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Study on the failure behavior of three different stabilized problematic soils

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Abstract. In this study laboratory testing of effectiveness of cement treatment has been made on geotechnical parameters of problematic soils encountered in southern coast line of Caspian Sea, Iran. Gorgan loess, Rasht clay, and Anzali sand were selected in this research. Addition of cement was found to improve workability and increased unconfined compressive strength, and elastic modulus of soils significantly. Triaxial test results indicated that cement treatment not only improved shear strength remarkably, but also it changed the type of failure greatly from ductile to brittle behavior. The large scale direct shear test results showed significant improvement in shear strength and shear modulus. Besides, the brittle behavior of cement treated samples was observed. Eventually, it was found that the trend of failure envelope of cement treated samples was non-linear, and some failure criteria such as modified Griffith theory, Hoek-Brown theory, and the Johnston criterion can describe the soil cement behavior satisfactorily.

1. Introduction

In recent years, due to population growth, suitable land for construction has been hard to locate. For improving and optimum use of available soils, great competition among civil engineers has been created. Distribution of problematic soils, those with high moisture and low efficiency pose a lot of difficulties to construction projects. All improvement techniques seek an increase in density and shear strength, providing stable condition, reduction of soil compressibility and control ground water flow, or increasing the rate of consolidation [1]. The soil-cement technique has been used successfully in pavement base layers, slope protection for earth dams, as a base layer to shallow foundations and to prevent sand liquefaction [2-5]. Many researchers have studied efficiencies of soil-cement [6-9]. A recent study has made an extensive laboratory testing of cement treatment effects on shear strength parameters and also estimating failure envelope of soils encountered in southern coastline of Caspian Sea.

2. Testing soils

In this study, three different soils including Gorgan loess, Rasht clay and Anzali sand from southern coastline of Caspian Sea, have been selected as shown in Figure 1, while the sampling procedure has been conducted in the shallow depths. Soils in the northern parts of Iran are mainly classified as clay, silt, loess and sand. Due to the unique geological condition, water level in these areas is high and soils are mostly saturated. Table 1 presents a summary of the geotechnical properties of these soils.

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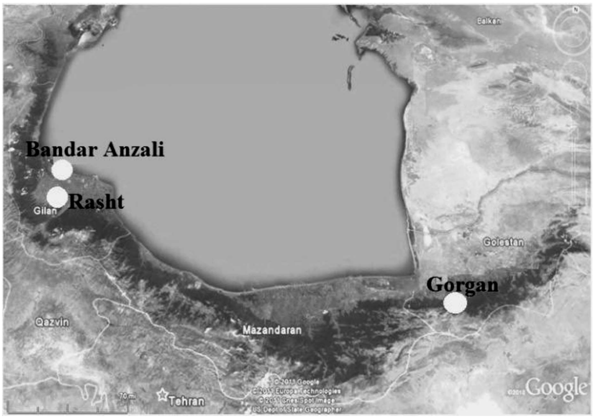

Figure 1. Three different zones in southern coast line of Caspian Sea.

Table 1. Geotechnical properties of the testing soils.

Soil properties	Rasht	Anazli	Gorgan	Standard methods
Soil group	CL	SP	CL-ML	ASTM D422-63
Passing no. 200	83	1.5	98.5	ASTM D422-63
Specific gravity	2.72	2.68	2.7	ASTM D854
LL (%)	48	—	25	ASTM D4318
PL (%)	26	NP	20	ASTM D4318
PI (%)	22	—	5	ASTM D4318
In situ density (g/cm^3)	1.89	2.08	1.63	ASTM D1556
Natural water content (%)	30	14	18	ASTM D2216
Maximum dry density (g/cm^3)	1.58	1.85	1.7	ASTM D698
Optimum water content (%)	22	7.5	14	ASTM D698

3. Laboratory testing methods

Laboratory works were carried out on the mixture of soils with different percentages of Portland cement type II by 2.5, 5 and 8% of dry weight of soil. Laboratory tests such as standard proctor, Unconfined Compressive Strength (UCS), Consolidated-Drained triaxial (CD), and large-scale direct shear tests were performed on both non-treated and cement treated samples. Soil-cement mixtures were tested to reach an optimum ratio [10-14].

To determine the optimum moisture content and maximum dry density, series of standard proctor tests were carried out on non-treated and cement treated samples of three types of soils, based on ASTM D558 [15] and ASTM D698 [16] standards.

Unconfined compressive strength test is the most common test in soil stabilization and plays a great role as an index for quantifying the soil improvement that has been carried out on the specimens based on ASTM D2166 [17]. Treated specimens were also prepared at an optimum moisture content and 95% of maximum dry density for each mixture of 2.5, 5 and 8% cement content by weight, using split mold with inner diameter and height of 5 and 10 cm, respectively. Materials were compacted in the molds in 7 equal layers to achieve desirable density, and it was carefully tried to prepare the most uniform specimens and less disturbance while bringing them out of the mold. For cohesive soils, hydraulic jacks were used to bring the samples out of the mold. However, for non-treated Anzali sand splitting the mold without disturbing the specimen was not possible. The prepared specimens divided into two series. One series of specimens was wrapped in a plastic sheet and kept for 7 days in a controlled room temperature. The second series was kept for 3 days in a humid room at a temperature of 25 degrees and then was immersed in water under soaked condition for 4 days before testing. It should be noted that the soaked condition was not applicable for the third series of non-treated specimens.

For a detailed analysis of shear strength of both natural and cement stabilized specimens, series of Consolidated-Drained triaxial tests (CD) were also conducted on all three types of samples. Confining pressure varied from 50 to 500 kPa. The diameter and height of the split mold were 3.8 and 7.6 cm, respectively. The specimens were then embedded in a triaxial chamber and backpressure was applied until saturation was reached. Specimens were consolidated until the height of water in burette did not rise. Finally, specimens were sheared at a deformation rate of 0.08 mm per minute (strain controlled condition). B values are dependent on the compressibility of soils. Cement treatment decreases the compressibility of soils significantly. B value for cement treated sand was obtained to be about 0.7, was equal to 0.8 for cement treated loess and clay soils, and 0.9 for both non-treated loess and clay.

Direct shear test was carried out according to the ASTM D3080 [18] as a standard procedure for soil test under consolidated drained condition. Large scale direct shear apparatus with a 300 × 300 × 150 mm shear box was used in this study. This test was carried out on the non-treated and cement treated (5 and

8%) sandy soil only. Materials passed sieve No 3/4″ and specimens were prepared at 95% of maximum dry density and optimum water content in a steel split mold ($300 \times 300 \times 100$) and cured for 5 days, then placed in a shear box for 48 hours in water to get full saturation. Vertical pressure was applied for 5 hours and then specimens were sheared at the rate of 0.5 mm/min. Vertical pressure varied from 50 to 200 kPa.

4. Test results and discussion

4.1. Unconfined Compressive Strength (UCS)

The effect of cement treatment on unconfined stress-strain behavior of clay soil for unsoaked and soaked conditions is shown in Figure 2. It is observed that the peak axial stress increases significantly due to cement treatment, but the corresponding strain decreases from approximately 3.5% to 1.5%. Effect of cement on unconfined compressive strength and modulus of elasticity is shown in Figure 3 for both soaked and unsoaked condition. It is observed that the soaked samples with 8% cement content exhibit greater unconfined compressive strength compared to the unsoaked samples, but the modulus of elasticity for unsoaked samples is greater compared to the soaked samples. Table 2 presents a summary of unconfined compressive strength and modulus of elasticity of the all 3 types of soils for both the unsoaked and soaked conditions.

4.2. Consolidated-Drained triaxial test (CD)

Deviator stress versus axial strain, and volumetric strain versus axial strain curves for non-treated and 8% cement treated Gorgan loess at different confining pressure are presented in Figure 4. The stress-strain and volume change of non-treated loess typically shows ductile behavior. This was also observed in the bulging type failure. It can be seen that deviatoric stress increases with an increase in confining pressure and cement addition. By addition of 8% cement, peak deviator stress occurred at about 1.5-2% strain increment and progressive softening observed up to 15%

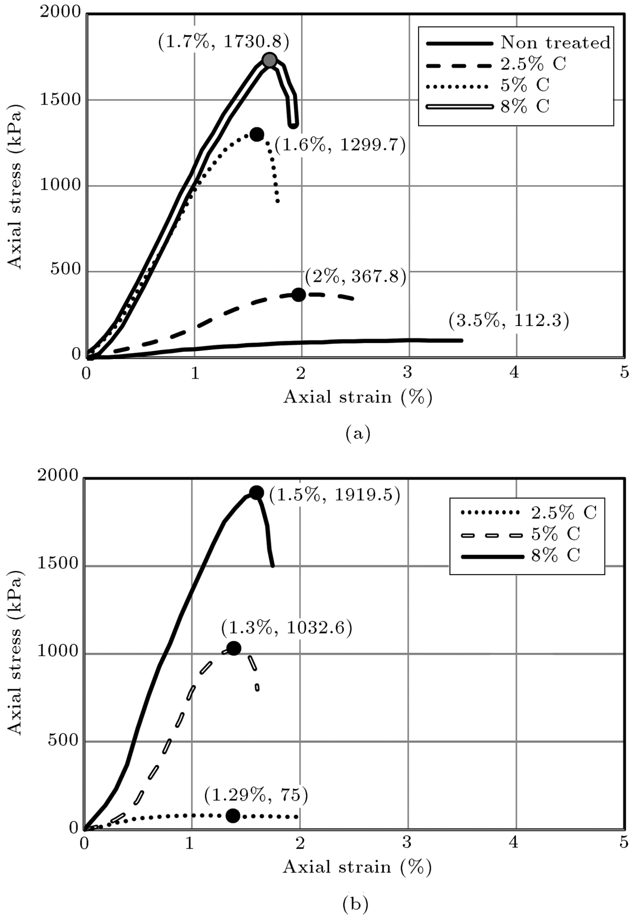

Figure 2. Effect of cement treatment on stress-strain curves of Rasht clay: a) Unsoaked; and b) soaked.

Table 2. Summary of unconfined compressive strength test and modulus of elasticity of non-treated and cement treated soils.

Soil type	Cement content (%)	UCS (kPa) Unsoaked	UCS (kPa) Soaked	E (MPa) Unsoaked	E (MPa) Soaked
Gorgan loess	0	87.6	0	5.7	0
	2.5	515.6	151.4	42.8	11.8
	5	1480.6	1166.1	93.2	68
	8	2137.4	1816.4	163.3	108.9
Rasht clay	0	112.3	0	4.3	0
	2.5	367.8	75	15	0
	5	1299.7	1032.6	76.1	40.3
	8	1730.8	1919.5	99.3	75.2
Anzali sand	0	5	0	0	0
	2.5	103.4	78.1	9	5.1
	5	909.3	715.2	63.5	50.5
	8	1603.6	1494.7	102	87.7

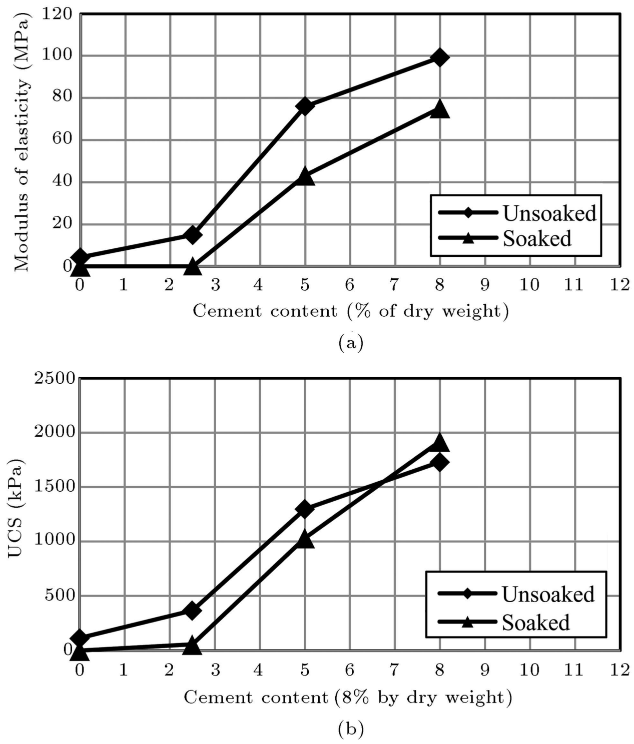

Figure 3. Effect of cement treatment on Rasht clay: a) Unconfined compressive strength; and b) modulus of elasticity.

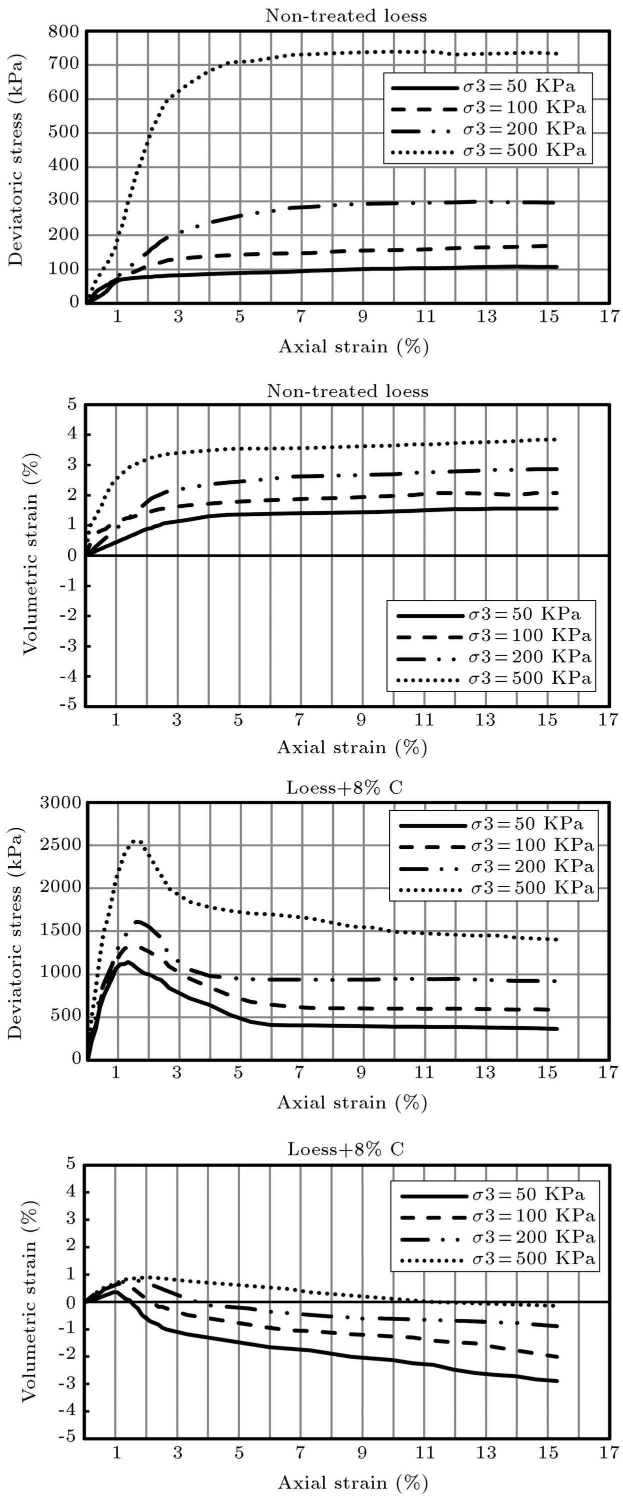

Figure 4. Results of consolidated-drained triaxial tests for non-treated and 8% cement treated of Gorgan loess.

axial strain. These cement treated specimens failed with either single or double shear band (Figure 5). For specimens with 8% cement, failure strain was about 1.2-1.5% and specimens failed with splitting type at low confining pressure, and with planar type at high confining pressures. Volumetric strain-axial strain curves showed a behavior similar to Over-Consolidated (OC) soils. Initial compression up to the failure point and then expansion were observed up to 15% axial strain. Results of triaxial tests on Anzali sand and Rasht clay followed the pattern of Gorgan loess. The strength parameters of all specimens which were obtained from Mohr circles and failure envelope are presented in Table 3. Cement treatment increased the cohesion parameter remarkably, but friction angle increased initially and then decreased (or remained constant) by increasing the cement content.

4.3. Large scale direct shear test

Shear stress versus horizontal displacement curves for non-treated and 5 and 8% cement treated sandy samples for vertical pressures equal to 200 kPa are illustrated in Figure 6. It is observed that peak shear stress increases significantly due to the cement treatment and progressive softening observed afterward, but the corresponding shear strain decreased from approximately 3.5% (displacement = 10.5 mm) to 1.7% (displacement = 5 mm). Thus, cement treated soils exhibited brittle behavior compare to non-treated soils. Table 4 presents the shear strength parameters of non-treated and cement-treated samples of Anzali sand based on Mohr-Coulomb criteria. Results showed that cement treatment led to a high increase in both cohesion and friction angle parameters. It is also obvious that shear modulus corresponding to the peak

Figure 5. Double shear band failure of 5% cement treated loess.

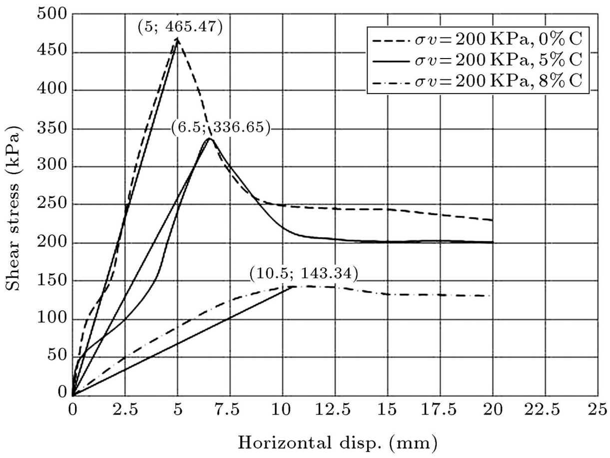

Figure 6. Results of large scale direct shear test for non-treated, 5 and 8% cement treated Anzali sand in σ_v = 200 kPa.

Table 3. Shear strength parameters of all three types of soils obtained from triaxial test.

Soil type	Coment (%)	C'(kPa)	ϕ' (deg)
Gorgan loess	0	15	23
	5	157	37
	8	244	38
Rasht clay	0	25	28
	5	121	38
	8	246	36
Anzali sand	0	—	—
	5	76	43
	8	198	43

shear stress increased significantly due to cement stabilization.

5. Failure criteria

Cement treated soils in low-confining pressure exhibit brittle failure and, at very high confining pressures, mostly show plastic failure [19]. In other words, Mohr-Coulomb failure envelope for cement treated soils is non-linear. Results show that the Mohr-Coulomb criteria, which is presented by Terzaghi [30] and is the basic of soil mechanics development, is valid for only limited range of stresses. Using the shear strength parameters of Terzaghi equation ($\tau_f = C + \sigma \tan \varphi$) may lead to some anomalies [9]. Several failure criteria such as Griffith theory, modified Griffith, Johntson criteria and Hoek-Brown failure criterion have been suggested for predicting the failure envelope of soil-cement samples. It must be noted that application of each failure criteria depends on material type and stress conditions.

Table 4. Shear strength parameters of non-treated and cement treated samples of Anzali sand from large scale direct shear test.

Coment content (%)	C'(kPa)	ϕ' (deg)
0	7	34
5	122	47
8	223	51

5.1. Griffith and modified Griffith crack theory

Griffith [20] proposed that the failure of brittle materials is governed by the initial presence of microcracks. Under uniaxial and biaxial compression, neglecting the influence of friction on the cracks when closed, and assuming that elliptical cracks will propagate from the points of maximum tensile stress concentration, a stress criterion is obtained as:

$$(\sigma_l - \sigma_3)^2 = -8\sigma_t(\sigma_l - \sigma_3) \quad (1)$$

$$\text{for} \quad \sigma_1 + 3\sigma_3 > 0$$

$$\text{if} \quad \sigma_1 + 3\sigma_3 < 0 \quad \text{then} \quad \sigma_3 = \sigma_t,$$

where σ_1 and σ_3 are the two principal stresses and σ_t is tensile strength of material. It can be seen from Eq. (1) that the Griffith stress criterion predicts a strength ratio of $R_G = \sigma_c/\sigma_t = 8$ [21]. When the tensile strength test data are not available, the general approach to estimate rock tensile strength makes use of the correlation between uniaxial compressive strength, σ_c, and tensile strength, σ_t, and applies the generally agreed relationship of $\sigma_c = R.\sigma_t$, where $R \sim 10$ [22]. Murrell [23] showed that the Griffith criterion can be represented in the Mohr plane in term of shear stress and normal stress as:

$$\tau^2 + 4\sigma_t\sigma_n - 4\sigma_t^2 = 0. \quad (2)$$

To allow for the frictional resistance on initially closed cracks, McClintock and Walsh [24] proposed a modification of Griffith's criterion. Following Brace [25] the modified criterion in terms of stresses, homogeneous at infinity, can be written as:

$$\sigma_l\left[(\mu^2+1)^{\frac{1}{2}}-\mu\right]-\sigma_3\left[(\mu^2+1)^{\frac{1}{2}}+\mu\right]=4\sigma_t, \quad (3)$$

where μ is the coefficient of joint friction. Brace [25] has shown that the fracture criterion, modified to account for the effects of crack closure in compression which can be represented by a limiting Mohr envelope. This line is straight having the equation in Mohr plane and expressed by Eqs. (4) and (5):

$$\tau^2+4\sigma_t\sigma_n-4\sigma_t^2=0 \quad \text{for} \quad \sigma_n<0, \quad (4)$$

$$\tau=2\sigma_t+\mu\sigma_n \quad \text{for} \quad \sigma_n>0. \quad (5)$$

5.2. Johnston failure criterion

An extensive study by Johnston and Chiu [26] on Melbourne mudstone resulted in a new failure criterion for soft rocks, given by:

$$\sigma'_{\mathrm{IN}}=(\frac{M}{B}\sigma'_{3N}+S)^B, \quad (6)$$

where M and B are intact material constant, S is the arameter that accounts for strength of discontinuities of rock or soil, with $S=1$ for the intact material. Also, σ'_{1N} and σ'_{3N} are effective principal stresses at failure, normalized by unconfined compressive strength, σ_c.

Based on a broad range of data for clays and rocks, Johnston [27] suggested that Eqs. (7) and (8) can be used to determine the B and M parameters as:

$$B=1-0.0172(\log\sigma_c)^2, \quad (7)$$

$$M=2.065+0.276(\log\sigma_c)^2. \quad (8)$$

5.3. The original Hoek-Brown failure criterion

The Hoek-Brown failure criterion is an empirical criterion developed through curve-fitting of triaxial test data. The conceptual starting point for the criterion was the Griffith theory for brittle fracture but the process of deriving the criterion was one of pure trial and error. The original Hoek-Brown criterion was proposed in [28] and is defined as:

$$\sigma_l=\sigma_3+\sqrt{m\sigma_3\sigma_c+s\sigma_c^2}, \quad (9)$$

where m is a constant depending on the characteristics of the rock mass, s is a constant depending on the characteristics of the rock mass, σ_c is the uniaxial compressive strength of the intact rock material, σ_1 is the major principle stress at failure, and σ_3 is the minor principle stress at failure.

The measured shear stress and normal stress values for non-treated, 5%, and 8% cement treated from triaxial tests are shown in Figure 7. Failure envelope lines including Mohr-Coulomb (obtained from Mohr circles), Griffith (Eq. (2)) and modified Griffith (Eq. (5)) are also drawn in this figure. Based on the Mean Squared Error analysis (MSE), to define μ coefficient, it is obvious that predicted shear strength, using modified Griffith theory has better agreement with measured shear strength compared with Griffith theory. It is clear that Mohr-Coulomb criteria results for cement stabilized soil are not applicable.

Measured normal stresses (σ_3 and σ_1) and those predicted by Johnston's theory (Eq. (6)) and also Hoek-Brown failure criterion (Eq. (9)) for non-treated and 5% and 8% cement treated specimens for all three types of soils are shown in Figure 8. Values of S parameter were determined by MSE analysis to obtain the lowest MSE value. For Hoek-Brown failure envelope m and s parameters were determined by trial and error regarding to the suggested range of values by Sjoberg [29]. It has been observed that both Johnston and Hoek-Brown criteria based on MSE analysis have a good compatibility with the cement treated test results. But Hoek-Brown cannot be used for non-treated samples under high confining pressures.

6. Conclusions

This study made an extensive laboratory testing for evaluating the effectiveness of cement treatment on shear strength parameters of soils encountered in southern coast line of Caspian Sea. Test results included unconfined compressive strength and consolidated-drained triaxialand large scale direct shear. Also failure envelope trend of cement treated samples was discussed by some criteria such as Mohr-Coulomb, Griffith, modified Griffith theory, the criterion suggested by Johnston, and Hoek-Brown failure criterion. According to the test results, the following conclusions can be stated:

- Addition of cement led to significant increase in unconfined compressive strength and modulus of elasticity for both soaked and unsoaked samples. Also reduction in failure strain and brittle type of rupture was observed.
- Triaxial test results indicated that cement treatment improved shear strength remarkably, but it changes the type of failure greatly from ductile to brittle behavior. 5% cement treated samples displayed planar and 8% cement treated showed both planar and splitting type of failure according to the confining pressures.
- It was derived that cemented treatment increases

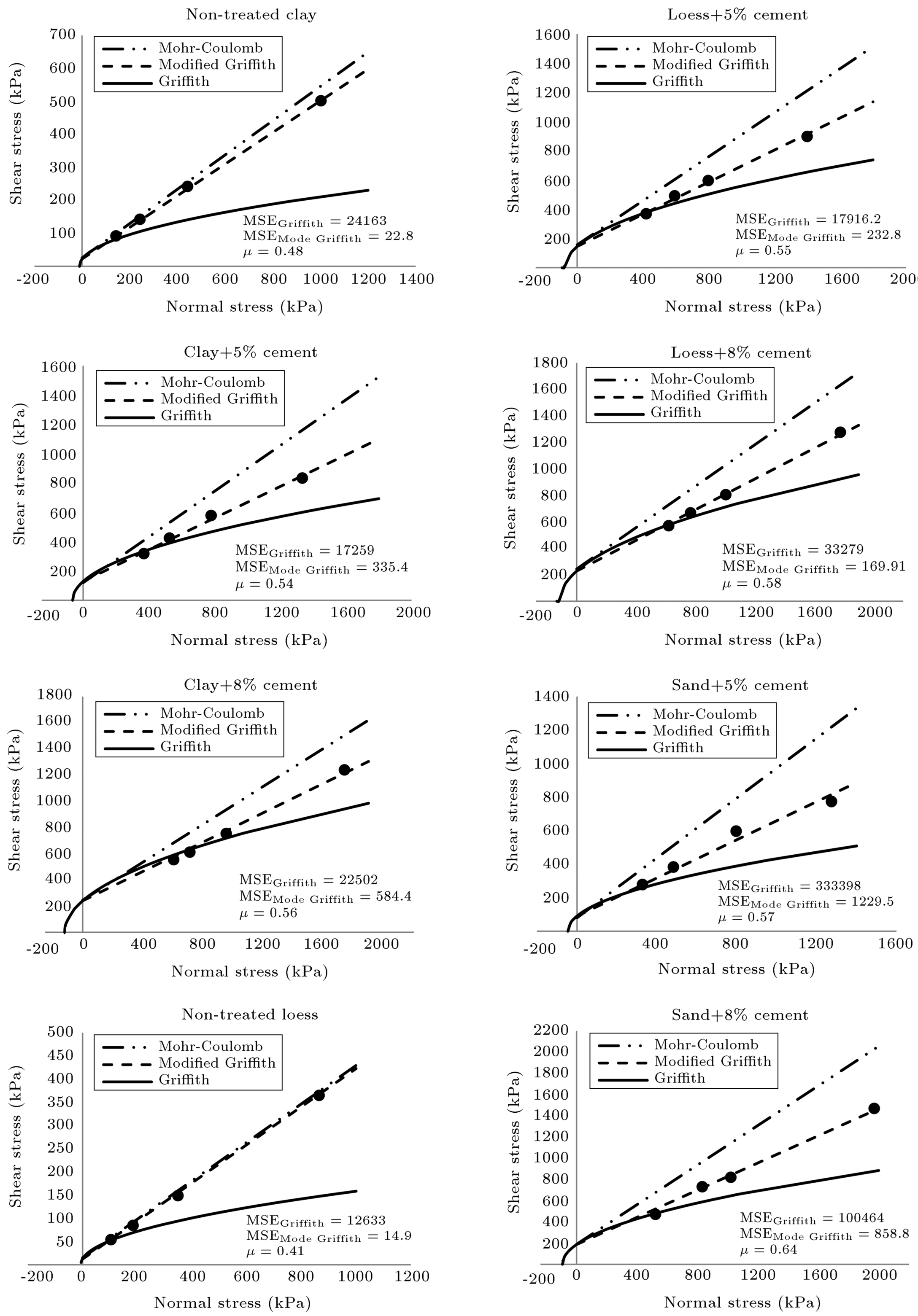

Figure 7. Triaxial test results and predicted failure envelope using Griffith, modified Griffith and Mohr-Coulomb theories.

the cohesion parameters due to the adhesion and bounding among the cemented soil particles.

- Confining pressure has an effective influence on soil behavior and it is essential to perform triaxial tests beside other simple and common tests such as UCS, CBR and DST.

- Based on MSE analysis, modified Griffith theory and Johnson criteria are compatible with obtained

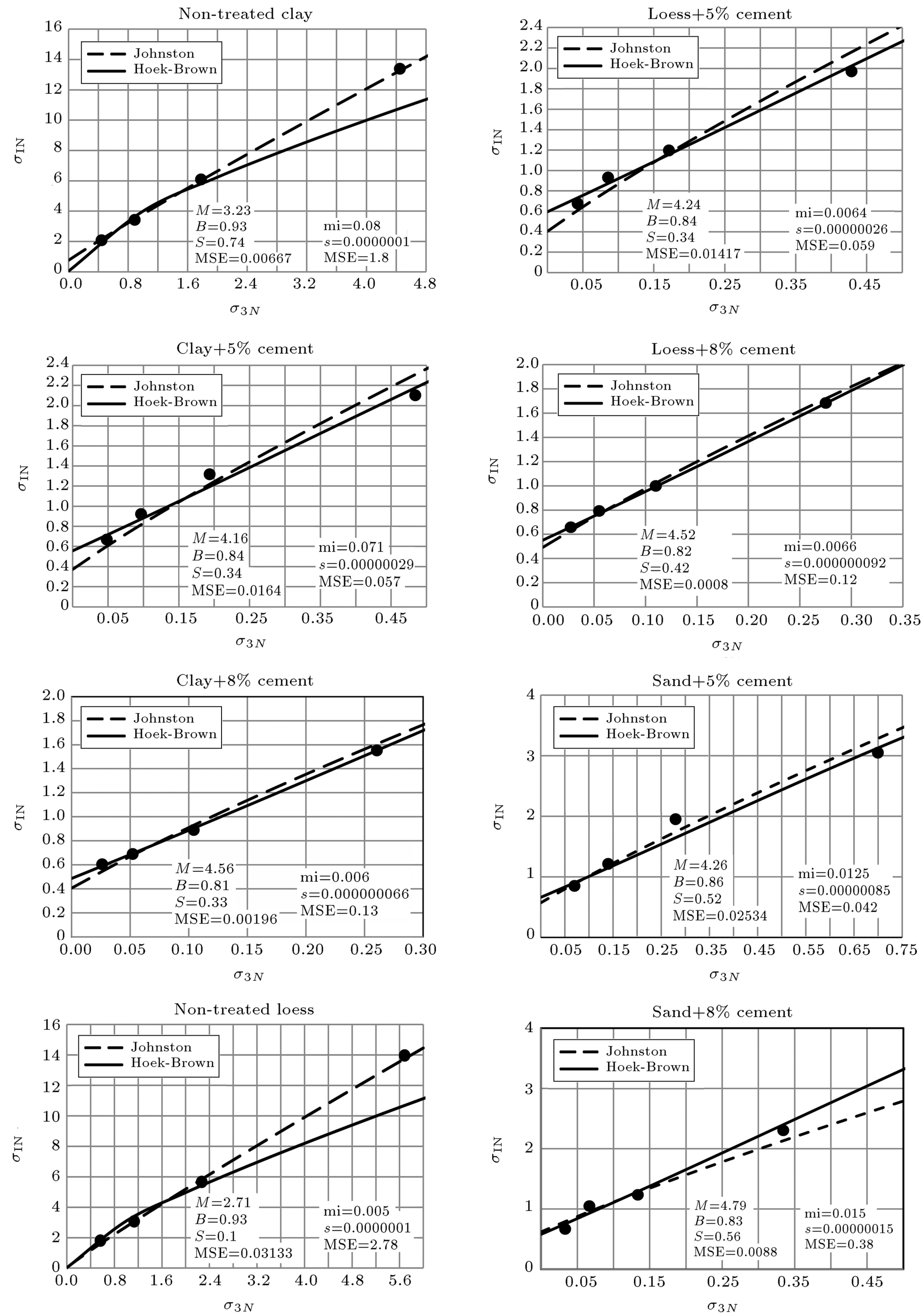

Figure 8. Triaxial test results and predicted failure envelope using Johnston criteria and Hoek-Brown failure criterion.

results. Therefore, the achievements are suitable for predicting the failure envelope of non-treated and cement treated soils. Also Hoek-Brown failure criterion can well describe the failure trend of cement treated soils, but it is not being used for non-treated soils. Moreover, using Mohr-Coulomb criteria for cement treated soils led to a number of errors and high values of friction angles.

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