EXPERIMENTAL INVESTIGATION OF THE INFLUENCE OF RELATIVE EFFECTIVE DIAMETER ON THE ULTIMATE SHEAR STRENGTH OF PARTIALLY SATURATED GRANULAR SOILS

Izvleček

Predstavljena eksperimentalna preiskava opisuje temeljne raziskave za raziskovanje vpliva relativnega učinkovitega premera na mehansko obnašanje delno zasičenih zrnatih zemljin. V tem okviru je bila izvedena vrsta laboratorijskih triosnih preizkusov na dveh skupinah peščenih zemljin s tremi maksimalnimi predmeti in dvema različnima učinkovitim diametrom. Preiskovanje je bilo obdelano v laboratoriju po metodih molkega usedanja pri začetnem gostotnem stanju, preizkušeni pri različnih parametrih Skemptonovega pora in obteženi s stalnim bočnim tlakom. Pridoženi podatki kažejo, da bi lahko bil parameter Skemptonovega pora koreliran z mejno strižno trdnostjo preizkušenih zemljin v srednje in višje parametre Skemptonovega pora in postal pomembnejši za nižje parametre Skempton-

Keywords

Skempton’s parameter, Relative effective diameter, Wet deposition, steady state, sandy soils, triaxial test

Abstract

This experimental investigation describes fundamental research to explore the influence of relative effective diameter on the mechanical behavior, in terms of ultimate shear strength, of partially saturated granular sandy soils. In this context, a series of laboratory triaxial experiments were performed on two groups of sandy soils with three maximum diameters and two different effective diameters under consideration. The sandy-soil samples were reconstituted in the laboratory using the wet deposition method at an initial loose relative density, tested under different Skempton pore-pressure parameters and subjected to a constant confining pressure. The obtained data indicate that the Skempton pore-pressure parameter could be correlated with the ultimate shear strength of the tested materials for medium and higher Skempton pore-pressure parameters and become more significant for lower Skempton
pore-pressure parameters in the case of a higher effective diameter. Moreover, the obtained test results demonstrate clearly that the maximum diameter is a suitable parameter for an assessment of the monotonic undrained shear strength (known as the static liquefaction resistance) and the brittleness index of the wet deposited samples. In addition, the introduced new parameter named as the relative effective diameter appears as an appropriate factor for predicting the partially saturated ultimate shear strength, the brittleness index and the mobilized ultimate internal friction angle of the two groups of sandy soils.

Abbreviations

\[ a, b, c \] = Equation’s coefficients  
\[ A_i, B_i, C_i \] = Sandy-soil samples  
\[ B \] = Skempton’s pore-pressure parameter  
\[ C_u \] = Coefficient of uniformity  
\[ C_c \] = Coefficient of curvature  
\[ D \] = Diameter of sample  
\[ D_{\text{max}} \] = Maximum diameter  
\[ D_{\text{min}} \] = Minimum diameter  
\[ D_{10} \] = Effective grain size  
\[ D_{50} \] = Mean grain size  
\[ D_i \] = Initial relative density  
\[ e_i \] = Initial void ratio  
\[ e_{\text{max}}, e_{\text{min}} \] = Extreme void ratios

\[ F_c \] = Fines content  
\[ G_c \] = Specific gravity  
\[ H \] = Height of the sample  
\[ H/D \] = Height-to-diameter ratio of the sample  
\[ I_B \] = Brittleness index  
\[ P_c' \] = Initial confining pressure  
\[ q \] = Deviator stress  
\[ q_{\text{peak}} \] = Undrained peak shear strength  
\[ q_{ss} \] = Steady-state shear strength  
\[ q_u \] = Ultimate shear strength  
\[ R^2 \] = Coefficient of determination  
\[ \text{RED} \] = Relative effective diameter  
\[ \text{USCS} \] = Unified Soil Classification System  
\[ \omega \] = Water content  
\[ \varphi_u \] = Mobilized ultimate internal friction angle

1 INTRODUCTION

The liquefaction of granular sandy soils is one of the most interesting, complex and controversial topics in the geotechnical earthquake engineering field. This phenomenon is characterized by a decrease in the undrained shear strength of saturated sandy soils due to a rapid buildup of the excess pore-water pressure within a short time under monotonic and cyclic loading [1, 2 and 3]. Several investigations into this phenomenon were initiated after the two earthquakes that occurred in Nigita and Alaska in 1964. Many investigations were reported in the published literature regarding the influence of some factors on the liquefaction phenomenon of sandy soils, such as sample preparation, sample size, grain size and shape, grading characteristics, confining pressure, stress history, pre-shearing and loading conditions [3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17 and 18]. However, the impacts of other variables such as the degree of saturation, particle size and shape, etc. are incomplete and require further investigations.

Moreover, the influence of the degree of saturation has been subjected to extensive research in the published literature. Indeed, many researchers have indicated that the liquefaction resistance of sandy soils is strongly affected by the degree of saturation which is expressed in terms of Skempton’s pore-pressure parameter (\( B \)) [19, 20, 21, 22, 23, 24, 25 and 26]. Several laboratory tests have shown that the undrained shear strength of sandy soils increased when the degree of saturation decreased [4, 26, 27, 28 and 29]. In [26] the authors found that the liquefaction resistance increased with a decrease in \( B \) value, although it was near zero. Indeed, they showed that with a Skempton’s coefficient \( B \geq 80 \% \), it was good enough to apply three cycles to obtain the sample liquefaction; however, eight cycles were needed to reach soil liquefaction of the samples having a coefficient \( B \) close to 50 %. Reference [10] indicated that an increase in Skempton’s pore-pressure coefficient \( B \) reduced the soil dilatancy and consequently amplified the phase of contractancy. The authors of [30] reported that an increase of Skempton’s coefficient (\( B \)) from 89% to 95% induced a decrease of the initial stiffness of sandy soil and its shear strength and consequently an increase of the contractancy phase leading to a significant increase in the excess pore-water pressure of the tested granular sandy soils. Moreover, they indicated that for lower...
loading amplitudes (CSR≤0.40) for the undrained cyclic tests, the number of cycles required to cause liquefaction increased appreciably with the decrease of the Skempton’s coefficient (B) of Chlef sandy soils.

On other hand, the influence of particle size distribution is an important subject when assessing the undrained shear strength (liquefaction resistance) response of soils [3, 5, 6, 7, 8, 15, 31, 32, 33, 34, 35 and 36]. In addition, [5] reported that the undrained shear strength of Chlef sand could be correlated with the mean grain size and coefficient (D₅₀ and Cₐ). Reference [6] indicated that the grain size distribution in terms of effective diameter (Dₑ), mean grain size (D₅₀), coefficient of uniformity (Cᵤ), effective size ratio (ESR), mean grain size ratio (MGSR) and coefficient of uniformity ratio (CᵤGR) had significant influences on the excess pore-water pressure of silty sand soils. The authors of [34] showed that for samples with the same relative density, the undrained shear strength and the phase transformation deviatoric stress gradually decreases with the increase of the coefficient of uniformity Cᵤ. The authors of [37] observed that the liquefaction resistance of clean sand decreases with a decrease of D₅₀sand and Cᵤsand with the same relative density for the loose samples; however, the undrained shear strength of silty sand soils decreases with an increase of the coefficient of uniformity (Cᵤ). In [7 and 8] it was reported that the gradation and particle shape have a significant influence on the undrained shear strength (liquefaction resistance) of two silty sand soils. Moreover, their test results confirm the existence of simple correlations between the liquefaction resistance and the different grading characteristics (Dₑ, D₅₀, Dₙ₀, Cᵤ, D₁₀R, D₅₀GR and CᵤGR) of the tested soils. The authors of [8] suggested that the instability stress and steady-state ratios can be correlated to the grading characteristics (Dₑ, D₅₀, Dₙ₀, Cᵤ, D₁₀R, D₅₀GR and CᵤGR). Indeed, they decrease in a logarithmic and a linear manner with the decrease of grain size (Dₑ, D₅₀, Dₙ₀) and an increase of fines content, respectively. However, they decrease logarithmically with an increase of the coefficient of uniformity for the different graded sand-silt mixtures. It was reported in [3] that the grain size distribution in terms of extreme diameters (maximum diameter “Dₑmax” and minimum diameter “Dₑmin”) and the mean grain size (D₅₀) had appropriate effects on the liquefaction resistance of the wet deposited sandy samples reconstituted in the laboratory with an initial relative density (Dₑ=25 %). However, in the published literature, previous studies have not reported the influence of the grain-size distribution on the shear strength of partially saturated sandy soils under consideration.

For this purpose the present study is undertaken to evaluate the influences of a newly proposed grain size distri-
soils [12, 13, 18 and 41]. In this study all the tested sandy-soil samples were prepared by the wet-deposition method with a constant water content (ω=5 %) and then placed in a cylindrical mold with a diameter of D=100 mm and a height of H=200 mm “H/D=2” in successive layers with a constant thickness of 20 mm for each layer (10 layers). A constant number of strokes were applied with a flat tamper to obtain a homogeneous and isotropic soil fabric. Then, the samples were placed in the classic monotonic triaxial compression. After that, the sandy-soil samples were purged by passing carbon dioxide (CO$_2$) for different times (15 min for B=20 %, 25 min for B=50 % and 35 min for B=90 %). In addition, the samples were also saturated with de-aerated and demineralized water. In this experimental investigation, a back pressure of 200 kPa was applied for all the performed tests and the sandy-soil samples were subjected to a constant effective stress of P’$_c$=100 kPa. All the undrained monotonic triaxial tests were carried out at a constant strain rate of 0.225 mm per minute, which was slow enough to allow the pore-pressure change to equalize throughout the sample with the pore pressure measured at the base of the sample.

2.3 Relationship between the void ratio index and the relative effective diameter

For the purpose of evaluating the relationship between the extreme void ratio index in terms of maximum void ratio (ε$_{max}$) and minimum void ratio (ε$_{min}$) with the proposed grain size ratio named as the relative effective diameter (RED=D$_{10}$/D$_{max}$) for the two groups of sandy soils named as A$_1$, B$_1$ and C$_1$ with the maximum diameter 1 mm ≤ D$_{max}$ ≤ 4 mm and effective diameter D$_{10}$=0.25 mm for group 1 and A$_2$, B$_2$ and C$_2$ with the maximum diameter 1 mm ≤ D$_{max}$ ≤ 4 mm and the effective diameter D$_{10}$=0.08 mm for group 2 under study. It is clear from Figure 2 that the extreme void ratios (ε$_{max}$ and ε$_{min}$) display a logarithmic relationship with the relative effective diameter ($R^2=0.99$) for all the tested materials under consideration. Indeed, the maximum and minimum void ratios (ε$_{max}$ and ε$_{min}$) increase with the increase of the relative effective diameter of the selected sandy soils for the two values of the effective diameter (D$_{10}$=0.25 mm and D$_{10}$=0.08 mm) and the maximum diameter range (1 mm ≤ D$_{max}$ ≤ 4 mm).

<table>
<thead>
<tr>
<th>Sample</th>
<th>$G_s$</th>
<th>D$_{max}$(mm)</th>
<th>D$_{min}$(mm)</th>
<th>D$_{10}$(mm)</th>
<th>$C_u$</th>
<th>$C_c$</th>
<th>$e_{max}$</th>
<th>$e_{min}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A$_1$</td>
<td>2.674</td>
<td>4</td>
<td>0.25</td>
<td>2.080</td>
<td>1.053</td>
<td>0.840</td>
<td>0.635</td>
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</tr>
<tr>
<td>B$_1$</td>
<td>2.675</td>
<td>2</td>
<td>0.25</td>
<td>1.960</td>
<td>1.058</td>
<td>0.880</td>
<td>0.664</td>
<td></td>
</tr>
<tr>
<td>C$_1$</td>
<td>2.675</td>
<td>1</td>
<td>0.25</td>
<td>1.920</td>
<td>0.963</td>
<td>0.889</td>
<td>0.666</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample</th>
<th>$G_s$</th>
<th>D$_{max}$(mm)</th>
<th>D$_{min}$(mm)</th>
<th>D$_{10}$(mm)</th>
<th>$C_u$</th>
<th>$C_c$</th>
<th>$e_{max}$</th>
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</tr>
</thead>
<tbody>
<tr>
<td>A$_2$</td>
<td>2.683</td>
<td>4</td>
<td>0.08</td>
<td>5.750</td>
<td>2.446</td>
<td>0.772</td>
<td>0.527</td>
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</tr>
<tr>
<td>B$_2$</td>
<td>2.684</td>
<td>2</td>
<td>0.08</td>
<td>5.875</td>
<td>2.237</td>
<td>0.804</td>
<td>0.552</td>
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</tr>
<tr>
<td>C$_2$</td>
<td>2.679</td>
<td>1</td>
<td>0.08</td>
<td>5.625</td>
<td>2.336</td>
<td>0.817</td>
<td>0.555</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Index properties of tested materials

Figure 1. Grain size distribution curves of tested materials. (a) Group 1 (D$_{10}$=0.25 mm), (b) Group 2 (D$_{10}$=0.08 mm).
3 RESULTS AND DISCUSSION

The result of the undrained monotonic compression triaxial tests performed on two groups and each group reconstituted of three sandy soil samples named as $A_1$, $B_1$ and $C_1$ with the same effective diameter ($D_{10}=0.25$ mm) and three values of the maximum diameter of $D_{\text{max}}=4$ mm for $A_1$, $D_{\text{max}}=2$ mm for $B_1$ and $D_{\text{max}}=1$ mm for $C_1$ for group 1 and three sandy soils termed as $A_2$, $B_2$ and $C_2$ reconstituted with an effective diameter of $D_{10}=0.08$ mm with three different maximum diameter values ($D_{\text{max}}=4$ mm, 2 mm and 1 mm), respectively, for group 2. All the samples were reconstituted with an initial relative density of $D_r=25$ % and subjected to three different values of Skempton's pore-pressure parameter ($B=20$, 50 and 90 %) and a constant confining pressure of $P'_c=100$ kPa.

3.1 Undrained monotonic triaxial compression test results

3.1.1 Group 1 ($A_1$, $B_1$ and $C_1$ with $D_{10}=0.25$ mm)

Figures 3, 4 and 5 illustrate the undrained monotonic triaxial compression tests performed on three laboratory-reconstituted sandy soil samples named $A_1$, $B_1$ and $C_1$ with the same effective diameter ($D_{10}=0.25$ mm). It can be observed from these figures that completed and limited static liquefaction cases were recorded for the different tested samples under different Skempton pore-pressure parameters ($B$) with a clear impact of the maximum diameter $D_{\text{max}}$ on the undrained shear strength (static liquefaction resistance) response. Moreover, it is clear that the undrained shear strength (liquefaction resistance) of the samples $A_1$, $B_1$ and $C_1$ increases with the decrease of
Figure 4. Undrained monotonic behavior of Chlef sandy soils (B1) ($D_{\text{max}}=2 \text{ mm}, D_{\text{min}}=0.0016 \text{ mm}, D_{r}=25 \%, P'_c=100 \text{ kPa}$). (a) Deviator stress versus axial strain. (b) Deviator stress versus Effective mean Stress.

Figure 5. Undrained monotonic behavior of Chlef sandy soils (C1) ($D_{\text{max}}=1 \text{ mm}, D_{\text{min}}=0.0016 \text{ mm}, D_{r}=25 \%, P'_c=100 \text{ kPa}$). (a) Deviator stress versus axial strain. (b) Deviator stress versus Effective mean Stress.

Table 2. Summary of undrained monotonic triaxial tests for group 1.

<table>
<thead>
<tr>
<th>Characteristics of materials</th>
<th>A1</th>
<th>B1</th>
<th>C1</th>
</tr>
</thead>
<tbody>
<tr>
<td>B (%)</td>
<td>20</td>
<td>50</td>
<td>90</td>
</tr>
<tr>
<td>$q_u$ (kPa)</td>
<td>117.85</td>
<td>21.83</td>
<td>17.67</td>
</tr>
<tr>
<td>RED (-)</td>
<td>0.0625</td>
<td>0.0625</td>
<td>0.0625</td>
</tr>
<tr>
<td>$\varphi_u$ (°)</td>
<td>34.48</td>
<td>57.62</td>
<td>57.71</td>
</tr>
<tr>
<td>$I_B$ (-)</td>
<td>0.17</td>
<td>0.71</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Soil response: All the samples exhibit flow behavior.

Skempton’s pore-pressure parameter in the range 50–90 % and becomes more obvious at a lower Skempton’s pore-pressure parameter B=20 %. In addition, the obtained data indicate that the percentage of increasing in the shear strength is 0.09 %, 22 % and 20 % for the range of Skempton’s pore-pressure parameter (B=50–90 %) and becomes very significant (87 %, 55 % and 53 %) in the range of Skempton’s pore-pressure parameter (B=20–50 %) for the
tested sandy-soil samples $A_1$, $B_1$ and $C_1$ respectively. This sandy-soils trend can be attributed to the increase in the particle interlocking between the coarse grains of sandy soils due to the decrease of Skempton's pore-pressure parameter inducing a contractive character of the tested samples. Therefore, the higher peak undrained shear strength corresponds to the maximum diameter ($D_{\text{max}}=4 \text{ mm}$) and the lower Skempton's pore-pressure parameter ($B=20 \%$). In contrast, the inverse tendency is observed for the higher Skempton's pore-pressure parameters ($B=50 \%$ and $B=90 \%$) where they exhibit the lower peak undrained shear strength with the increasing of the maximum diameter from $D_{\text{max}}=1 \text{ mm}$ to $D_{\text{max}}=4 \text{ mm}$ for the tested materials. In addition, all the samples $A_1$, $B_1$ and $C_1$ indicate that the ultimate shear strength is reached within 10–20 % axial strain (Figures 3a, 4a and 5a). Our findings are in good agreement with the observations of [4, 10 and 24]. The stress path in the ($p', q$) plane clearly shows the role of the Skempton's pore-pressure parameter and the maximum diameter ($B$ and $D_{\text{max}}$) in the decrease of the average effective stress and the maximum deviatoric stress (Figures 3b, 4b and 5b). A summary of the undrained monotonic triaxial tests results for group 1 are summarized in Table 2.

3.1.2 Group 2 ($A_2$, $B_2$ and $C_2$ with $D_{10}=0.08 \text{ mm}$)

Figures 6, 7 and 8 show the undrained-shear-strength response of three granular sandy soils termed as $A_2$, $B_2$ and $C_2$, reconstituted with an effective diameter of $D_{10}=0.08 \text{ mm}$. The test results demonstrate clearly that the completed and limited static liquefaction cases were recorded for the different tested samples under different values of the Skempton's pore-pressure parameter ($B=20 \%, 50 \%$ and $90 \%$) with a clear impact of

![Figure 6](image6.png)  
**Figure 6.** Undrained monotonic behavior of Chlef sandy soils ($A_2$) ($D_{\text{max}}=4 \text{ mm}$, $D_{\text{min}}=0.0016 \text{ mm}$, $D_{10}=0.08 \text{ mm}$, $D_r=25 \%$, $P'_c=100 \text{ kPa}$). (a) Deviator stress versus axial strain. (b) Deviator stress versus Effective mean Stress.

![Figure 7](image7.png)  
**Figure 7.** Undrained monotonic behavior of Chlef sandy soils ($B_2$) ($D_{\text{max}}=2 \text{ mm}$, $D_{\text{min}}=0.0016 \text{ mm}$, $D_{10}=0.08 \text{ mm}$, $D_r=25 \%$, $P'_c=100 \text{ kPa}$). (a) Deviator stress versus axial strain. (b) Deviator stress versus Effective mean Stress.
the maximum diameter $D_{\text{max}}$ on the undrained shear strength behavior.

Moreover, the undrained shear strength of the samples $A_2$, $B_2$ and $C_2$ decreases with the increase of the Skempton's pore-pressure parameter. In addition, for the lower Skempton's pore-pressure parameter ($B=20\%$), the higher maximum diameter ($D_{\text{max}}=4\text{ mm}$) exhibits the lower peak undrained shear strength. The inverse tendency is observed for the higher values of the Skempton's pore-pressure parameter ($B=50\%$ and $B=90\%$). Therefore, the obtained data indicate that the ultimate shear strength is reached within 4–20\% axial strain for all the sandy-soil samples $A_2$, $B_2$ and $C_2$, see Figures 6a, 7a and 8a, respectively. The obtained results are in good agreement with the results of [4, 10 and 24]. The stress path in the $(p', q)$ plane shows clearly the role of the Skempton's pore-pressure parameter ($B$) and the maximum diameter ($D_{\text{max}}$) in decreasing the average effective stress and the maximum deviatoric stress (Figures 6b, 7b and 8b). A summary of the undrained monotonic triaxial tests results for group 2 are summarized in Table 3.

3.2 Effect of Skempton's pore pressure on the undrained ultimate shear strength

Figure 9 summarizes the effect of Skempton's pore-pressure parameter ($B=20\%$, $B=50\%$ and $B=90\%$) on the undrained ultimate shear strength of six sandy-soil samples ($A_1$, $B_1$, $C_1$, $A_2$, $B_2$ and $C_2$). It is clear from Figure 9 that the decrease of the Skempton's pore-pressure parameter leads to a remarkable increase of the undrained ultimate shear strength and becomes more significant for the lower Skempton's pore-water pressure ($B=20\%$) of the tested sandy-soil samples. The obtained data indicate that the sandy-soil samples with the higher maximum diameter ($D_{\text{max}}=4\text{ mm}$) and the lower Skempton's pore-pressure parameter ($B=20\%$) exhibit a higher undrained ultimate shear strength compared to that induced by higher Skempton's pore-pressure parameters ($B=50\%$ and $90\%$) for the tested samples of group 1.

In contrast, the inverse tendency was observed in the case of group 2, where the sandy-soil samples with the higher maximum diameter ($D_{\text{max}}=4\text{ mm}$) exhibited a lower undrained ultimate shear strength for the same Skempton's pore-pressure parameter ($B=20\%$).

### Table 3. Summary of undrained monotonic triaxial tests for group 2.

<table>
<thead>
<tr>
<th>Characteristics of materials</th>
<th>$A_2$</th>
<th>$B_2$</th>
<th>$C_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B$ (%)</td>
<td>20</td>
<td>50</td>
<td>90</td>
</tr>
<tr>
<td>20</td>
<td>50</td>
<td>90</td>
<td></td>
</tr>
<tr>
<td>$q_u$ (kPa)</td>
<td>27.57</td>
<td>24.46</td>
<td>24.58</td>
</tr>
<tr>
<td>24.33</td>
<td>23.95</td>
<td>56.92</td>
<td></td>
</tr>
<tr>
<td>RED (-)</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>0.04</td>
<td>0.04</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>$\phi_u$ (°)</td>
<td>49.65</td>
<td>58.58</td>
<td>57.71</td>
</tr>
<tr>
<td>42.78</td>
<td>57.69</td>
<td>37.99</td>
<td></td>
</tr>
<tr>
<td>$I_B$ (-)</td>
<td>0.71</td>
<td>0.69</td>
<td>0.63</td>
</tr>
<tr>
<td>0.62</td>
<td>0.71</td>
<td>0.46</td>
<td></td>
</tr>
<tr>
<td>Soil response</td>
<td>All the samples exhibit flow behavior</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
compared to the higher Skempton's pore-pressure parameters ($B=50\%$ and $90\%$). In addition, the ultimate shear strength of the sandy-soil samples of group 1 is more significant than that of group 2. This behavior indicates that the increasing in the effective diameter leads to a significant increase in the undrained ultimate shear strength of the tested sandy soils under consideration. Moreover, the sandy-soil samples of group 2 with an effective diameter $D_{10}=0.08$ mm demonstrate clearly that the presence of the low plastic fines between the coarse grains of $A_2$, $B_2$ and $C_2$ makes the sandy soils more compressible, leading to a reduction of the interparticle forces and consequently to a decrease in the undrained ultimate shear strength of the used materials.

3.3 Effect of the maximum diameter on the undrained ultimate shear strength

The influence of maximum diameter ($D_{max}$) on the undrained ultimate shear strength ($q_u$) of the two groups of sandy soils is illustrated in Figure (10). It is clear from the bar chart that the maximum diameter ($D_{max}$) exhibits a significant influence on the undrained ultimate shear strength of the tested materials. Indeed, the undrained ultimate shear strength increases with the increase of the maximum diameter for the lower value of the Skempton's pore pressure ($B=20\%$) in the case of the group 1 samples, while the inverse tendency was observed for the sandy-soil samples of group 2, where the undrained ultimate shear strength decreases with the increase of the maximum diameter for the same Skempton's coefficient ($B=20\%$). The observed tendency can be attributed to the combined effects of the maximum and effective diameters with ($B=20\%$) in the increasing and decreasing of the interparticle forces between the coarse grains for group 1 ($D_{10}=0.25$ mm), the small amount of low plastic fines $F_c\leq3\%$) and group 2 ($D_{10}=0.08$ mm), the presence of low plastic fines $F_c=10\%$ leading to a decrease of the interlocking of the coarse particles and consequently to a significant decrease in the undrained shear strength. In contrast, quite similar observations were recorded for the ultimate shear strength in the case of the intermediate and higher Skempton's coefficients ($B=50\%$ and $90\%$) for all the tested sandy-soil samples.

3.4 Effect of the relative effective diameter on the undrained ultimate shear strength

The effect of the relative effective diameter ($RED=D_{10}/D_{max}$) on the undrained ultimate shear strength ($q_u$) of the two groups of sandy soils is shown in Figure (11). The obtained data from the current study indicate that the relative effective diameter ($RED$) could be correlated...
Figure 11. Undrained ultimate shear strength versus relative effective diameter ($D_{r}=25\%$, $P_{c}'=100$ kPa).
(a) Group 1 ($D_{10}=0.25$ mm). (b) Group 2 ($D_{10}=0.08$ mm).

3.5 Effect of the maximum diameter and Skempton’s pore-pressure parameter on the brittleness index of the sandy soils

To quantify the amount of strain softening during undrained loading, [42] proposed a new parameter to identify this behavior named the brittleness index ($I_B$), which is defined as:

$$I_B = \frac{q_{peak} - q_{ss}}{q_{peak}}$$

where $q_{peak}$ is the peak shear strength (instability point in the plane of the stress path) and $q_{ss}$ is the steady-state shear strength. In which $I_B$ ranges $0 \leq I_B \leq 1$. If $I_B=1$, the soil exhibits a very brittle response associated with lower steady-state shear strength (complete liquefaction). In contrast, if $I_B=0$, the soil occurs in a non-brittle or strain-hardening response (non-flow). Based on these findings, Figure 12 presents the impacts of the maximum diameter and Skempton’s pore-pressure parameter on the brittleness index ($I_B$) of the two types of sandy soils. In general, the obtained data indicate that the brittleness index ($I_B$) affects significantly the mechanical behavior of the tested sandy soils, considering the effects of the maximum diameter ($D_{max}$) and the Skempton’s pore-pressure parameter ($B$). Indeed, it is clear from the bar chart (Figure12a) that the increase of maximum diameter leads to a remarkable decreasing of the brittleness index for the lower Skempton's pore-pressure parameter ($B=20\%$) of the tested materials. The inverse tendency was observed for the sandy soils of group 2 considering the same Skempton’s pore-pressure parameter ($B=20\%$) (Figure12b). This variation is more significantly affected for group 2, compared to group 1. However, similar brittleness index ($I_B$) values were obtained for intermediate and higher Skempton’s coefficients ($B=50\%$ and $90\%$) for all the tested sandy-soil samples. The results are summarized in Tables 2 and 3 for group 1 and group 2, respectively.
Figure 12. Brittleness index versus maximum diameter of sandy soils \((D_\text{r}=25 \% , P'_c=100 \text{kPa})\).
(a) Group 1 \((D_{10}=0.25 \text{ mm})\). (b) Group 2 \((D_{10}=0.08 \text{ mm})\).

Figure 13. Brittleness index versus relative effective diameter of sandy soils \((D_\text{r}=25 \% , P'_c=100 \text{kPa})\).
(a) Group 1 \((D_{10}=0.25 \text{ mm})\). (b) Group 2 \((D_{10}=0.08 \text{ mm})\).

3.6 Relationship between the brittleness index and the relative effective diameter

The relationship between the brittleness index \((I_B)\) and the relative effective diameter \((RED)\) of six sandy-soil samples \((A_1, B_1 \text{ and } C_1 \text{ for group } 1 \text{ and } A_2, B_2 \text{ and } C_2 \text{ for group } 2)\) is discussed in this section. The test results show that the brittleness index could be correlated with the relative effective diameter of the tested materials for all the parameters under consideration. In addition, it is clear from Figure 13a that the brittleness index \((I_B)\) increases with an increase of the relative effective diameter \((RED)\) for the lower Skempton's pore-pressure parameter \((B=20 \% )\) of the tested sandy-soil samples of group 1. However, the inverse tendency was observed for the intermediate and higher Skempton's pore-pressure parameter \((B=50 \% \text{ and } 90 \% )\), where the brittleness index \((I_B)\) decreases with the increase of the relative effective diameter \((RED)\) of the tested materials \((A_1, B_1 \text{ and } C_1)\) under study. Moreover, it is clear from Figure 13b that the brittleness index decreases with an increase of the relative effective diameter \((RED)\) of the tested materials under a lower Skempton's pore-pressure parameter \((B=20 \% )\). In contrast, the inverse tendency was shown for two other Skempton pore-pressures parameters \((B=50 \% \text{ and } 90 \% )\), where the brittleness index increases with an increase of the relative effective diameter for the sandy-soil samples \((A_2, B_2 \text{ and } C_2)\).
3.7 Relationship between Skempton's pore-pressure parameter and the mobilized ultimate internal friction angle of sandy soils

The correlation between the Skempton's pore-pressure parameter ($B$) and the mobilized ultimate internal friction angle ($\phi_u$) for the two groups is presented in Figure (14). The obtained data indicate that the Skempton's pore-pressure parameter could be correlated with the mobilized ultimate internal friction angle of the tested materials. Indeed, the mobilized ultimate internal friction angle increases in a good polynomial manner ($R^2=0.83$) with an increase of the Skempton's pore-pressure parameter from $B=20\%$ to $B=90\%$ of the $A_1$, $B_1$ and $C_1$ for group 1 and $A_2$, $B_2$ and $C_2$ for group 2. Moreover, it is clear from Figure 14 that the higher Skempton's pore-pressure parameter ($B=90\%$) indicates a similar undrained mobilized ultimate internal friction angle for all the used materials compared to the lower Skempton's pore-pressure parameter ($B=20\%$ and $50\%$), where the tendency shows a higher range between the values of the mobilized ultimate internal friction angle of the tested materials under study. This trend confirms that the Skempton's pore-pressure parameter has a significant influence on the undrained shear strength and consequently on the undrained mobilized ultimate internal friction angle of the sandy soils, where it has a remarkable effect on increasing the interparticle forces between the coarse and fine grains, leading to an important increase in the mobilized ultimate internal friction angle of the tested sandy soils.

![Figure 14](image1.png)

**Figure 14.** Mobilized ultimate internal friction angle versus Skempton's pore-pressure parameter of sandy soils ($D_r=25\%$, $P'_c=100\,kPa$).

3.8 Relationship between the mobilized ultimate internal friction angle and the relative effective diameter of the sandy soils

The variation of the mobilized ultimate internal friction angle ($\phi_u$) and the relative effective diameter (RED) of the two groups of sandy-soil samples is presented in Figure 15. The obtained data indicate that the relative

![Figure 15](image2.png)

**Figure 15.** Mobilized ultimate internal friction angle versus relative effective diameter of sandy-soil samples ($D_r=25\%$, $P'_c=100\,kPa$). (a) $B=20\%$, (b) $B=20\%$, (c) $B=90\%$. 
effective diameter \((\text{RED})\) could be correlated with the mobilized ultimate internal friction angle \((\varphi_{mu})\) for the materials under study. Indeed, the mobilized ultimate internal friction angle decreases in a polynomial manner with an increase of the relative effective diameter for the lower and intermediate Skempton’s pore-pressure parameters \((B=20\%\) and 50\%). However, the influence of the relative effective diameter on the mobilized ultimate internal friction angle is insignificant for the higher Skempton’s pore-pressure parameter \((B=90\%)\). The obtained sandy-soils tendency confirms that the decrement of the Skempton’s pore-pressure parameter plays a major role in increasing the mobilized ultimate internal friction angle-relative effective diameter response leading to a significant increase in the interparticle forces between the coarse grains of the tested sandy soils under consideration.

4 CONCLUSION

This laboratory research work is based on a series of undrained compression tests using static triaxial apparatus for the purpose of evaluating the effects of the relative effective diameter \((\text{RED}=D_{10}/D_{max})\) on the mechanical behavior of partially saturated sandy soils. The tested samples were subdivided into two groups: \(A_1, B_1, C_1\) and \(A_2, B_2, C_2\). They were reconstituted with the wet-deposition method \((\omega=5\%)\) at an initial relative density \((D_r=25\%)\), examined under three different Skempton’s pore-pressure parameter values \((B=20\%, 50\%\) and 90\%) and subjected to a constant confining pressure \((P_c^{\prime}=100\text{kPa})\). The main conclusions of this study are summarized below:

1. The obtained test results show that the Skempton’s pore-water-pressure parameter has a significant influence on the undrained-shear-strength response of the wet-deposited sandy-soil samples. Indeed, the ultimate shear strength increases with the decrease of the Skempton’s pore-pressure parameter from the higher value \((B=90\%)\) to the lower value \((B=20\%)\), and it becomes more significant for the effective diameter \((D_{10}=0.25\text{mm})\). Completed and limited static liquefaction response cases were observed for all the tested partially saturated sandy-soil samples, as illustrated in Figures 3b, 4b, 5b, 6b, 7b and 8b.

2. The test results demonstrate clearly that the maximum diameter could be correlated with the ultimate shear strength and the brittleness index of the used materials. Indeed, the increase of the maximum diameter leads to an increase of the ultimate shear strength and a decrease of the brittleness index of the samples of group 1 \((A_1, B_1\) and \(C_1)\) for \((B=20\%)\) and the inverse tendency was observed for group 2, where the ultimate shear strength decreases and the brittleness index increases with an increase of the maximum diameter for the same Skempton’s coefficient \((B=20\%)\). In contrast, similar observations were made for the ultimate shear strength in the case of medium and higher Skempton’s coefficients \((B=50\%\) and 90\%) for all the tested sandy-soil samples, as shown in Figure (12 and 13).

3. The obtained findings confirm that the relative effective diameter \((\text{RED})\) has a significant influence on the mechanical behavior of sandy soils in terms of the ultimate shear strength \((\varphi_{mu})\). The increase of the relative effective diameter leads to a decrease of the ultimate shear strength for the lower Skempton’s pore-pressure parameter \((B=20\%)\) of the tested sandy-soil samples of group 1 and increase it for the intermediate \((B=50\%)\) and higher \((B=90\%)\) Skempton’s pore-pressure parameter for the same group and consequently to an increase of the undrained shear strength (liquefaction resistance). Moreover, the inverse trend was observed in the case of group 2 for all the tested Skempton’s pore-pressure-parameter values \((B=20\%, 50\%\) and 90\%), as illustrated in Figures 10 and 11.

4. Finally, the relative effective diameter \((\text{RED})\) could be correlated with the brittleness index \((I_b)\) and the mobilized ultimate internal friction angle \((\varphi_{mu})\) of the two groups and control effectively the undrained shear strength of the sandy soils tested under three Skempton’s pore-pressure-parameter values \((B=20\%, 50\%\) and 90\%) at an initial relative density \((D_r=25\%)\) and subjected to a constant confining pressure \((P_c^{\prime}=100\text{kPa})\), as presented in Figure (15).

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