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A NEW APPROACH FOR INVESTIGATING THE ENGINEERING BEHAVIOR AND MECHANICAL PROPERTIES OF BONDED SANDY SOILS

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ABSTRACT: Cemented soils are widely encountered in the world, and with the growth of population, their exploitation becomes a necessity, especially, by building on diverse engineering infrastructures. The stability of those infrastructures is directly related to the nature and the properties of bonding in presence. This paper aims to investigate a new approach in studying the engineering behavior and mechanical properties of bonded sandy soils of different bonding levels, by using artificial soils similar to natural ones. The investigation was performed on four types of artificially bonded materials and one unbonded soil. The experiment was conducted in the laboratory under consolidated undrained isotropic triaxial compression tests subjected to three different levels of confining pressure. A new parameter, bonding index (B_i), was defined to implement the new approach. It is found that the variation of bonding and confining pressure alters significantly the response of soils. The cohesion intercept, the friction angle, the shear stress, and the brittleness index increase together with B_i, especially at low confining stress. However, B_i decreases when the confining pressure increases, restraining the role of bonding strength. Soils with higher B_i are identified to be more brittle. The determination of the confining stress level is found to be related to the bonding degree, which is expressed in terms of B_i. The bonding index is assessed as an efficient parameter whereby the bonded soil response can be investigated. Thus, the new approach offers an alternative for studying the engineering behavior of bonded soils.

Keywords: Bonded soil, Bonding index, Brittleness, Triaxial test, Sand

1. INTRODUCTION

In the natural world, some geo-materials have structures produced by the geological process which creates interparticle bonds. Due to the expansion of construction infrastructure into these soils, their study becomes a necessity. However, it is often difficult to describe their characteristics based on conventional soil and rock mechanics [1]. Residual soils are an illustrative case of these structured soils. Several studies have been carried out in order to study the behavior of natural and artificial structured soils [2-11]. It was concluded that their behavior is similar (e.g., [12]). Burland [13] defined the term “structure” as consisting of a combination of fabric and bonding. But, for the sake of simplicity, in this paper, the term “bonding” will be used to refer to structure.

Bonding results from the natural or artificial process. The study of the behavior of bonded sandy soils in practice is mostly based on parameters such as cement percentage, unconfined compression strength, and tensile strength. Airey [4] may be considered as one of the earlier researchers to propose a way to study structured materials behavior. Airey proposed the use of the degree of cementation as an indicator of structured behavior. In addition, for its estimation, the use of split-cylinder tests combined with dry-density determination to be done on off-cuts from natural structured specimens was recommended. In the same register, Gens and Nova [14] proposed a similar parameter called the degree of bonding, in which they set the uniaxial strength of soft rocks as a reference property. This is because the degree to which a bonded soil was structured (e.g., cementation degree) was an important parameter in the evaluation of its response. Besides, Abdulla and Kiousis [6] made an interesting observation by pointing out the difficulties arising from the estimation of the exact percentage of cement in structured soils. Furthermore, Schnaid et *al*. [7] suggested that even if structured soil samples are made with cement as a hydration agent, the cement content is not an appropriate parameter for evaluating cementation degree. Instead, they stated that unconfined compression strength is a reliable variable in the examination of cementation degree. They concluded that quantitative evaluation of cement degree in natural or artificial soils is important to assess the behavior. Another significant experimental study was conducted in [15] on cemented fine sandy materials by evaluating the effect of cement type on their mechanical behavior. Unconfined compression strength was used as an indicator of the cementing bonds.

In contrast, Haeri et *al*. [16], who investigated a reference parameter in a preliminary step by evaluating four variables: cement content, unconfined compression strength, Brazilian tensile strength, and triaxial compressive strength under low confining pressure, concluded that the cement content was a reliable parameter for their study. But at the same time, they stressed the fact that this criterion must not be applied when the cement acts as a filler of the voids. Recently, the conception of the degree of bonding proposed in [14] was adapted to artificial cemented materials in terms of indirect tensile strength and unconfined compressive strength in [8].

Therefore, from the literature, two important limitations are observed. Firstly, parameters such as cement content and unconfined compression can only be accounted as reliable quantitative parameters for artificially cemented soil samples made with a known percentage of cement, but they cannot be integrated as indicators for natural bonded soils tested under triaxial test. The latter remark concerns also the artificial soil specimen obtained from the thermal process in the laboratory, in which the cement percentage is unknown. In addition, the unconfined compression test cannot be performed on a saturated unbonded sample. Secondly, the alternative parameter like bonding degree as adapted in [8] does not integrate the aspect related to the bonding degradation and the confining pressure level under which a specimen is tested. Thus, for the study of bonded materials of unknown cement percentage under triaxial conditions, there is space for an investigation into a new reliable parameter for studying their engineering behavior and mechanical properties.

This paper presents a series of consolidated undrained isotropic triaxial (CU) tests performed on samples of different bonding levels and subjected to different confining stresses. Unbonded samples were prepared alongside and tested as the bonded materials. The behavior of bonded and unbonded soils was compared and analyzed based on a new approach using a reference parameter called "bonding index B_i". Interesting observations from the analysis are reported.

2. RESEARCH SIGNIFICANCE

This research has shown that bonded soils of unknown cement percentage can also be studied effectively under triaxial conditions based on an alternative approach. Indeed, most of the proposed methods in literature use cement percentage and unconfined compression strength as reference parameters. However, for CU tests conducted on saturated soils of unknown cement percentage, these parameters cannot be applied. Thus, other approaches have to be explored. The new method proposed here was evaluated as a reliable one. By using the proposed approach, it was found that all the main engineering behavior and mechanical properties of bonded soils were effectively assessed.

3. BONDING INDEX AS A REFERENCE PARAMETER

To establish a reliable reference parameter in order to overcome the aforementioned shortcomings, the reference state of material needs to be fixed. Bonded soils tested under triaxial conditions possess some major states such as first yield, second yield, bounding surface, critical state, and maximum rate of dilation. One of them should serve to establish the reference parameter.

Vaughan [2] defined the first yield as a stress state at which debonding begins. Afterward, with the increase in stress and strain, a second yield takes place and bong strength decreases partially, but this does not coincide with the complete degradation of the bonds. Complete breakage of the structure occurs at much larger strains. Also, determination of both first and second yields are still somewhat subjective since it is based on graphic plotted either at the natural [17] or log-log scales [18,19], where it is often difficult to determine the yield point. Furthermore, the two yield points cannot, by definition, be determined for unbonded soils. Thus, first and second yields cannot be recommended as the reference state.

At the critical state, the strength identified represents mostly the frictional strength since much of the bonds have been destroyed. It can be pointed out that after structured soil is sheared and failure has occurred, bonds are weakened. Hence, the bonding strength can be considered negligible compare to frictional strength at this stage. It is also uncertain to reach a critical state. Indeed, to reach the critical state the variation of either volumetric strain or excess pore water pressure under drained or undrained conditions, respectively, must be null. But this is not always the case. Therefore, the use of the critical state as the reference state seems inappropriate.

Bolton [20] defined the maximum rate of dilation as $(-d\varepsilon_v/d\varepsilon_1)_{max}$. In which ε_v is the volumetric strain while ε_1 the main strain, positive in compression. If the dilative behavior can be considered corresponding to extension response under the undrained regime, the mathematic evaluation cannot follow the same analogy since there is no variation of volumetric strain under undrained conditions. As a consequence, the point of maximum dilation is unsuitable as the reference state.

Bounding surface has been used as a set reference between unbonded and bonded soils by some authors (e.g., [9]). This surface is determined via the failure envelope, the latter being the limiting stress ratio, which a bonded geomaterial can sustain [21]. By definition, the bounding surface controls the process of destructuration through its interaction. Beyond this state, bonds are damaged considerably, and material strength tends to reach its residual value quickly. Consequently, the use of bounding surface as a reference state seems appropriate, for structured soils.

This is independent of the drainage regime in place (drained or undrained). Based on the preceding analysis, the bounding surface is chosen as the reference state. The reference parameter is named "bonding index B_i ". The determination of this parameter is based on the comparison between the strength response of unbonded and bonded soils. The construction of the bounding surface is made in q-p' space. Bounding surface is identified from the deviatoric stress q and the mean effective stress p' both corresponding to the maximum q/p' ratio [9]. The deviatoric stress q and the mean effective stress p', under asymmetric conditions, are defined by Eqs. (1) and (2), respectively.

$$q = \sigma_1 - \sigma_3 \tag{1}$$

$$p' = (\sigma_1' + 2\sigma_3')/3 \tag{2}$$

Furthermore, the bonding index is established based on q and p' values from $(q/p')_{max}$ under different bonding concentrations. Therefore, the bonding index is defined as a parameter whereby the level of bonding in a structured soil can be evaluated. it is also used to ascertain the gain of strength due to bonding in soil materials over the corresponding confining pressure.

The same logic can be used for unbonded material although in this case there is no bonding, the bonding surface is made purely for comparison purposes. The interval between two points of maximum q/p' ratio (p_b', q_b) and (p_u', q_u) of bonded and unbonded samples curves, respectively, at the same confining stress σ_3 represents the bonding degree of the structured soil, which can be expressed as: $\sqrt{(q_b - q_u)^2 + (p_b' - p_u')^2}$.

In which q_b, q_u, and p_b', p_u' are the deviatoric stresses of bonded and unbonded soils, and the mean effective stresses of bonded and unbonded samples, respectively. This expression is normalized to confining pressure σ_3. Then, the suggested parameter bonding index is defined as:

$$B_i = (1/\sigma_3)\sqrt{(q_b - q_u)^2 + (p_b' - p_u')^2} \tag{3}$$

Confining pressure $\sigma_3$is positive and non-null. In Eq. (3) B_i is equal to zero when bonding is absent, i.e. for unbonded or destructured samples, while B_i is superior to zero for bonded or structured materials.

The enhancement of mechanical properties of bonded soil such as cohesion intercept, the angle of friction intern, strength, and bonding yield stiffness is attributed to the structure developed between grains.

In the current study, the bonding strength is independent of curing time because of the nature of the samples used.

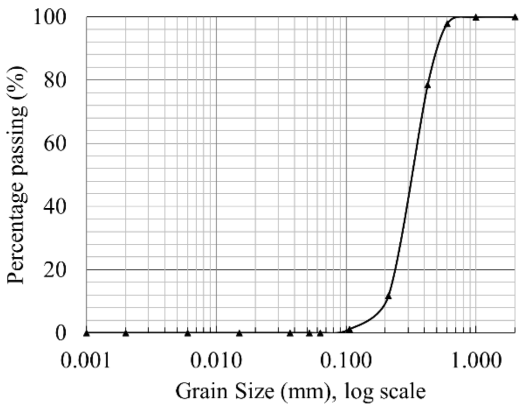


Fig.1 Particle size distribution of the Sile sand.

One should note that the bonding index (B_i) is calculated at single confining stress. It is then important to associate every B_i with the corresponding confining pressure. For instance, B_{i30} will be interpreted as the bonding index at the confining pressure of 30kPa. By considering Eq. (3) at various confining stresses, the effect of bonding on soil behavior can be gauged. This aspect will be discussed later in this paper.

4. MATERIALS TESTED AND EXPERIMENTAL PROCEDURES

4.1. Materials Tested

Using artificial bonded soils in the current study sought to simulate the bonded properties of natural structured soils such as residual soil [17]. It is made by mixing sand, kaolin, and water in a predetermined proportion, air drying it. Afterward, it is fired in a high-temperature furnace. This way of making a sample allows also to avoid variation of strength properties with time.

The "Sile" sand used in the testing (Fig.1) is widely commercialized, mostly in Eastern Turkey. This sand is from the district of Şile in Istanbul city. Specific gravity according to [22] of the solids is 2.70. This is predominantly quartz sand according to mineralogical analysis. The minimum and maximum index void ratios are 0.52 and 0.86, respectively, according to [23] and [24]. The Kaolin used had specific gravity, liquid limit, and plastic limits, respectively, of 2.61, 62%, and 34%. Liquid and plastic limits were performed according to [25].

The artificial soil used for this study was prepared by modifying the method proposed in [17]. All samples were prepared at the same index void ratio (e_0 =0.6). The procedure used for making samples are summarized as follows:

a. Cylindrical PVC molds (4.4 cm diameter and 9 cm height) were prepared. Every mold had two half parts for avoiding sample disturbance when removing the mold. Every piece of mold had twelve holes drilled in three rows. Two filter papers were prepared, one inside the mold capturing its shape and another placed at the bottom side of the mold as a base. Filter papers were interconnected with glue. Both half parts of the mold were fixed by two rubber bands (one at the top and the other at the bottom).

b. Sand and kaolin were dry mixed for around 5 minutes in a container. Sand and kaolin represented 87% and 13% by dry weight ratio, respectively. Distilled water was added to the sand-kaolin mixture. After trial sample preparation, the water contents of artificial bonded and unbonded samples were determined as 24% and 27%, respectively. Mixtures were made for a batch of six samples. The three components: sank, kaolin, and water were mixed for 2 minutes.
c. The wet mixture was carefully poured into the molds by using a spoon. Possible air trapped inside the molds was removed by vibrating a spatula. The Samples were left inside the molds for 3 days to starting the drying process by enabling samples to stand themselves. Thereafter, the two rubber bands and the two half parts of every mold were dismantled. Samples were left in filter papers to dry at room temperature for further 4 days, or until they had reached a constant weight.
d. Bonded samples were fired at different temperatures during a specific time to establish uniform bond strength (from fired kaolin), as specified in Table 1. Temperature and time duration varied according to the target level of bond strength. Thus, four categories of artificially bonded samples and one unbonded specimen were developed (Table 1). Unbonded samples were not fired.
e. Bonded and unbonded samples were trimmed carefully to 38 mm diameter by 76 mm high for the triaxial tests.

By varying temperatures and times of firing of artificial material made with sand and kaolin, their bond strength changes also [2]. Then, every category of bonded samples (B, C, D, E) has a different bonding degree. Even though the variation in bonding is not linear, but it increases in the following order: soils B, C, D, and E corresponding to the temperature level and time of firing prementioned in Table 1.

4.2. Experimental Procedures

Conventional isotropic triaxial consolidated undrained (CU) monotonic compression tests were performed on twelve artificially bonded and three unbonded specimens. All tested specimens were 38 mm by 76 mm as diameter and height, respectively, and with an initial void ratio of 0.6, and relative density of 90%. Each bonded sample was saturated by boiling. The procedure is the same as the one proposed in [17].

Table 1 Sample information

N°	Temperature of firing sample	Duration of firing sample	Confining stress (σ_3)	Soil remark	Soil state
1	-	-	30kPa	A	Unbonded
2	-	-	200kPa	A	Unbonded
3	-	-	700kPa	A	Unbonded
4	500°C	5 hours	30kPa	B	Bonded
5	500°C	5 hours	200kPa	B	Bonded
6	500°C	5 hours	700kPa	B	Bonded
7	750°C	3.5 hours	30kPa	C	Bonded
8	750°C	3.5 hours	200kPa	C	Bonded
9	750°C	3.5 hours	700kPa	C	Bonded
10	1000°C	3 hours	30kPa	D	Bonded
11	1000°C	3 hours	200kPa	D	Bonded
12	1000°C	3 hours	700kPa	D	Bonded
13	1250°C	2 hours	30kPa	E	Bonded
14	1250°C	2 hours	200kPa	E	Bonded
15	1250°C	2 hours	700kPa	E	Bonded

In general, once the sample was transferred to the testing equipment if no air bulb was allowed while transferring the sample, the backpressure was not necessary. But, as a precaution, only a small back pressure was applied during 3 hours to archived Skempton's parameter B value of at least 0.95.

For unbonded samples, a different procedure, the back-pressure method for saturating was followed because of their nature. The procedure would maintain for 24 hours, as they contain 87% of sand. The sample was considered saturated once Skempton's parameter B value reached at least 0.95.

Artificially bonded and unbonded samples were isotropically consolidated at three confining pressures 30kPa, 200kPa, and 700kPa defined as low, medium, and high levels, respectively. Consolidation time was set at 2 hours before shearing. Undrained shearing was finally applied at a constant displacement rate of 0.076 mm/min. Throughout the test, the cell pressure was kept constant while the axial stress was increased. The conventional triaxial testing procedures are more detailed in [26].

During the test, the excess pore water pressure variation was monitored by an automated pore water measurement system, while axial displacement was measured employing a Linear Variable Differential Transducer (LVDT). The axial load, however, was measured by a load cell of 5KN as the maximum capacity. All measurements achieved accuracies beyond the requirements in [27].

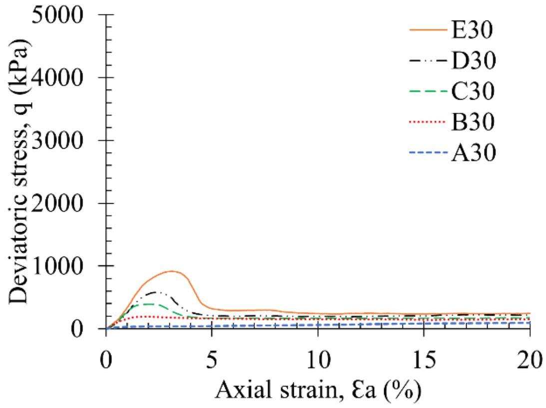
5. RESULTS AND DISCUSSIONS

5.1. Stress-strain Behavior

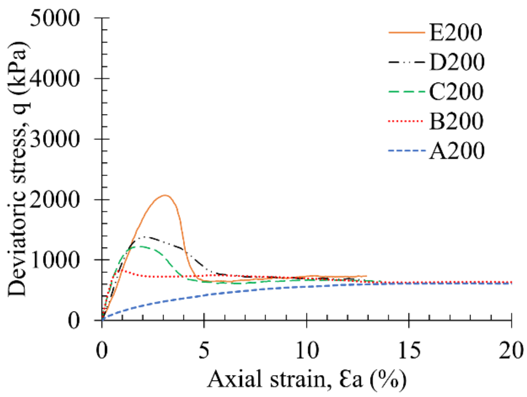
The stress-strain curves got in isotropic triaxial compression tests carried out on both bonded and unbonded samples are given in Fig.2(a), (b), (c) for samples subjected to 30kPa, 200kPa, 700kPa confining pressures, respectively.

It can be seen from Fig.2a that at low confining stress σ_3 = 30 kPa the unbonded sample ($B_i = 0$) sheared did not show a peak in deviatoric stress in the early stage (axial strain $\varepsilon_a < 4\%$) like the bonded soils. The bonded materials can be considered as displaying a fully cohesive peak strength. An interesting observation is that at the same time a common trend is observed when the shearing process progresses to high axial strain, around $\varepsilon_a = 20\%$. This is caused by the gradual loss of bonding strength after the bonds are broken at failure. It is in close agreement with the observation reported in [28]. The presence of bonding is testified by a non-null value of the bonding index B_i as explained in section 3. Unless, among bonded soils, Soil B displayed slight peak stress, which may be also because of the effect of scale, compared to the others. The same observation is seen under moderate confining pressure σ_3 = 200 kPa (Fig.2b). The deviatoric stresses vary together with bonding level expressed by B_i. The bonded grains permit the enhancement of contact surface and apparent cohesion whereby the strength of the soil is improved.

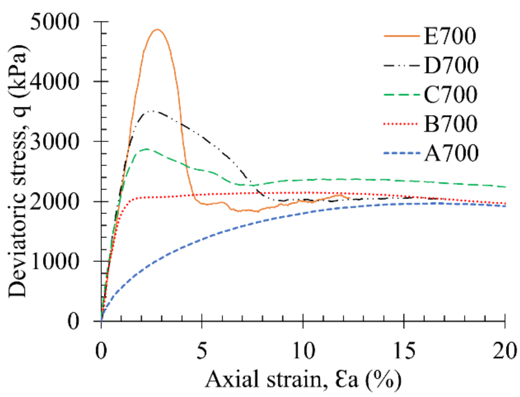
However, from Fig.2c under 700kPa, soils A and B did not show a distinct peak deviatoric stress like soils C, D, and E. The bonded sample of the highest B_{i30} (see Table 2), soil E, showed a sharp of deviatoric stress even at a high confining pressure level.



(a)



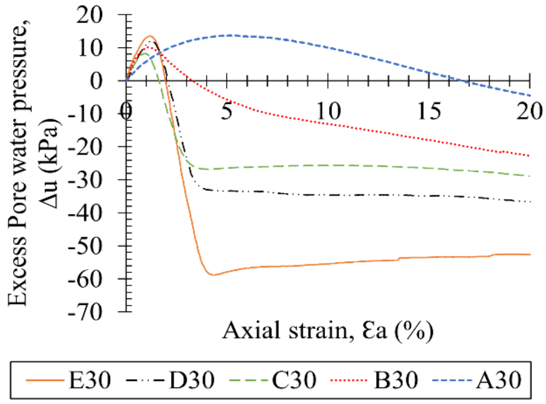
(b)



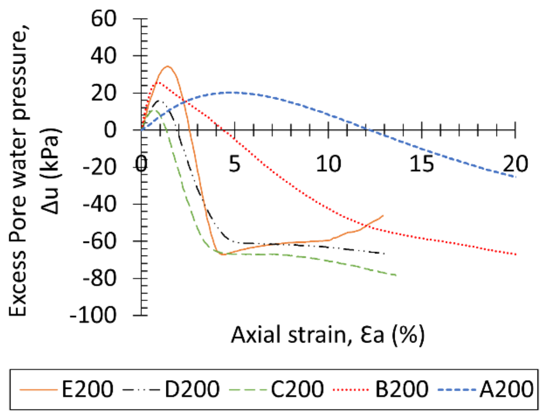
(c)

Fig.2 The stress-strain response of bonded and unbonded samples at: a) 30kPa, b) 200kPa, c) 700kPa.

This may suggest that this level of confining pressure was not as high as considered initially but still low, or moderate, compared to the level of bonding in presence. All samples sheared at high confining pressure seem to converge to a steady-state at high strain ($\varepsilon_a > 10\%$), which indicates that the residual strength is reached. This is undoubtedly due to some bonded clumps that were not completely broken. Other scholars also expressed this observation [29,30].



(a)



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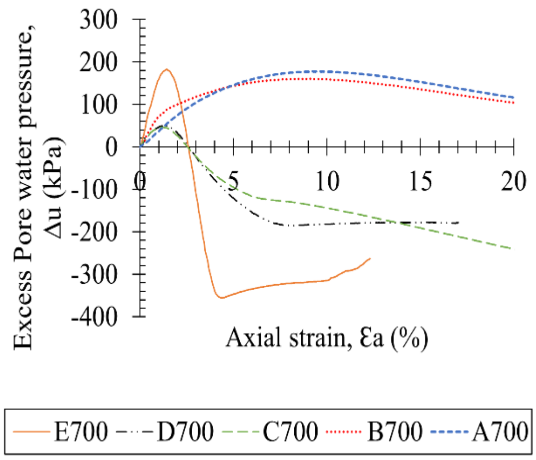

(c)

Fig.3 The variation of pore water pressure with the strain of bonded and unbonded samples at a) 30kPa, b) 200kPa, c) 700kPa.

5.2. Excess Pore Water Pressure and Strain Behavior

The set of data gathered from triaxial tests conducted on both artificial unbonded and bonded specimens illustrated in Figs.3(a), (b), (c) draw the excess pore water pressure against axial strain. Since the tests were performed under undrained conditions, the shearing process of samples is related to variation in excess pore water pressure Δu.

In terms of pore water pressure, all samples presented positive excess pore water pressure until the peak value was reached. Then, it is dropped down followed by a negative pore water pressure due to the dilative behavior of tested samples. A noticeable difference at an earlier stage of axial strain ($\varepsilon_a \leq 5\%$) was observed. Indeed, destructured samples (A) developed excess positive pore water beyond $\varepsilon_a =$ 5% while bonded soils B, C, D, and E globally showed an opposite tendency. The role played by bonding in the earlier stage of strain during shearing can justify it, before being broken later.

It is observed from Figs.3(a), (b), (c) that bonding significantly alters the pore water response of bonding materials, enhancing their dilative behavior mainly at low stress, as dilation implies that bond breakage must have occurred. Variation of excess pore water pressure up to the end of tests suggests that the soils did not achieve their true critical state, probably caused by the strain localization.

Analysis of bonded soil tested under drained conditions suggested that destructuration of bonded soils can be related to both compressive and dilatant volumetric strains [1,14], which are substituted by positive and negative pore water pressure under undrained conditions, respectively. It is observed from the above analysis that this is also borne out by the data presented.

5.3. Soil Properties Link up with Bonding Index B_i

The results of CU tests performed on unbonded and bonded specimens, at different confining pressure levels, were used to assess their properties associated with the bonding index in this section.

5.3.1. Correlation between bonding index B_i and bond strength

In subsection 5.1 the impact of bond strength at low confining pressure was outlined. Based on that, the bonding index B_i was calculated from Eq. (3) at low confining stress σ_3 =30 kPa. In the present study, the components of Eq. (3) were taken from Fig.4. The values of B_{i30} are shown in Table 2. The authors notice that B_{i30} follows the trend of bond strength. Indeed, the bond strength increases in the subsequent order: soils A, B, C, D, and E which correspond to B_{i30} values of 0.0, 3.9, 9.4, 13.8, and 20.0, respectively.

Table 2 Peak Strength Parameters, Deviatoric stress at first and second bond yield for unbonded and bonded soils in triaxial compression Tests

Soil state	Bonding index at $\boldsymbol{\sigma_3}$ = 30kPa, $\boldsymbol{B_{i30}}$	First bond Yield strength, $\boldsymbol{q_{Y1}}$ (kPa)			Second bond Yield strength, $\boldsymbol{q_{Y2}}$ (kPa)			Peak Strength	
		$\boldsymbol{\sigma_3}$=30	$\boldsymbol{\sigma_3}$=200	$\boldsymbol{\sigma_3}$=700	$\boldsymbol{\sigma_3}$=30	$\boldsymbol{\sigma_3}$=200	$\boldsymbol{\sigma_3}$=700	Cohesion intercept, c' (kPa)	Friction angle, $Øp'$ (Degree)
A	0.0	-	-	-	-	-	-	0.0	39.1
B	3.9	13.9	225.3	641.3	177.2	689.9	1505.2	30.9	40.5
C	9.4	14.9	321.3	1277.4	338.2	1114.1	2706.1	68.5	40.8
D	13.8	18.4	795.5	1364.1	448.3	1208.2	3262.6	87.5	43.5
E	20.0	30.3	1213.1	3067.0	645.0	1748.0	4449.0	105.8	47.8

Fig.4 outlines the variation of bonding with $(q/p')_{max}$. From this figure, the interconnection of grains due to bonding is illustrated by the gain of strength at $(q/p')_{max}$, where the bonding index is calculated. Consequently, the bonding index increases together with the shear stress.

5.3.2. The effect of bonding, illustrated by B_{i30}, on c' and $\emptyset'_p$

The cohesion intercept c' and peak friction angle $\emptyset'_p$ vary in the same direction with B_{i30} (Table 2). For instance, c' equals to 30.9kPa where B_{i30}=3.9 and increases to 68.5kPa when B_{i30} reaches 9.4. Likewise, $\emptyset'_p$ varies from 39.1° to 40.5° corresponding to B_{i30} values of 0.0 and 3.9. Similarly, Lade and Overton [31] suggested that the increase of cement content implies cohesion intercept and friction angle to increase also, especially at low stress. The energy applied at the peak state destroys bonding and then allows a sample to reach the residual strength [32]. A careful analysis reveals that B_{i30} is more correlated to c' than $\emptyset'_p$, especially for soils A, B, C. It is worth

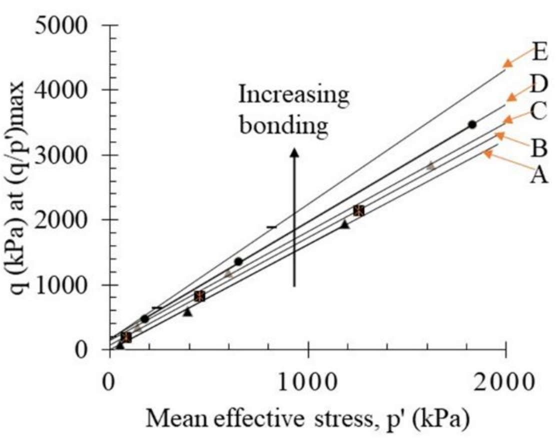

Fig.4 Bounding envelopes of artificial soils at various levels of bonding.

pointing out that there is no common agreement regarding the effect of bonding on $\emptyset'_p$. Some scholars (e.g., [32,33]) identified the increase in $\emptyset'_p$ as a result of bonding enhancement, traduced by the augmentation of the slope of the failure envelope. However, others reported no variation in $\emptyset'_p$ while the bonding level increases, illustrated by constancy in the slope of the failure envelope (e.g., [34,35]). This justifies the fact that some authors (e.g. [12,36]) suggested that variation in bonding should be more likely related to cohesion intercept than friction angle.

5.3.3. Variation of the first and second bond yields strength with B_{i30}

It is readily observed from Table 2 that the level of bonding present influences significantly the mechanical properties of samples. The first yield q_{Y1} and the second yield q_{Y2} strengths augment as a consequence of bonding strength improvement illustrated by bonding index B_{i30}. The first and second bond yields were identified based on the method proposed in [19], where both yields are expressed in terms of tangential stiffness versus axial strain plotted to a log-log scale. Comparing the variation of q_{Y1} and q_{Y2} with B_{i30}, one can see that they increase together. At low confining pressure 30kPa for example, q_{Y1} varies from 13.9kPa to 30.3kPa when B_{i30} increases from 3.9 to 20.0, respectively. The increase in bonding enables higher structured samples to develop higher yield stress than the lower bonded soils. The variation of cement content with yield strength was reported in the literature [37,38], but for soils of unknown cement percentage, the proposed method seems to be useful.

5.3.4. Effect of confining stress on B_i

Fig.5 outlines the role of bonding in structured materials sheared at different confining pressure levels. The bonding strength seems to play a big role at low stress, and as the confining pressure level

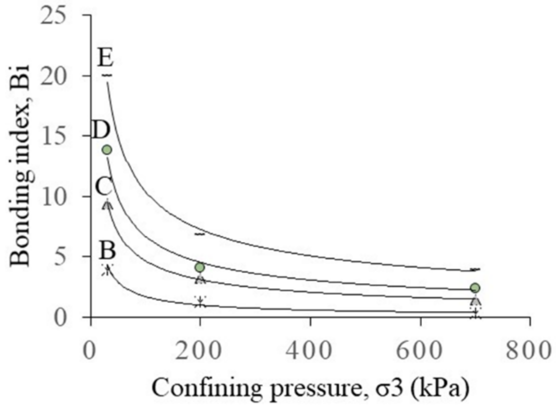

Fig.5 Effect of confining pressure on the bonding index of four bonded soils.

intensifies bonding strength is restrained. The contribution of bonding to the soil response becomes lesser because of the breakage of the bond structure [2]. The comparison between soils B and E shows that samples from E, of the highest bonding level, even at high confining pressure level still shows a non-negligible impact of bond on the specimen response, which is illustrated by B_i values B_{i700}=3.93 for soil E higer than B_{i30} =3.9 for soil B. The aforementioned observation suggests that bonds present in the structured soils still play a role in the total response at high confining stress, and the bonding strength evaluation is related to the bonding index B_i. Thus, it is necessary to identify the so-called "high confining pressure level" clearly for a specific soil, at which the bonding index B_i should be close to zero. The last observation suggests that the confining pressure level under what soil is tested is related to its B_i. The bonding index B_i is higher at low confining pressure because of the influence of bonding. This is consistent with the observation reported in [12], where at zero effective confining pressure the deviatoric peak stress mobilized in the triaxial test was suggested to correspond to a fully cohesive (i.e., non-dilative) shear strength.

Therefore, the variation of the bonding index with confining pressure can be regrouped in three stages:

(1) The first stage is at low confining pressure where an increase of confining pressure impacts significantly the role played by bonds. This is indicative of the role bonding plays at low stress in structured soils; the bonding in presence entirely controls the major response of soil up to failure, also the grains of bonded material sustain higher limiting stress ratios (q/p') than those of the unbonded soil.

(2) The second stage is what can be qualified as a transitional level between the first and third stages. In this stage, the effect of bonding strength on the response of the soil becomes relatively less, or moderate, because the bonds only control partially the behaviour of soil at failure. Consequently, bonded soils still display higher strengths than those of the unbonded material.

(3) The third stage is where the effect of bonding on the general behavior of soils becomes least and negligible compared to fiction strength. In this phase, the increment of confining pressure has a limited effect on bonding strength, since it is already lower. And, the bonding strength contributes less during the shearing process. B_i will tend to reach zero. As a result, the strength of bonded materials becomes very close to that of unbonded soils and, soil response is based essentially on the frictional strength component.

Interestingly, a parallel can be established with the observation revealed in [5], who identified four zones of behavior for a bonded soil sheared under classical drained and undrained paths. This can be considered a further validation of the bonding index as a key indicator in analyzing the behavior of structured soil.

5.3.5. Variation of maximum q/p' ratio with B_i

B_i plotted against $(q/p')_{max}$ expresses the evolution of bonding strength (Fig.6). In Fig.6, B_i corresponds to unbonded (A) and bonded soils (B, C, D, E) tested at three confining pressures (30kPa, 200kPa, and 700kPa). It can be seen that $(q/p')_{max}$ increases as well as B_i. Besides, this variation seems to be pronounced and consistent at low stress (30kPa).

The path of soils tested at medium and high confining pressures converges. As the applied confining stress increases the bonding strength is restrained, causing the bonded soils to sustain a lower $(q/p')_{max}$. Soils of higher B_i sustain a higher $(q/p')_{max}$ than those of lower B_i, at a specific cell pressure. The increase of confining pressure is accompanied by $(q/p')_{max}$ loss, illustrating the reduction of bonding effect on the soil response when the stress increases. This behavior has been found in

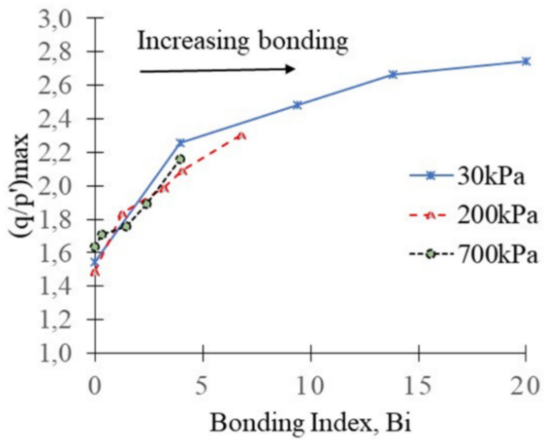

Fig.6 Maximum $\mathbf{q/p'}$ ratio against Bonding index, $\mathbf{B_i}$.

other bonded soils, for instance, in artificially cemented material investigated after being cured under stress [39]. Therefore, there is evidence in the results presented here that the new approach, based on B_i, can contribute effectively to the analysis of bonded soil behavior.

5.3.6. Change in brittleness response of soils

The brittleness index, I_B, expresses the loss of strength after archiving the peak deviatoric stress, q_p, as specified in Eq. (4) [40] as

$$\boldsymbol{I_B = (q_p - q_r)/q_p} \quad (4)$$

Where q_r is the residual deviatoric stress after the initial peak. In the present study q_r was considered at $\varepsilon_a = 20\%$ or at the end of the test, whatever comes first. I_B varies from 0 to 1. $I_B = 1$ indicates a highly brittle response, while $I_B = 0$ is a synonym of total ductile behavior. In general, I_B is one of the main characteristics of bonded soils.

Confining pressure is correlated to the brittleness index in Fig.7. By observing this figure, one can state that the increase in confining pressure involves the decrease in brittleness of tested materials. When comparing curves of soils A, B, C, D, and E, it can be seen that I_B and bonding increases dramatically while confining pressure drops. A moderate increase in ductileness for unbonded soil (A) is shown and accompanied by the augmentation of stress in contrast to bonded materials.

Fig.8 shows the evolution of B_i, I_B, and σ_3. The highest brittle response is set at low confining pressure, particularly for the higher bonded specimen (e.g., E). Some researchers (e.g., [30,33,36]) presented similar test results for bonded soils, where the stress-strain behavior of structured samples changes from brittle to ductile because of the increase in confining stress. From Fig.8, it is also observed that the behavior of higher bonded materials still appears as highly brittle even at high confining

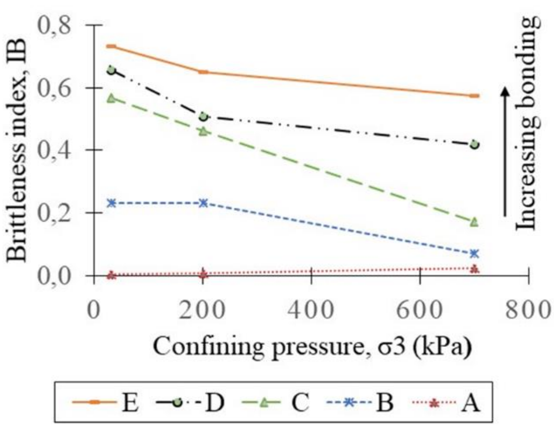

Fig.7 Variation of Brittleness index with confining pressure

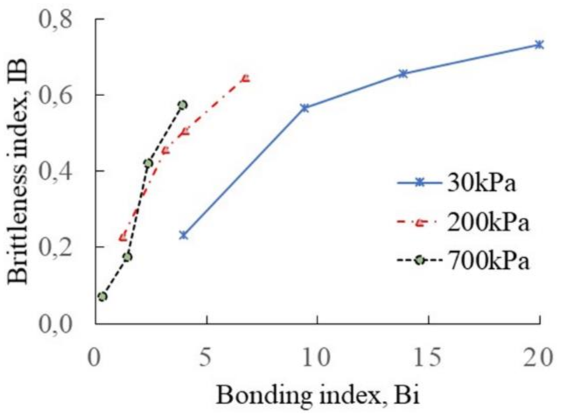

Fig.8 variation of Brittleness index in respect to Bonding index

stress. This supports the observation made previously, which suggests that the stress level (700kPa) is still low, or moderate, for higher bonded soils such as E. Furthermore, the observation made in [16] is confirmed, in which the increment of cement percentage increases the brittleness property of coarse-grained soils, this shows the usefulness of the new approach especially for soils of unknown cement percentage. Thus, it is noticed that the soil of higher B_i is less ductile.

The results reported herein suggest that, once $(q/p')_{max}$ is determined and B_i is established from the knowledge of Eq. (3), the behavior of structured soils might be assessed and analyzed. This finding is significant: the determination of the bonding strength level is an important step in the evaluation and evolution of the response of bonded soils. Samples taken from in-situ can be studied using this approach, and civil engineers can design appropriately the structures to be built on. Thus, the proposed approach offers an alternative means for the geotechnical analysis and interpretation of the response of structured soils.

6. CONCLUSIONS

An investigation was conducted in the laboratory to study the behavior of structured soils with different levels of bonding tested under isotropic consolidated undrained triaxial tests. Twelve artificial bonded and three unbonded samples were subjected to a shearing load process at three different confining pressures levels (30kPa, 200kPa, and 700kPa). The results were analyzed based on a new approach using the bonding index B_i as a reference parameter. The latter was found to be a reliable variable for the analysis of the structured material. From the present work, the following conclusions can be drawn:

- The variation of bonding and confining pressure impact significantly the response of soils. The change in confining stress contrasts with the variation of the soil stiffness. Bonding meaningfully alters the excess pore water behavior of bonded materials and enhances their dilative behavior, mainly at low stress (30kpa).

- Some parameters such as the cohesion intercept, friction angle, deviatoric stress, and $(q/p')_{max}$ are closely related to the bonding index, as a result of the bonding level of the tested specimens.
- The determination of the confining stress level was found to be closely related to the bonding index. For instance, the high confining stress level is defined where the bonding index is close to zero.
- Three stages were identified where the effects of bonding are divergent according to the stress level. The first stage, at low confining pressure (30kPa), was manifested by high B_i value which traduces the role played by bonds as they control the major response of soil up to failure. In the second stage, bonding strength contributes relatively less, or moderately, to the response of the soil. The third stage or the stage of high confining stress level was identified as where a major part of the stress is carried by the frictional strength component and the bonding strength is negligible. At the third stage, the value of B_i is equal or closed zero.
- The new parameter developed, bonding index B_i, satisfactorily enabled the analysis of the behavior of bonded soils, capturing their main mechanical characteristics.

7. REFERENCES

[1] Leroueil S., and Vaughan P.R., The general and congruent effects of structure in natural soils and weak rocks. Géotechnique, 40(3), 467-488, 1990. https://doi.org/10.1680/geot.1990.40.3.467

[2] Vaughan P.R., Characterising the mechanical properties of the in-situ residual soil. Proc. 2nd Int. Conf. Geomechanics Tropical Soils. Singapore 2, 469-487, 1988.

[3] Wesley L.D., Influence of structure and composition on residual soils. ASCE J. Geotech. Eng. 116(4), 589-603, 1990. https://doi.org/10.1061/(ASCE)0733-9410(1990)116:4(589)

[4] Airey D.W., Triaxial testing of naturally cemented carbonate soil, J. Geotechnical Engineering. ASCE 119(9), 1379-1398, 1993. https://doi.org/10.1061/(ASCE)0733-9410(1993)119:9(1379)

[5] Malandraki V., and Toll D.G., The definition of yield for bonded materials. Geotech. Geol. Eng. 14(1), 67-82, 1996. https://doi.org/10.1007/BF00431235

[6] Abdulla A.A., and Kiousis P.D., Behavior of cemented sands: I. Testing. Int. J. Numer. and Analytical Methods in Geomech. 21(8), 533-547, 1997. https://doi.org/10.1002/(SICI)1096-9853(199708)21:8<533::AID-NAG889>3.0.CO;2-0

[7] Schnaid F., Prietto P.D.M., and Consoli N.C., Characterization of cemented sand in triaxial compression. ASCE J. Geotech. Geoenviron. Eng. 127(10), 857–868, 2001. https://doi.org/10.1061/(ASCE)1090-0241(2001)127:10(857)

[8] Cardoso R., Ribeiro D., and Neri R., Bonding effect on the evolution with curing time of compressive and tensile strength of sand-cement mixtures. Soils Found. 57(4), 655-668, 2017. https://doi.org/10.1016/j.sandf.2017.04.006

[9] Ali Rahman Z., Toll D.G., and Gallipoli D., Critical state behaviour of weakly bonded soil in drained state. Geomech. Geoengng. 13(4), 233-245, 2018. https://doi.org/10.1080/17486025.2018.1454608

[10] Dararat S., Kongkitkul W., Posribink T., and Jongpradist P., Compressive strength and quasi-elastic modulus behaviours of a cemented lateritic soil. I. J. GEOMATE, Feb., 2021, Vol.20, Issue 78, pp. 158-164. https://doi.org/10.21660/2021.78.GX128

[11] Zein A.M., and Elsharief A.M., Shear strength behaviour of remoulded fine-grained soils treated with hydrated lime. I. J. GEOMATE, July, 2021, Vol.21, Issue 83, pp.-142-149. https://doi.org/10.21660/2021.83.j2107

[12] Lo S.C.R., Lade P.V., and Wardani S.P.R., An experimental study of the mechanics of two weakly cemented soils. Geotech. Test. J. 26(3), 328-341, 2003. https://doi.org/10.1520/GTJ11301J

[13] Burland J.B., On the compressibility and shear strength of natural clays. Geotechnique, 40(3), 329-378, 1990. https://doi.org/10.1680/geot.1990.40.3.329

[14] Gens A., and Nova R., Conceptual bases for a constitutive model for bonded soils and weak rocks. Geotechnical Engineering of Hard Soils – Soft Rocks, Balkema. Rotterdam, pp. 485–494. 1993.

[15] Ismail M.A., Joer H.A., Sim W.H., and Randolph M.F., Effect of cement type on shear behavior of cemented calcareous soil. ASCE J. Geotech. Geoenviron. Eng. 128(6), 520–529, 2002. https://doi.org/10.1061/(ASCE)1090-0241(2002)128:6(520)

[16] Haeri M.S., Hamidi A., Hosseini S.M., Asghari E., and Toll D.G., Effect of cement type on the mechanical behavior of a gravely sand. Geotech. Geol. Eng. 24, 335-360, 2006. https://doi.org/10.1007/s10706-004-7793-1

[17] Maccarini M., Laboratory studies of a weakly bonded artificial soil. PhD thesis, University of London. 1987.

[18] Bressani L.A., Experimental properties of bonded soils. PhD thesis, University of London,

1990.
[19] Toll D.G., and Malandraki V., Triaxial testing of a weakly bonded soil. Proceedings of an international symposium on the geotechnical engineering of hard soils-soft rocks, Athens (eds A. Anagnostopoulos, F. Schlosser, N. Kalteziotis & R. Frank), Vol. 1. pp. 817-824. Rotterdam: Balkema. 1993.
[20] Bolton M.D., The strength and dilatancy of sands. Géotechnique, 36(1), 65–78, 1986. https://doi.org/10.1680/geot.1986.36.1.65
[21] Malandraki V., and Toll D.G., Drained probing triaxial test on a weakly bonded artificial soil. Geotechnique, 50(2), 141–151, 2000. https://doi.org/10.1680/geot.2000.50.2.141
[22] ASTM D 854-14, Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer. ASTM International, West Conshohocken, PA. 2014.
[23] ASTM D 4254-16, Standard test method for minimum index density and unit weight of soils and calculation of relative density. ASTM International, West Conshohocken, PA. 2016.
[24] ASTM D 4253-16, Standard test method for maximum index density and unit weight of soils using a vibratory table. ASTM International, West Conshohocken, PA. 2016.
[25] ASTM D 4318-00, Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. ASTM International, West Conshohocken, PA. 2003.
[26] Bishop A.W., and Henkel D.J., The measurement of soil properties in the triaxial test. 2nd edn. London: Edwin Arnold. 1957.
[27] ASTM D 4767-11, Standard test method for consolidated undrained triaxial compression test for cohesive soils. ASTM International, West Conshohocken, PA. 2020.
[28] Coop M.R., The mechanics of uncemented carbonate sands. Géotechnique, 40(4), 607-626, 1990. https://doi.org/10.1680/geot.1990.40.4.607
[29] Rios S., Viana da Fonseca A., and Baudet B.A., On the shearing behaviour of an artificially cemented soil. Acta Gèotechnica 9(2), 215–226, 2014. https://doi.org/10.1007/s11440-013-0242-7
[30] Porcino D.D., and Marcianò V., Bonding degradation and stress–dilatancy response of weakly cemented sands. Geomech. Geoengng. 12(4), 221-233, 2017. https://doi.org/10.1080/17486025.2017.1347287
[31] Lade P.V., and Overton D.D., Cementation effects in frictional materials. ASCE J. Geotech. Geoenviron. Eng. 115 (10), 1373–1387, 1989. https://doi.org/10.1061/(ASCE)0733-9410(1989)115:10(1373)
[32] Wang Y.H., and Leung S.C., Characterization of cemented sand by experimental and numerical investigations. ASCE J. Geotech. Geoenviron. Eng. 134(7), 992–1004, 2008. https://doi.org/10.1061/(ASCE)1090-0241(2008)134:7(992)
[33] Marri A., Wanatowski D., and Yu H.S., Drained behaviour of cemented sand in high pressure triaxial compression tests. Geomech. Geoengng. 7(3), 159–174, 2012. https://doi.org/10.1080/17486025.2012.663938
[34] Porcino, D., Marciano, V., and R. Granata. 2012. Static and Dynamic Properties of a Lightly Cemented Silicate-Grouted Sand. Canadian Geotechnical Journal. 49(10), 1117–1133. https://doi.org/10.1139/t2012-069
[35] Fu, Z., S. Chen, and H. Han. 2020. Large-Scale Triaxial Experiments on the Static and Dynamic Behavior of an Artificially Cemented Gravel Material. European Journal of Environmental and Civil Engineering. https://doi.org/10.1080/19648189.2020.1792350
[36] Mola-Abasi H., Saberian M., Semsani S.N., Li J., and Khajeh A., Triaxial behaviour of zeolite-cemented sand. Proceedings of the Institution of Civil Engineers - Ground Improvement 173(2), 82-92, 2018. https://doi.org/10.1680/jgrim.18.00009
[37] Kasama K., Ochiai H., and Yasufuku N., On the stress-strain behaviour of lightly cemented clay based on an extended critical state concept. Soils Found. 40 (5), 37–47, 2000. https://doi.org/10.3208/sandf.40.5_37
[38] Kamruzzaman A.H., Chew S.H., and Lee F.H., Structuration and destructuration behavior of cement-treated Singapore marine clay. ASCE J. Geotech. Geoenviron. Eng. 135(4), 573–589, 2009. https://doi.org/10.1061/(ASCE)1090-0241(2009)135:4(573)
[39] Dalla-Rosa F., Consoli N.C., and Baudet B.A., An experimental investigation of the behaviour of artificial soil cured under stress. Géotechnique, 58(8), 675–679, 2008. https://doi.org/10.1680/geot.2008.58.8.675
[40] Bishop A.W., Shear strength parameters for undisturbed and remoulded soil specimens. In: Parry RHG, editor. Stress-strain Behaviour of Soils. Proceedings of the Roscoe Memorial Symposium. G. T. Foulis & Co. Ltd. 3–58. 1971.