, presented by Sharma *et al*. (2010), more reliable in approximating mechanical properties from unconfined compressive tests.

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