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Effect of Cementation on the Shear Strength of Tehran Gravelly Sand Using Triaxial Tests

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Abstract

Coarse-grained soils of Tehran are cemented. These soils are heterogeneous not only in gradation and density but also in cementation. Due to this heterogeneity and the extreme difficulty in obtaining undisturbed samples, artificially cemented specimens using lime as the cementing agent are used to understand the effects of cementation on the shear strength parameters. A base soil with 45% gravel, 49% sand and 6% fine material was used, based on gradation curves of the coarse-grained soils of Tehran. Specimens were prepared by mixing the soil with 1.5, 3, and 4.5 percent lime and after curing were tested using triaxial compression tests. Specimens were also tested in uncemented and destructured conditions. The results of the tests indicated that the cementation increases peak shear strength, suggesting an increase in cohesion. Cemented specimens show a brittle failure mode at low confining pressure with a transition to ductile failure mode at higher confining pressures. The stress-strain curves for cemented specimens show a clear peak stress followed by a sudden drop in stress and strain softening. There is no clear peak in shear stress for uncemented and destructured specimens. The failure envelope is curved for cemented specimens. The influence of cementation on the friction angle of the tested specimens is a function of confining pressure, and degree of cementing. The results show that the peak friction angle at low confining pressure increases with increasing cementation.

Keywords: Cementation; Gravelly sand; Shear strength; Tehran alluvium

Introduction

Earth slopes and high vertical cuts are frequently observed to be stable for long period in the coarse-grained soils of Tehran. Their stability is often attributed to cementation effects that produce improved shear strength parameters.

In recent years many important projects such as metro and water conducting tunnels, deep cuts for highways and foundations have been constructed in

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Tehran alluvium. It is extremely difficult to acquire undisturbed samples using conventional soil sampling from this material, and so the designers do not consider the cementation effect on the shear strength and usually use test results on disturbed or remoulded samples. The engineering behaviour of these soils is quite different from that of uncemented soils.

There are some differences in the behaviour and shear strength of cemented sands and cemented gravelly sands, as well. Clough *et al.* [1], Airey [2], Coop and Atkinson [3], Schnaid *et al.* [4], Asghari *et al.* [5] and some other researchers reported the results of tests on cemented soils.

In order to understand the shear strength behaviour of cemented gravelly sand with special references to Tehran alluvium, a series of triaxial tests on artificially cemented specimens using lime as the cementing agent, were performed. This paper presents the test procedures and examines the influence of the cementation on the shear strength parameters of Tehran gravelly sand soils.

Geology of Tehran Alluvium

The soil used in the present study was obtained from the cemented coarse-grained alluvium of Tehran. Tehran alluvium has been deposited by frequent flow of floods and rivers originating from mountains in the north of Tehran city. This alluvium is divided into four series: A, B, C and D depending on the age of deposition. A detailed geology of Tehran alluvium is described by Berberian *et al.* [6]. The materials have been deposited in several alluvial fans and the layers consist of different gradings. Coarse-grained particles consist of fragments of tuff, shale and volcanic rocks. The particle shape is dominantly subangular. Cementation of the Tehran alluvium is a secondary event, and is predominantly deposited by ground water. Cement materials are mainly carbonate materials such as calcite. The cementation degree of this alluvium varies in different zones and sites.

Grading curves for the Tehran Alluvium taken from a number of sites in the north of Tehran are shown in Figure 1. The materials are generally gravelly sands and sandy gravels and can be classified according to the Unified Classification System as GW-GM, GW-GC, GP-GM, GM, GC, SW-SM, SW-SC, SM and SC. The natural dry unit weight of undisturbed samples varied between 16.5 to 19.5 kN/m^3, with an average of 18 kN/m^3.

Experimental Program

The cemented coarse-grained alluvium of Tehran is heterogeneous in grading, density and cementation. Moreover, it is extremely difficult to acquire undisturbed samples and prepare test specimens for triaxial testing. Therefore, to understand the mechanical behaviour and study the effects of cementation on the shear strength of these alluvial soils, artificially cemented specimens with lime were prepared and tested in this research.

A typical base soil with the grading curve shown in Figure 2 was chosen, based on gradation curves of Tehran coarse-grained soils and regarding the limitation on maximum particle size for triaxial testing of 100 mm diameter specimens. The physical properties of the selected soil are given in Table 1.

The base soil was gravelly sand with 45% gravel, 49% sand and 6% fine material. Hydrated lime, $Ca(OH)_2$, was used as the cementing agent for the artificially cemented specimens.

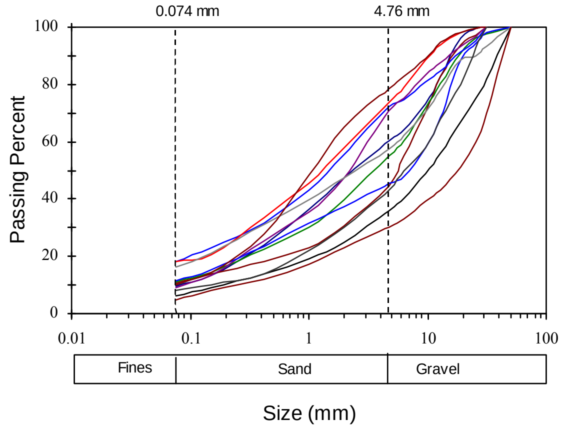

Figure 1. Grain size distribution curve of some samples obtained form various sites in north Tehran.

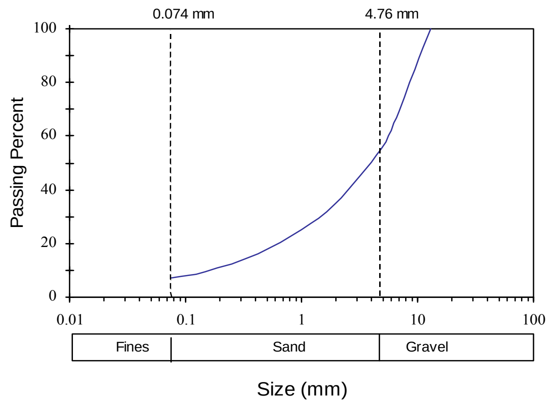

Figure 2. Grain size distribution curve of base soil used in this study.

Table 1. Average physical properties of base soil

Property	Value
Specific gravity (G_s)	2.58
Uniformity coefficient (C_u)	28
Percent gravel (>4.76 mm)	45 %
Percent fine material (<0.074 mm)	6 %
Liquid limit of passing 425 μm	44 %
Plastic index of passing 425 μm	16 %
Unified classification of soil	SW-SM

Grain size distribution, dry density and cementation were the main controlling factors in specimen preparation. After preparation of an appropriate amount of the base soil for each specimen, it was mixed with the cementing material (hydrated lime) and the correct amount of distilled water. Sections of high strength P.V.C. perforated pipes, 200 mm high and 100 mm diameter, were used as moulds for specimen preparation. The prepared soil-lime mixture was then placed in the mould in 10 layers and each layer was compacted statically to achieve the required height. The dry unit weight of the specimens was set to be 18 kN/m^3 with 8.5% water content. This is the average dry insitu unit weight of the deposit and the average optimum water content of the compacted base soil obtained from Proctor compaction tests. The soil was cured for 6 weeks before testing in order to produce the pozzolanic cementation compounds formed by reaction between lime and soil silica (fine particles of base soil), when they are saturated. The specimen was kept in the perforated moulds during the curing period. The temperature of the curing water tank was kept constant at 25 °C.

Cementation effects were studied in three main series of drained and undrained triaxial compression tests on uncemented, cemented and destructured specimens. Artificially cemented specimens with lime contents of 1.5, 3 and 4.5% of the weight of dry soil were prepared. Destructured soil was prepared by breaking down the artificially cemented 3% soils by hand. The destructured soil may well perform differently to the uncemented soil since the presence of the cementing agent will affect the overall grading. Therefore, to study the effects of cementation on various mechanical parameters of cemented soils it is better to compare the behaviour of the cemented soils with the associated destructured specimens, rather than the uncemented material.

The tests performed are summarized in Table 2. Consolidated drained and undrained triaxial compression tests were carried out. The specimens were set up in the triaxial apparatus, and were flushed through with deaired water under a small pressure gradient (10 kPa). After flushing, the cell and back pressures were increased in a controlled manner up to 300-400 kPa, so that the remaining air in the voids would be forced into solution. A check was made periodically on each sample to determine the pore pressure parameter B. When the B value was greater than 0.92, saturation was assumed to be complete. After completing of saturation, the specimen was then consolidated isotropically under the desired confining pressure. Shearing was carried out at a constant rate of strain of 3% per hour for drained tests (0.1 mm per minute) which was guaranteed full drainage during shear and 6% per hour for undrained tests as used by many researchers such as Schnaid *et al.* [4], Saxena and Lastrico [7], and Malandraki and Toll, [8].

Analysis of Test Data

The test results have been analysed using σ'_1, σ'_3, $(\sigma'_1 - \sigma'_3)$, ε_a, ϕ, c, where σ'_1 and σ'_3 are the axial and radial (confining) effective stresses on a cylindrical specimen, $(\sigma'_1 - \sigma'_3)$ is deviatoric stress or shear strength, ε_a is the axial strain, ϕ is the friction angle and c is the cohesion. A summary of test results is shown in Table 2.

Shear strengths of cemented specimens are considerable more than uncemented specimens. The common characteristic of all the results is that the peak strength increases with increasing of confining pressure.

Typical stress-strain curves for cemented and uncemented in drained and undrained condition are given in Figure 3. There is no clear peak in deviatoric stress for uncemented specimens but the presence of a peak for all cemented specimens is evident. In cemented specimens the peak strength is followed by strain softening.

Cemented specimens tested in undrained condition reach high level of strengths, but the opposite is true in uncemented specimens (*e.g.* Fig. 3). The difference between undrained and drained tests is likely to be due to the fact that volumetric straining (as happens in the drained tests) will contribute to breakdown of the cemented bonds (Malandraki and Toll [9]). When volume strains are prevented (in undrained tests) a higher stress ratio can be sustained before the bonds break down. Production of positive excess pore pressure in uncemented specimens causes a lower effective strength in undrained conditions than in drained condition.

Almost all uncemented specimens showed ductile failure modes during shear and underwent contraction. The destructured specimens also showed similar modes to the uncemented specimens. Cemented specimens, however, showed a brittle failure mode accompanied with a shear zone at low confining pressures with a transition to a ductile failure mode at higher confining pressures. Figure 4 shows a picture of the typical failure modes of specimens.

An absolute measure of such behaviour is provided by brittleness index (I_B) defined by the following expression by Consoli *et al.* [10]:

$$I_B = \frac{(\sigma_1' - \sigma_3')_f}{(\sigma_1' - \sigma_3')_u} - 1 \qquad (1)$$

in which $(\sigma'_1\text{-}\sigma'_2)_f$ is deviatoric stress at failure and $(\sigma'_1\text{-}\sigma'_3)_u$ is ultimate deviatoric stress. As the index decreases, approaching zero, the failure behaviour becomes increasingly ductile. As indicated in Figure 5, the brittleness index of specimens increases with cementation. With increases of confining pressure the brittleness index decreases.

The plot of stress ratio at failure in term of confining pressure (Fig. 6) shows that the cementation effect disappears at confining pressures more than about 1200 kPa. That means yielding of cemented bonds is occurred in applying of confining pressure.

The peak and ultimate strength values for the specimens are presented in Figures 7 and 8 respectively, plotted in the form of a t'-s' diagram with coordinates as follow that was defined by Lamb [11]:

$$t' = \frac{(\sigma_1' - \sigma_3')}{2} \qquad (2) \qquad\qquad s' = \frac{(\sigma_1' + \sigma_3')}{2} \qquad (3)$$

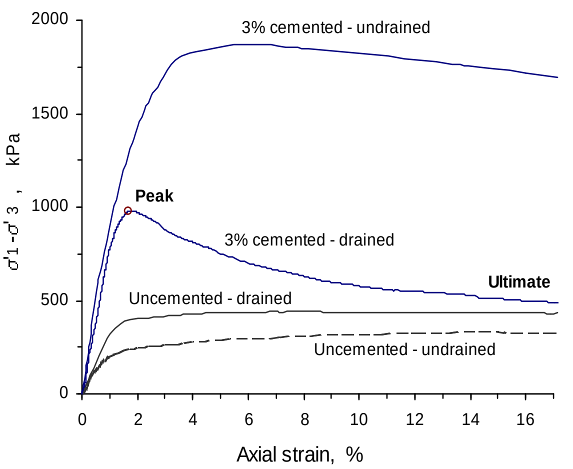

Figure 3. Examples of stress-strain curves of cemented and uncemented specimens (σ'_3 =110 kPa).

Table 2. Triaxial tests performed and summary of test results

Specimens	**Confining pressure (kPa)**	**Deviatoric stress (kPa)**	
		Peak	Ultimate
Uncemented			
Drained	25	155.8	135.0
	55	209.8	196.7
	110	439.6	431.2
	300	816.2	816.2
	500	1518.8	1518.8
	1000	2930.9	2930.9
Undrained	110	332.1	328.4
	300	651.1	638.1
	500	555.3	549.1
1.5% cemented			
Drained	110	916.43	389.9
	300	1679.9	1033.5
	500	2334.0	1736.7
Undrained	110	1625.7	1430.4
3% cemented			
Drained	25	684.5	160.5
	55	765.7	249.3
	110	978.1	484.9
	300	1747.9	1105.5
	500	2463.6	2011.5
	1000	3551.9	2741.7
Undrained	110	1871.9	1684.0
	300	2208.3	1800.1
	500	3030.8	2535.6
4.5% cemented			
Drained	10	1115.5	89.7
	110	1514.0	450.1
	300	2216.7	1402.6
	500	2970.7	1996.6
Undrained	110	2236.0	1797
Destructured			
Drained	55	288.7	210.3
	300	1171.6	1077.1
	500	1800.0	1745.0
Undrained	110	759.8	702.3
	300	931.5	900.8
	500	1329.2	1233.0

Figure 4. Prevailing failure modes of specimens; barreling in uncemented and destructured specimens and shear zone in cemented specimens.

The slope (α') and the intercept (a') of the envelopes plotted in Figures 7 and 8 were used for calculating effective friction angle (ϕ') and the effective cohesion (c') by the following expressions:

$$\sin\phi' = \tan\alpha' \quad (4) \qquad c' = \frac{a'}{\cos\phi'} \quad (5)$$

The slope of the failure envelope decreases as s' increases, indicating a loss of the cementation strength at failure. This is due to cementing bonds being broken down by consolidation at high pressures, as has been seen for other cemented materials (*e.g.* Malandraki and Toll [8]). This means that the influence of cementation is greater at low confining stresses and this effect reduces with an increase in confining pressure.

The calculated values of angle of peak and ultimate friction and peak and ultimate cohesion thus obtained are summarized in Table 3.

The ϕ'-values are found to be very close to each other for cemented and destructured specimens but are significantly lower for uncemented specimens. Peak friction angles are little affected by cementing material although it is slightly higher for cemented specimens. It is also noteworthy that the peak and ultimate friction angles are similar, but the ultimate cohesion is significantly less than the peak value.

Table 3. Shear strength parameters in peak and ultimate states

Specimens	**Peak**		**Ultimate**	
	$\phi_p'\ ^o$	c' (kPa)	$\phi_u'\ ^o$	c' (kPa)
Uncemented	36.0	23	36.8	19.3
1.5% cemented	40.0	111	39.3	3.1
3% cemented	40.4	144	39.2	12.5
4.5% cemented	40.5	247	40.0	11.7
Destructured	39.4	14	39.4	4.8

Regarding the curvature of failure envelopes in cemented specimens, it can be seen that the peak friction angle is dependent on confining pressure as it decreases with increasing confining pressure (Fig. 9). The cohesion intercept at peak strength (c'_p) increases with lime content, as is shown in Figure 10.

Conclusions

A gravelly sand that is representative of the Tehran alluvium has been artificially cemented with lime in order to study the effect of cementation on shear strength. Drained and undrained triaxial compression tests were conducted on uncemented, artificially cemented and destructured specimens.

The results of triaxial tests on specimens indicate that cementation increases peak shear strength, suggesting an increase in cohesion.

Cemented specimens show a brittle failure mode at low confining pressure with a transition to ductile failure mode at higher confining pressures.

The stress-strain curves for cemented specimens show a clear peak strength followed by a sudden drop in stress and strain softening. There is no clear peak in shear stress for uncemented and destructured specimens.

The failure envelope is curved for cemented specimens. The influence of cementation on the friction angle of the tested specimens is a function of confining pressure, and degree of cementing. The results show that the peak friction angle at low confining pressure increases with increasing cementation.

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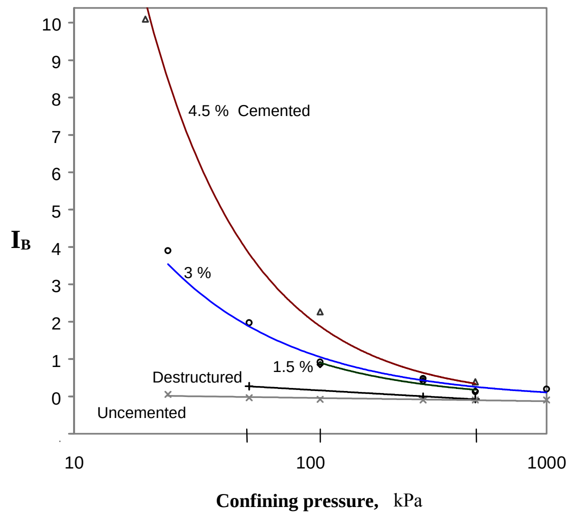

Figure 5. Brittleness index increases with cementation and decreases with confining pressure.

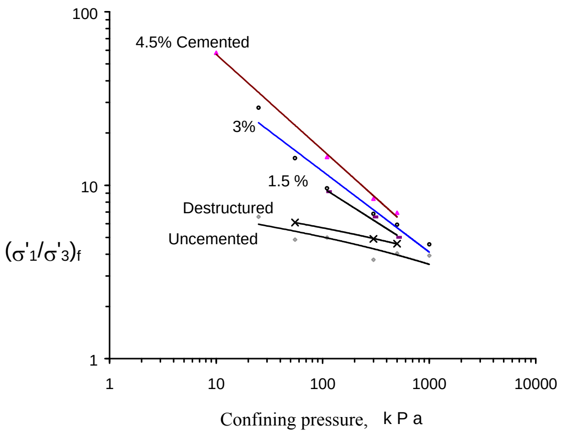

Figure 6. The stress ratio at failure decreases with increasing confining pressure.

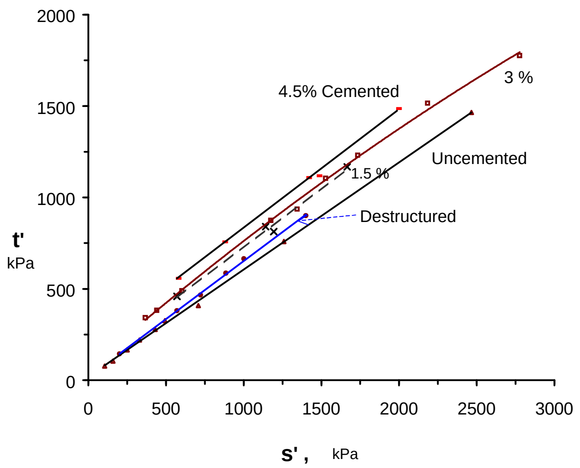

Figure 7. Peak shear strength envelopes of specimens.

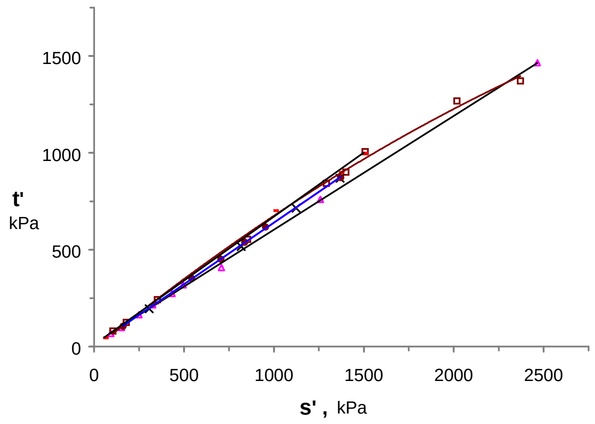

Figure 8. Ultimate shear strength envelopes of specimens.

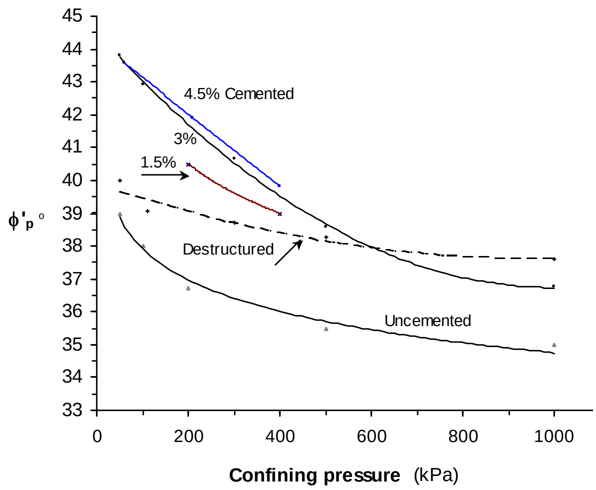

Figure 9. Variation of peak friction angle with confining pressure for three series of specimens.

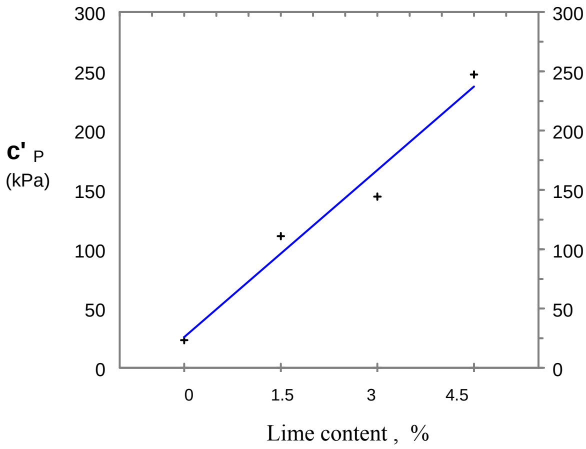

Figure 10. Variation of peak cohesion with lime content.

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