CHAPTER 3

Laboratory Investigation of Sand Strength Degradation

I  INTRODUCTION

During vibratory pile driving in a saturated cohesionless soil, the cyclic loading applied by the vibrator induces liquefaction of the soil around the pile. The result is a reduction of the shear resistance and the pile penetration under the action of its own weight. However, to model the pile and soil behaviour during the vibratory driving, there is a need to investigate more in detail the soil degradation and the transition towards a liquefied soil. It is well known that the soil degradation is strongly influenced by the shear strain amplitude, the relative density and the initial stress state. However, what is the respective influence of each parameter? What is their influence on the soil degradation? How does the soil behave under large cyclic strain amplitudes? What is the influence of the dilative behaviour?

In that context, the present chapter presents the results of the experimental investigation of sand degradation during a cyclic deformation. Two types of test were used: the cyclic triaxial test and the cyclic direct simple shear test. These tests were chosen because the soil condition during these tests is probably the most representative of the soil condition around the pile during the vibratory pile driving. Indeed, the triaxial condition is comparable with the soil condition at the pile base and the simple shearing simulates the soil around the pile shaft.

The test were strain controlled because the objective of the research is to describe the evolution of the soil degradation. During a strain controlled test, the degradation progresses regularly whereas during a stress controlled test, the liquefaction occurs suddenly and the analysis of degradation is more difficult.
Furthermore, during a vibratory driving, even if the vibrator applies a cyclic load on the pile, it can be considered that, due to the high number of cycles regarding to the penetration speed, the vibrations induced to the pile apply on the surrounding soil a cyclic displacement instead of a cyclic load.

The tests were performed on medium and dense Brusselian sand with a range of strains amplitudes from 0.1% to 9%. One of the purpose is to investigate the cyclic soil behaviour in the similar range of large shear strains that are observed during vibratory pile driving.

The chapter is organised in three paragraphs. The first section introduces the Brusselian sand and the preliminary tests that were performed in order to determine the characteristics of this sand. The second section presents the experimental setup and the tests results of the cyclic triaxial test. The third section summarises the experimental setup and the tests results of the cyclic direct simple shear tests.

II BRUSSELIAN SAND DESCRIPTION

II.1 Geology of the Brusselian sand

Between the Asse Clay and the Ypresien clay layers, the layer of Brusselian sand outcrops in the area of Brussels (Belgium) and is largely present in all the Walloon Brabant province (Fig. 3-1) where the sand is extracted for its use in the construction industry. This sand results from a deposit during the Eocene age in a seacoast area of probably a tropical sea. Some fossils give evidence of this assumption. The layer is about 50 meters thick and presents a large panel of different sands with different grain size distribution, grain shape and carbonate content. The layer is more or less horizontal with a light slope towards the North (Fig. 3-2). The sand is often covered by a layer of silty soil resulting from wind erosion during the last glaciation.

The Brusselian sand used during this investigation was collected in November 1997 in the Mont-Saint-Guibert sand quarry.
II. Brusselian Sand Description

Fig. 3-1: Extension of the Brusselian sand layer and outcropping area (Wouters, 1994)

Fig. 3-2: NNE-SSW general Belgian geological profile (Wouters, 1994)
II.2 Brusselian sand characteristics

II.2.1 Grain size analysis

The grain size curve of the Brusselian sand is drawn on Fig. 3-3. This grain size distribution is characterised by a mean diameter, \( d_{50} \), equals to 0.18mm. The coefficient of uniformity is equal to 2.2. Following the ASTM norm (ASTM D2487), this soil is classified as poorly graded sand. The grain shape was analysed using a profile projector (Fig. 3-4). It appears from that analysis that the Brusselian sand used for the present research is subrounded. The specific weight of the grains is equal to 25.97kN/m³.

The maximum void ratio of the sand, measured following the NBN 589-205 procedure, is equal to 1.18 (\( \gamma_d = 11.98\text{kN/m}^3 \)).

Four standard tests for determination of the maximum index density using vibratory table were performed. The test procedure followed the ASTM norm (ASTM D4253-93). With this method, the minimum void ratio was found equal to 0.63 (\( \gamma_d = 16.1 \text{kN/m}^3 \)). This value is similar to the value of 0.61 determined by De Jaeger (1991) on a similar Brusselian sand.

On another hand, nine modified proctor compacting tests were carried out with water contents from 0 to 16%. The Proctor optimum was reached for a water content of 12% and corresponded to a void ratio equal to 0.52 (\( \gamma_d = 17.0 \text{kN/m}^3 \)). Similar values of void ratio were obtained in a range of water content from 6 to 14% and no grain crushing were detected on the grain size curves measured before and after the compacting tests.

During laboratory works, it is regularly observed that the compacting test using the vibratory table does not provide necessarily the absolute maximum density. For the Brusselian sand, the void ratio determined with the Proctor test was about 17% smaller than with the vibrating table test. Therefore, in the present research, it was decided to consider the void ratio determined by the Proctor test as the minimum void ratio (\( e_{\text{min}} = 0.52 \)) and to use that value to calculate the relative density.

Despite the marine origins of Brusselian sand, a low value of carbonate content was measured: 1.54%. The carbonate content was measured by the mean of a HCl attack (NBN 589-209 Procedure).
II.2.2 Compressibility analysis

Ten oedometer tests were performed on reconstituted Brusselian sand specimens (Fig. 3-5). These tests investigated 5 initial void ratios on duplicate samples from dense soil (Dr = 91%) to loose soil (Dr=8%). The specimens were prepared using the moist tamping method in one layer. The compressibility index C (Eq 3-1) varies from 40 to 60 and the swelling index A (Eq 3-2) varies from 200 to 280.

\[
\frac{A \cdot h}{h} = \frac{1}{C} \cdot \ln \left( \frac{\sigma_{0}'+\Delta \sigma'}{\sigma_{0}'} \right) \text{ during loading}
\]

(Eq. 3-1)
\[
\frac{\Delta h}{h} = \frac{1}{A} \ln \left( \frac{\sigma_0' - \Delta \sigma'}{\sigma_0'} \right) \text{ during unloading or reloading}
\]  

(Eq. 3-2)

where:
- \( C \) is the compression index
- \( A \) is the swelling index
- \( \Delta \sigma' \) is the stress increment
- \( \Delta h \) is the specimen settlement consecutive of the stress increment \( \Delta \sigma' \)
- \( h \) is the specimen height before the application of the new load increment
- \( \sigma_0' \) is the stress before the application of the new load increment

Although the specimens were loaded to a maximum stress of 3MPa, no grain crushing was observed. This is illustrated by Fig. 3-6 where the grain size curves measured on each specimen after the test is compared with the initial grain size curve. Since the maximum stresses used during the cyclic tests presented in the following paragraphs never exceed 500kPa, the grain crushing can be neglected in the analysis of soil degradation.
II.2.3 Shear resistance analysis

Eleven monotonic drained and undrained triaxial tests\textsuperscript{1} were performed in order to determine the friction angle of the Brusselian sand and the position of the critical void ratio line. The specimens were prepared using the moist tamping method combined with the undercompaction procedure that will be described in the next section. The critical state points of each tests are drawn on Fig. 3-7. The friction angle of the Brusselian sand is equal to $33^\circ$ (Fig. 3-8)

A grain size analysis was conducted on the sand after a triaxial test was performed with an initial pressure of 4700kPa. The results are shown on Fig. 3-6. Like for the oedometer test, no grain-crushing was observed during the triaxial tests.

\textsuperscript{1} The test results are available in appendix on the enclosed CD-rom.
Three monotonic direct simple shear tests were performed on remoulded specimens with a void ratio equal to 0.6. The procedure of the specimens preparation (moist tamping method) and the experimental setup will be described in the next section. The test results are presented on Fig. 3-9.

**II.2.4 Shear modulus analysis**

The maximum shear modulus $G_{\text{max}}$ of the Brusselian sand was measured using bender elements mounted in a triaxial cell. This technique (Dyvik, 1985, Brignoli, 1996, Viggiani, 1995) consists of measuring the velocity of an elastic shear wave through the specimen. This elastic shear wave is generated by a piezoelectric transducer placed at one extremity of the specimen and is received at the other end of the specimen by another piezoelectric transducer. A typical result of measurements is shown on Fig. 3-10. Since the shear strains induced by the wave (approximately $10^{-5}$%) stay within the linear elastic domain, the maximum shear modulus can be calculated based on the shear waves velocity, using the theory of elasticity (Eq 3-3).

$$G_{\text{max}} = \rho V_s^2$$  

(Eq. 3-3)

where $G_{\text{max}}$ is the maximum shear modulus
$V_s$ is the shear waves velocity
$\rho$ is the total mass density of the specimen
Fig. 3-11 presents the maximum shear modulus of the Brusselian sand measured in the triaxial cell at different stresses for a relative density of 0.6.

Fig. 3-10: Shear wave velocity measurement using bender elements

Fig. 3-11: Maximum shear modulus of Brusselian sand as a function of the effective mean stress (\(e = 0.6\))
III CYCLIC TRIAXIAL TEST

III.1 Introduction

The following paragraphs present the results of the cyclic triaxial tests. The first paragraphs describe the experimental setup followed during the tests, and different assumptions made during data processing. The next paragraphs describe in details typical results of a cyclic triaxial test, and discuss the influence of the dilation on the shape of the hysteresis loops and on the stress path. After an analysis of the influence of the experimental setup on the test results, the last paragraphs analyse the influence of the shear strain amplitude, the relative density and the consolidation stress on soil degradation during cyclic triaxial test.

III.2 Experimental setup

III.2.1 Description of the triaxial tests apparatus

Cyclic triaxial tests were performed in the Laboratory of Civil Engineering of the Université catholique de Louvain (Belgium) using a Wykeham Farrance triaxial cell mounted within a 50kN load frame (Fig. 3-12). The capabilities of the apparatus were extended to regulate the axial displacement of the specimen head and the lateral pressure of the cell.

The apparatus allows one to control the vertical deformation of a cylindrical soil specimen. This is accomplished by moving the piston vertically, relative to the fixed cell base. During this test, the pressure of the cell can be regulated in order to impose the desired stress path. As a result, no radial strain condition is imposed a such.

The regulation of the vertical deformation and the confining stress is based on two closed loops of regulation. The first loop controls the vertical deformation by adapting the speed of the linear actuator driving the load frame, based on the difference between the measured and desired vertical displacements. The second loop imposes the desired confining stress by regulating a pneumatic regulator of pressure.

The tests are performed on saturated specimens under undrained conditions. The resulting pore pressure is continuously measured.
III.2.2 Sample preparation

The cyclic triaxial tests are performed on compacted Brusselian sand specimens characterized by a height of 200mm and a diameter of 100mm.

Each specimen is prepared using the method of moist tamping in 10 layers following the undercompaction procedure (Ladd, 1978). The procedure takes into account that, when a sand is compacted in layers, the compaction of each succeeding layer can further densify the sand below it. Therefore, each layer is compacted to a lower density than the final desired value by predicting the amount of required undercompaction (Fig. 3-13).

This undercompaction amount in each layer linearly varies from the bottom to the top of the specimen (Eq 3-4). During the tamping, the required height of the top of the specimen for each layer can be calculated with Eq 3-5.

\[ U_n = U_{n1} - \left( \frac{U_{n1} - U_{n}}{n-1} \right)^{n-1} \]  

(Eq. 3-4)
\[ h_{\text{tamping}} = h_{\text{final}} + \Delta h \frac{U_n}{100} \]  

(Eq. 3-5)

where \( U_n \) is the percentage of undercompaction for the layer \( n \)

\( U_{n1} \) is the percentage of undercompaction selected for the first layer

\( U_{nt} \) is the percentage of undercompaction selected for the final layer (usually zero)

\( n \) is the number of layers

\( n_t \) is the number total of layers

\( h_{\text{tamping}} \) is the proposed tamping height of the \( n \) first layers

\( h_{\text{final}} \) is the final desired height of the \( n \) first layers

\( \Delta h \) is the average height of each layer

The percentage of undercompaction of the first layer is generally function of the final relative density desired. To assess this parameter, Van Impe (1981) and Baldi (1988) have proposed the following equations (Eq 3-6)

\[ U_{n1} = (95 - D_r) D_r^{2/3} k_{00} \]  

Baldi (1988)

\[ U_{n1} = 18 \frac{D_r}{5} \]  

Van Impe (1981)  

(Eq. 3-6)

where \( U_{n1} \) is the percentage of undercompaction selected for the first layer

\( D_r \) is the relative density [%]

The comparison of these equation (Fig. 3-14) shows that, in the range of densities between 60 and 100%, the proposed initial percentages are similar. Since the cyclic triaxial tests are performed in the same range of relative densities, these equations are used to determine the initial undercompaction percentage.

In practice, the dry sand is moistened to a water content corresponding to the Proctor optimum (\( w_{\text{m}} = 12\% \)). The weight of soil layer corresponding to the desired
final relative density is taken and dynamically tamped until the specimen height of the corresponding layer equals the height proposed by the undercompaction method. The tamping is made in a cylindrical mould formed by three shells linked together. The tamping uses a piston whose the diameter is equal to the specimen diameter (100mm). In order to avoid to create a weakness at the interface between two layers, the surface of a damped layer is scarified before adding the next layer. After the tamping, the specimen is weighed and place on the base of the triaxial apparatus. The mould is removed by unlinking the different shells of the mould and removing each one carefully. The last operation of the sample preparation is to place the rubber membrane around the specimen and the top platen.

### III.2.3 Saturation and consolidation

After compaction, the air in the specimen is replaced by CO\textsubscript{2} in order to facilitate the specimen saturation. A bottle filled with C0\textsubscript{2} is connected at the base of the specimen and CO\textsubscript{2} is sent in the specimen by increasing the pore pressure to a pressure of 200kPa. During this operation, the cell pressure is regulated in order to maintain continuously a positive difference between the lateral stress and the pore pressure equal to 20kPa. The drainage at the bottom of the specimen is then closed and the drainage at the top is open permitting the air to leave the specimen. During this operation, the pore pressure decreases to the atmospheric pressure. This operation is repeated 3 times.

After the introduction of CO\textsubscript{2}, the specimen is saturated by introducing demineralised and deaired water. Water is first introduced at the bottom of the specimen, permitting the air bubbles to leave the specimen by the top. When no more bubbles are flushing out, the drainage of the top is closed and the pore pressure is increased to a back-pressure equal to 200kPa. The air bubbles may have not flushed out are assumed to be dissolved in the water.

The degree of saturation is measured using the Skempton method. This method consists in increasing lightly the lateral pressure ($\Delta\sigma_3=5$ kPa) keeping the drainage closed and measuring the corresponding augmentation of pore pressure ($\Delta\sigma$). The Skempton coefficient $B$ is defined as the ratio between these 2 measurements:

$$B = \frac{\Delta\sigma}{\Delta\sigma_3}$$  \hspace{1cm} (Eq. 3-7)

Many researchers have investigated the correlation between the Skempton coefficient and the degree of saturation (Black and Lee (1973), Martin et al (1978), Chaney et al (1979)). Table 3-1 summarises the results of these investigations. Based on these results, the required minimum value of the Skempton coefficient was chosen equal to 0.90. This value correspond to a degree of saturation higher than 99.5% i.e. an error on the volume of the specimen less than 0.5%. The Skempton parameters measured during the test are generally equal to 0.94 and 0.96.
After the saturation, the specimen is consolidated under an isotropic stress field, \( P_0' \) (\( K_0 = 1 \)) by increasing slowly the cell pressure to the desired pressure. The volume variations of the specimen are measured based on the volume of water expelled from the specimen.

<table>
<thead>
<tr>
<th>Degree of saturation [%]</th>
<th>Loose sand</th>
<th>Medium sand</th>
<th>Compact sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>98.0</td>
<td>Martin et al</td>
<td>0.53</td>
<td>Martin et al</td>
</tr>
<tr>
<td>98.5</td>
<td>0.60</td>
<td>0.53</td>
<td>Black et al</td>
</tr>
<tr>
<td>99.0</td>
<td>0.69</td>
<td>0.62</td>
<td>0.51</td>
</tr>
<tr>
<td>99.5</td>
<td>0.85</td>
<td>0.80</td>
<td>0.69</td>
</tr>
<tr>
<td>100</td>
<td>1.00</td>
<td>1.00</td>
<td>0.9877</td>
</tr>
</tbody>
</table>

Table 3-1: Relationships between the Skempton coefficient and the degree of saturation

### III.2.4 Cyclic test

After the soil reaches equilibrium, the drainage of the specimen is closed in order to impose undrained condition (i.e. no volume change) and the sample is sheared by a cyclic symmetric variation of the axial strain. The axial strain amplitudes, \( \varepsilon_a \), used in this investigation cover the shear strains from 0.001\% to 6\%. The frequency of this cyclic displacement is 0.005Hz (or a period of 200 seconds/cycle).

The tests are undrained while the pore pressure is continuously measured at the top and the bottom of the specimen.

During the test, the lateral pressure is regulated to keep the total mean stress \( P \) constant (i.e. \( \Delta \sigma_3 = -\Delta \sigma_1 / 2 \)). This kind of stress path was selected because it is the opinion of the author that, among the different possible stress paths, this choice provides probably a more realistic representation of the soil condition at the pile base during vibratory driving.

During the tests, the following measurements are continuously performed (Fig. 3-12):

- The vertical displacement measured by a LVDT transducer fixed on the piston,
- The axial force applied by the piston. This force is measured with a load cell placed inside the triaxial cell in order to avoid the friction of the interface between the piston and the load cell,
- The pore pressure measured at the top and the bottom of the specimen,
- The cell pressure.

After the completion of the test, the water content of the specimen is measured. This measure is compared to the value of the water content deduced from the initial
data of the specimen and the measurement performed during the saturation and consolidation. Generally, a very good agreement between these values is observed.

### III.2.5 Data acquisition system and data processing

**A. Regulation and data acquisition**

A new data acquisition program was developed to expand the capabilities of the triaxial test apparatus to allow cyclic strain controlled testing and the lateral stress regulation. This program was written using LabVIEW software. The program acquires the measurements of the vertical displacement transducer, the vertical load cell and the 3 pressure transducers. It converts the measured voltages to physical values and calculates the corresponding strains and stresses. Based on the measured vertical strain and stresses, the program drives the motor of the load frame to obtain a cyclic axial strain of the specimen and it regulates the cell pressure to maintain the total mean stress constant.

The measurements are taken with a scan rate of 1Hz i.e. 200 measurements by cycle.

**B. Data Processing**

At the end of the consolidation, the volume of the specimen is calculated with Eq 3-8. This equation assumes a completely saturated specimen. This assumption is verified for each test, by measuring the Skempton coefficient before and after cyclic shearing and by comparing the measured water content after the completion of the cyclic test with the water content calculated based on the measurements performed during specimen preparation, saturation and consolidation.

\[
V_c = V_0 + \Delta V_{sat} - V_{air} - \Delta V_{con}
\]  
(Eq. 3-8)

where:
- \( V_c \) is the volume of the specimen just before the beginning of the triaxial test,
- \( V_0 \) is the volume of the specimen after its compaction (calculated based on the diameter and height measured before placing the specimen in the triaxial cell),
- \( \Delta V_{sat} \) is the volume of water introduced in the specimen during the saturation,
- \( V_{air} \) is the volume of air present in the specimen just after its preparation. This volume is calculated based on the initial weight, the initial volume \( V_0 \), the specific weight of grains and the initial water content.
- \( \Delta V_{con} \) is the volume of water expelled during the consolidation.

The height \( h_c \) of the consolidated specimen is calculated based on both the initial height and the axial displacement measured during the saturation and consolidation (Eq 3-9). The diameter of consolidated specimen \( d_c \) is calculated with Eq 3-10 assuming the specimen remains cylindrical.
where $hc$ is the height of the consolidated specimen, $h_0$ is the measured initial height, $\Delta h_{\text{com}}$ is the vertical displacement measured during saturation and consolidation, $d_c$ is the diameter of the consolidated specimen.

During the test, the volume of the specimen is supposed constant due to the undrained condition. The specimen height $h$ and diameter $d$ are calculated assuming that the specimen stays cylindrical during the test (Eq 3-11 and 3-12).

\begin{align*}
h &= h_c + VD \\
d &= \sqrt{\frac{4.V_c}{h_c \pi}}
\end{align*}

where $h$ is the specimen height, $d$ is the specimen diameter, $V_D$ is the measured vertical displacement of the piston.

The stresses and strains are calculated using Eq 3-13 to 3-18. These equations assume the stress and strain distributions are homogenously distributed through the specimen. It is also assumed that the vertical and lateral stress are the principal stresses. The measured axial load and the lateral stress were corrected in order to take into account the membrane resistance. The correction depends of the type of membrane, the membrane thickness and the axial and displacement shear strain.

\begin{align*}
\varepsilon_1 &= \frac{VD}{h_c} \quad >0 \text{ during compression} \\
\varepsilon_3 &= \frac{d_c - d}{d_c} \\
\gamma &= \varepsilon_1 - \varepsilon_3 \\
\sigma_3 &= LP + \Delta LP \\
\sigma_1 &= \sigma_3 + \frac{4.AL + \Delta AL}{d_c^2 \pi} \\
\tau &= \frac{\sigma_1 - \sigma_3}{2}
\end{align*}

where $\varepsilon_1$ is the axial normal strain, $\varepsilon_3$ is the radial normal strain, $\gamma$ is the maximum shear strain, $\sigma_1$ is the axial normal stress, $\sigma_3$ is the radial normal stress, $\tau$ is the maximum shear stress, $d$ is the specimen diameter, $V_D$ is the measured vertical displacement, $LP$ is the measured lateral pressure, $\Delta LP$ is the membrane correction on lateral pressure, $AL$ is the measured axial load, $\Delta AL$ is the membrane correction on axial load, $d_c$ is the diameter of the consolidated specimen.
The pore pressure is supposed to be equal to the mean value between the pore pressures measured at the top and bottom of the specimen. This assumption is not really taxing because the measured pressures are virtually equal: the difference between them never exceeded 0.05%.

\[
u = \frac{u_{\text{top}} + u_{\text{bottom}}}{2}
\]  
(Eq. 3-19)

where \( u \) is the pore pressure,

\( u_{\text{top}} \) is the pore pressure measured at the specimen top,

\( u_{\text{bottom}} \) is the pore pressure measured at the specimen bottom.

The effective stresses are deduced from the total stresses and the pore pressure (Eq 3-20 and Eq 3-21):

\[
\sigma_1' = \sigma_1 - u
\]  
(Eq. 3-20)

\[
\sigma_3' = \sigma_3 - u
\]  
(Eq. 3-21)

where \( \sigma_1' \) is the effective axial normal stress

\( \sigma_3' \) is the effective radial normal stress

The effective and total mean stresses and the deviator are calculated using the Cambridge’s definition (Eq 3-22 to 3-24).

\[
P' = \frac{\sigma_1' + 2\sigma_3'}{3}
\]  
(Eq. 3-22)

\[
P = \frac{\sigma_1 + 2\sigma_3}{3}
\]  
(Eq. 3-23)

\[
q = \sigma_1' - \sigma_3'
\]  
(Eq. 3-24)

where \( P' \) is the effective mean stress

\( P \) is the total mean stress

\( q \) is the deviator

C. Post processing

The degradation of the specimen during the cyclic triaxial test is analysed based on the evolution of the hysteresis loops of the shear stress \( \tau \) and the shear strain \( \gamma \) (Fig. 3-15) and the build-up of pore pressure. The soil condition during each cycle is characterised using four parameters: the secant shear modulus \( G_{s_n} \), the dissipated energy \( W_n \), the accumulated dissipated energy \( W_{\text{tot},n} \) and the degree of liquefaction \( d_{\text{liq},n} \). The secant shear modulus \( G_{s_n} \) represents the slope between the two extrema of the hysteresis loop (Eq 3-25). The dissipated energy \( W_n \) is calculated by integrating the hysteresis loop on one period starting from the zero shear strain during the compression phase of the cycle (Eq 3-26).
Due to the soil degradation, this integration is not performed on a closed loop as shown on Fig. 3-15. Indeed, the shear stress at the end of a cycle is smaller than the value at the beginning of the cycle.

The accumulated energy $W_{tot\_n}$ is the total energy dissipated since the beginning of the cyclic test (Eq 3-27) and is calculated by adding the energy dissipated during each cycle (Eq 3-28). The degree of liquefaction $d_{liq\_n}$ is defined as the ratio between the excess pore pressure at the end of the cycle (i.e. when the axial strain is equal to 0) and the effective mean stress $P'_{c}$ applied during the consolidation.

\[
G_{Sn} = \frac{\tau_{max}^n - \tau_{min}^n}{\gamma_{max}^n - \gamma_{min}^n} \times 100
\]  
(Eq. 3-25)

\[
W_n = \text{surface of the hysteresis } \gamma - \tau
\]  
(Eq. 3-26)

\[
W_{tot\_n} = \sum_{i=1}^{n} W_i
\]  
(Eq. 3-27)

\[
d_{liq\_n} = \frac{\Delta u_{\text{end of n-th cycle}}}{P'_{c}}
\]  
(Eq. 3-28)

where $\tau_{max}^n$ and $\tau_{min}^n$ are the maximum and minimum shear stresses measured during the $n^{th}$ cycle,

$\gamma_{max}^n$ and $\gamma_{min}^n$ are the maximum and minimum shear strains measured during the $n^{th}$ cycle,

$\Delta u_{\text{end of n-th cycle}}$ is the measured excess pore pressure at the end of the $n^{th}$ cycle,

$P'_{c}$ is the effective mean stress applied during the consolidation.

Fig. 3-15: Characterisation of hysteresis loops during a cyclic triaxial test
III.3 Cyclic triaxial tests results

III.3.1 Introduction

During this investigation, 45 cyclic triaxial tests2 were performed on Brusselian sand. These tests analysed the influence of the shear strain amplitude $\gamma_a$, the relative density $D_r$ and the initial effective mean stress $P_0'$ (or total mean stress $P$) on the sand degradation.

The values of parameters investigated cover the following ranges:

- Shear strains ($\gamma_a$): from 0.002% to 9%,
- Relative densities ($D_r$): from 65% to 86%, and
- Effective initial mean stress ($P_0'$): from 50kPa to 200kPa.

Table 3-2 summarises the parameters selected for the different tests that were performed and the number of tests that were performed with the same set of parameters in order to investigate the influence of the experimental setup on the test results.

Table 3-2: Summary of cyclic triaxial tests

*Because there is no degradation for such strain amplitudes, these tests were performed on the same specimen in order to determine the relationship between the strain amplitude and the secant shear modulus.

The first paragraph describes in detail the results of a typical cyclic triaxial test. The next paragraph analyses the influence of the experimental setup on the tests results (repeatability, stress path influence, specimen homogeneity, …). The last paragraph compares the tests results together and proposes some explanations of the soil degradation during cyclic triaxial tests.

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2 A summary of the results of the tests is available in the appendixes on the enclosed CD-rom.
III.3.2 Typical result of a strain controlled triaxial test

A. Introduction

The purpose of the following paragraphs is to describe in details the results of a cyclic triaxial test. The first paragraph will give a general description of the specimen during the preparation. The second paragraph will present the evolution of the different stress and strains as a function of time. The next paragraph will investigate the shape and evolution of the hysteresis loops and of the stress path. Finally, the last paragraph will study the sample degradation as a function of cycle number and of the dissipated energy.

B. General description of the test

After the sample compaction (in 10 layers), the specimen diameter was measured 9 times at different height and in different direction. The mean value was 100.5mm in a range of 0.5mm. Based on 3 measurements, the mean specimen height was equal 198.1mm in a range of 0.2mm. The average weight of the moisten specimen was 2890.0g and the average initial water content was equal to 12.03%.

After the introduction of CO₂, 160cm³ of water entered in the specimen during the saturation phase with air escapement. When no more air bubbles were flushed out, the corresponding drainage was closed and the pore pressure was increased to a value of 200kPa. During this second phase of saturation, 141.5cm³ of additional water entered in the specimen. The Skempton coefficient measured just when the pore pressure of 200kPa was reached was 0.93. Two days later, this coefficient grew to 0.95. During the saturation phase, the lateral stress was kept 20kPa higher than the pore pressure measured at the specimen extremities.

Two days after the beginning of the saturation, the lateral pressure was increased to a value of 400kPa, stressing the specimen by a isotropic effective stress equal to 200kPa. During the consolidation, 16.35cm³ of water were expulsed from the specimen.

Table 3-3 summarises the measurements performed during the sample preparation, saturation and consolidation.

After the consolidation, a cyclic displacement was imposed at the top of the specimen. The amplitude of the displacement was 2mm (corresponding to a vertical deformation ε₁ = 1%) and the frequency was equal to 0.05Hz (i.e. a period of 200sec). 40 cycles were applied on the specimen. After the 15th cycle, a neck appeared close to the top of the specimen.

After the dismounting, the water content of the specimen was equal to 23.03%. This value must be compared with the final water content after consolidation deduced
from the measurements performed during the sample preparation: 23.05% (i.e. an error
of 0.1% with the measured value). This result validates the procedure used to calculate
the specimen parameters just before the cyclic test.

<table>
<thead>
<tr>
<th>Measurements performed after the Specimen preparation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial diameter: 100.56 mm</td>
</tr>
<tr>
<td>Initial height: 198.16 mm</td>
</tr>
<tr>
<td>Initial moisten weight: 2890.0 g</td>
</tr>
<tr>
<td>Initial water content: 12.03 %</td>
</tr>
<tr>
<td>Specific weight of grains: 25.95 kN/m³</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Measurements performed during specimen saturation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water volume entered: 301.5 cm³</td>
</tr>
<tr>
<td>Vertical displacement: -0.12 mm</td>
</tr>
<tr>
<td>Skempton coefficient : 0.95</td>
</tr>
<tr>
<td>Initial pore pressure = 200kPa</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Measurements performed during specimen consolidation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isotropic consolidation (Kₒ=1)</td>
</tr>
<tr>
<td>Water volume expelled: 16.35 cm³</td>
</tr>
<tr>
<td>Vertical displacement: 0.26 mm</td>
</tr>
<tr>
<td>Total consolidation stress P = 400 kPa</td>
</tr>
<tr>
<td>Effective consolidation stress Pₒ’ = 200 kPa</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Summary of the initial specimen characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density γᵣ [kN/m³]</td>
</tr>
<tr>
<td>Initial</td>
</tr>
<tr>
<td>After saturation</td>
</tr>
<tr>
<td>After consolidation</td>
</tr>
</tbody>
</table>

* assumed value

Table 3-3: Measurements performed during a typical specimen preparation, saturation
and consolidation.
C. Soil behaviour as a function of time

Fig. 3-16 shows the evolution of the total radial and axial stresses during a cyclic triaxial test. The axial stress is measured and corresponds to the response of the specimen to the cyclic deformation. The radial stress is imposed in order to maintain the total mean stress \( P = (\sigma_1 + 2\sigma_3)/3 \) equal to 400 kPa. Therefore, the two stresses (\( \sigma_1 \) and \( \sigma_3 \)) are continuously in phase opposition.

The difference between the two total stresses represents the maximum shear stress in the specimen. The evolution of the shear stress during the cycles (Fig. 3-17) that is not regular and not symmetrical in extension (negative value) and in compression (positive value). When the shear strain is maximum, the shear stress is also maximum. During the decrease of the shear strain (corresponding to a extension of the specimen), the shear stress returns rapidly to a value close to zero and stays low during the return of the piston to its initial position. After a while, the decrease of the shear stress becomes more important. The shear stress reaches the minimum value at the shear strain reversal. When the direction of the movement changes from extension to compression, the shear stress behaves like in extension: returns rapidly to zero, stays stable with a low value during a while and increases to the maximum value.

The pore pressure grows progressively from an initial value of 200 kPa to the value of the total mean stress, 400 kPa (Fig. 3-16). When the pore pressure reaches this value, the effective mean stress vanishes. The specimen is liquefied and no more shear resistance is observed. For the test presented here, 17 cycles were needed to obtain this
state of liquefaction\(^3\). Fig. 3-17 shows also that the maximum pore pressure observed since the beginning of the tests grows only when the specimen is deformed in compression. Indeed, during the extension phases, the pore pressure reaches the maximum pore pressure observed during the previous cycle, but never exceeds this value.

The frequency of the pore pressure is twice the frequency of the axial displacement. During each cycle, two maxima and two minima of pore pressure are observed. Each decrease of the pore pressure corresponds to a tendency of the specimen to dilate (i.e. increase its volume). An increase of the pore pressure means the specimen tends to contract (i.e. reduce its volume). During a cycle, two phases of dilation\(^4\) and two phases of contraction\(^1\) are observed. The end of each dilation phase corresponds to the maxima of displacement, whereas the end of each contraction phase is simultaneous to the inflection point in the maximum shear stress curve (Fig. 3-17). The comparison between the shear stress and the pore pressure shows also that the moment where the pore pressure equals the maximum pore pressure observed since the beginning of the test corresponds more or less with the moment where the shear stress vanishes.

---

\(^3\) As explained in Chapter 2, in the following text, the term liquefaction is synonym of cyclic mobility

\(^4\) Since the specimen is saturated and undrained, no volume changes are allowed. Therefore, in the following, the term “dilation” must be understood as “tendency to dilate” and “contraction” as “tendency to contract”.

Fig. 3-17: Comparison between the evolution of maximum shear stress \(\tau\) and pore pressure \(u\) during a cyclic triaxial test.
D. Analysis of the hysteresis loops and of the stress path

Fig. 3-18: Hysteresis loops during a cyclic triaxial test

Fig. 3-18 presents the evolution of the hysteresis loops between the maximum shear strain $\gamma$ and the maximum shear stress $\tau$ during the cyclic triaxial test. The maximum and minimum shear stresses decrease progressively and reach zero after 17 cycles. The hysteresis loops presents two fixed points. The stresses and strains corresponding to these fixed point are:

- Shear strain $\gamma = 0.95\%$ and shear stress $\tau = -3\text{kPa}$
- Shear strain $\gamma = -1\%$ and shear stress $\tau = -1\text{kPa}$

Every cycle crosses these points excepted the first one. The fixed points correspond to the points where the shear stress vanishes (Fig. 3-19) i.e. when the axial $\sigma_1'$ and the radial $\sigma_3'$ stresses are equal (isotropic state). This means the deformation corresponding to the equilibrium state of the specimen does not depend on the number of cycles.

Fig. 3-19 compares the shape of the hysteresis loop of the 5th cycle with particular values of the pore pressure:

- The minima of pore pressure correspond with the minimum and maximum shear strain of each cycle. When the displacement changes direction, the specimen behaviour changes from dilative to contractive.
- The maxima of pore pressure correspond with inflexion point of each branch of the loop. This means the specimen requires more energy to be deformed in dilation than in contraction.
At the granular scale, the following explanation allows a better understanding of the shape of the hysteresis loops. During the dilative phase of the cycle, the grains are rolling on each other to follow the imposed deformation, and are trying to increase the volume of the specimen. Because of the undrained condition of the test that prevents global volume changes, this tendency creates a decrease of the pore pressure and an increase of the stresses between the grains. When the direction of the deformation changes, the stress distribution between the grains is modified and the grains move backward in the hole they were in before the dilation. During this reorganisation, the stresses between grains decrease and the contact between them can even be broken. The grains become more free and tend to a more stable state by trying to reduce the volume of the specimen. This tendency results in an increase of the pore pressure. For a certain deformation, a new structure is formed and the specimen cannot be deformed anymore by reducing the volume any further. To follow the imposed further deformation, the grains have to roll on each other and a new phase of dilation is observed. The successive reorganisations of grain structure lead progressively the specimen to a structure where the grains can move freely without being in contact. No more stresses between grains are observed and no more resistance can be obtained. The specimen is liquefied.

Fig. 3-19: Hysteresis loop during the 5th cycle and comparison with the different state of pore pressure.

Fig. 3-20 presents the stress path followed by the specimen during the triaxial test. This stress path is characterised by the “butterfly” shape of the loops. During each dilative phase, the specimen follows a straight line. The slope of the line is different in extension and in compression. It is equal to 58.5° in compression and 46° in extension. The shapes of the stress path during each loop seem to be an homothetic transformation of each other. This observation is confirmed by the analysis of the evolution of the stress ratio (Fig. 3-21). Indeed, the loops of the stress ratio are not
function of the cycle number and are identical during all the cycles except for the first loading. The shape more erratic observed for Cycle 15 is the consequence of the low values of the stresses measured during that cycle.

Fig. 3-20: Stress path followed during a cyclic triaxial test.

Fig. 3-21: Evolution of the stress ratio during a cyclic triaxial test.
**E. Sample degradation**

The following paragraphs describe the degradation of the specimen resistance during a cyclic triaxial test. The evolution of the degradation is analysed based on two parameters: the number of cycles and the energy dissipated during the cycles. These paragraphs will introduce some empirical equation that fit the experimental data.

**E.1 Sample degradation as a function of cycle number**

The evolution of the secant shear modulus is represented on Fig. 3-22 as a function of the cycle number. The degradation of the secant shear modulus represents the decrease of the difference between the maximum and minimum shear stresses. Until the 12th cycle, the degradation of the secant shear modulus follows a semi-logarithmic relationship that can be described using Eq. 3-29.

\[
G_{s_n} = \frac{\Delta \tau_{\text{max}}}{\Delta \gamma_{\text{max}}} = G_{s_1} \cdot \left(1 - \log_{10}(N^{\alpha})\right)
\]

(Eq. 3-29)

where

- \(G_{s_n}\) is the secant shear modulus at the \(n^{th}\) cycle
- \(G_{s_1}\) is the secant shear modulus at the 1st cycle
- \(N\) is the cycle number
- \(\alpha\) is an empirical parameter

The values of these parameters that fit the measurements of the analysed test are:

\[G_{s_1} = 13 \text{ MPa} \quad \text{(measured value = 12.3MPa)}
\]

\[\alpha = 0.8\]

The parameter \(\alpha\) represents the number of cycles required to reach a zero secant shear modulus and can be calculated using Eq 3-30. For the test presented, the calculated number of cycles \(N_{G_{s_n}=0}\) required to liquefy the specimen is 17.8.

\[N_{G_{s_n}=0} = 10^{\frac{1}{\alpha}} \]

(Eq. 3-30)

where \(N_{G_{s_n}=0}\) is the number of cycle required to reach a zero secant shear modulus

\(\alpha\) is the empirical parameter of Eq. 3-29.

After the 12th cycle, a neck appeared near the top of the specimen. After that event, the measurements and the assumptions made during the processing of the data are no more representative of the stress and strain in the specimen. Indeed, the region of the specimen where the neck appeared is the weakness of the specimen and the deformations imposed to the specimen are probably localised in that area. The assumption of homogeneity of stresses and strains is no more respected. This observation explains that the end of the curve turns progressively.
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Fig. 3-22: Degradation of the secant shear modulus during cyclic triaxial test.

Fig. 3-23 compares the evolutions of the different degrees of liquefaction that can be defined as a function of the number of cycles. The different degrees of liquefaction of \( n \) th cycle are defined as the ratio between a particular excess pore pressure and the initial effective mean stress imposed to the specimen during the consolidation \( P_0' \). On Fig. 3-23, six different choices of excess pore pressure were selected:

1. The pore pressure at the first strain reversal (maximum strain)
2. The pore pressure at the half of the cycle (i.e. when the strain equal to 0 during extension phase)
3. The pore pressure at the second strain reversal (minimum strain)
4. The pore pressure at the end of the cycle (i.e. when the strain equal to 0 during compression phase)
5. The maximum pore pressure during the cycle
6. The average pore pressure during the cycle.

The value of degree of liquefaction vary strongly as a function of the selected definition. However, the degrees of liquefaction grow progressively to a value equal to 1 for which the effective stresses vanish: there are no more stresses between the grains and no more resistance is measured. This state corresponds to liquefaction.

Due to the dilation phases that induce a decrease of the pore pressure, it is possible to calculate negative values of the degree of liquefaction. This phenomena occurs principally for the degrees of liquefaction defined at the strain reversal (corresponding to the end of each dilation phases).
Fig. 3-23: Comparison between the different possible definitions of the degree of liquefaction during a cyclic triaxial test.

Among the different possible definitions of the degree of liquefaction, the present research defined the degree of liquefaction \( d_{\text{liq}} \) as the ratio between the excess pore pressure measured at the end of the cycle and the initial effective mean stress imposed to the specimen during the consolidation \( P_0' \). This definition corresponds roughly to the maximum possible value of degree of liquefaction (Fig. 3-23). Fig. 3-24 shows the evolution of this parameter as a function of the cycle number \( N \). Different simple analytical equations (semi-logarithmic, exponential, inverse, …) were tried to fit the experimental curve, but none was able to reproduce correctly the experimental data. Therefore, to find the number of cycles needed to reach the liquefaction (\( d_{\text{liq}}=1 \)), the following equation has been used (for \( d_{\text{liq}}<1 \)):

\[
d_{\text{liq}} = \frac{u_{\text{end of the } n^{th} \text{ cycle}}}{\sigma_{\text{v,consolidation}}} = d_{\text{liq}}^1 + \frac{D(N - 1)}{1 + E(N - 1)}
\]

(Eq. 3-31)

where \( d_{\text{liq}}^n \) is the degree of liquefaction at the \( N^{th} \) cycle
\( d_{\text{liq}}^1 \) is the degree of liquefaction at the 1st cycle
\( D \) and \( E \) are empirical parameters

The values of these parameters for the analysed test are:
- \( d_{\text{liq}}^1 = 0.641 \) (measured value = 0.64)
- \( D = 0.201 \)
- \( E = 0.500 \)

The result of the regression is presented on Fig. 3-24. With this equation, the number of cycles needed to obtain the liquefaction (\( d_{\text{liq}}^n=1 \)) is 17.4. This value is similar to the value found with the model for the degradation of the secant shear modulus.
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Fig. 3-24: Build-up of pore pressure during cyclic triaxial test.

E.2 Sample degradation as a function of the dissipated energy

The evolution of the specimen degradation during a cyclic triaxial test can also be described as a function of the energy dissipated during the N first cycles. This energy is calculated by adding the surface of the N first hysteresis loops (Eq 3-32).

\[ W_{tot, n} = \sum_{i=1}^{n} \text{surface of the hysteresis } \gamma - \tau \text{ of the cycle } i \]  

(Eq. 3-32)

Fig. 3-25 and Fig. 3-26 show the evolution of the secant shear modulus \( G_s_n \) and the degree of liquefaction \( d_{liq} \) as a function of the total energy dissipated \( W_{tot} \). The evolution of the secant shear modulus during a cyclic triaxial test is linear. This relationship can be modeled with the following equation (Eq 3-33).

\[ G_s_n = G_s_1 \left( 1 - \beta \left( \frac{W_{tot, n}}{W_{tot, 1}} \right) - 1 \right) \]  

(Eq. 3-33)

where \( G_s_n \) is the secant shear modulus of the \( n^{th} \) cycle
\( G_s_1 \) is the secant shear modulus of the 1\(^{st}\) cycle
\( W_{tot, n} \) is the energy dissipated during the \( n \) first cycles
\( W_{tot, 1} \) is the energy dissipated during the 1\(^{st}\) cycle
\( \beta \) is an empirical parameter.

The values of those parameters for the analysed test are:
\( G_s_1 = 11.9 \text{MPa} \)  
(measured value = 12.3MPa)
\( W_{tot, 1} = 0.0036 \text{MJ/m}^3 \)  
(measured value = 0.0034MJ/m3)
\( \beta = 0.27 \)
Based on Eq 3-33, the energy needed to liquefy the specimen during the cyclic triaxial test is equal to 0.0167MJ/m³.

**Fig. 3-25:** Degradation of the secant shear modulus as a function of the dissipated energy during cyclic triaxial test.

**Fig. 3-26:** Build-up of pore pressure as a function of the dissipated energy during cyclic triaxial test.
Fig. 3-26 shows (that) the relationship between the total dissipated energy $W_{\text{tot}}$ and the degree of liquefaction can be described with a semi-logarithmic equation (Eq 3-34).

$$d_{\text{liq}} = \frac{d_{\text{liq, end of n-th cycle}}}{P} = d_{\text{liq, 1st cycle}} + \log_{10}\left(\frac{W_n}{W_1}\right)$$

Eq. 3-34

where $d_{\text{liq, n}}$ is the mobilised liquefaction pore pressure after the $n^{th}$ cycle

$d_{\text{liq, 1}}$ is the mobilised liquefaction pore pressure after the 1$^{th}$ cycle

$W_{\text{tot, n}}$ is the energy dissipated during the $n$ first cycles

$W_{\text{tot, 1}}$ is the energy dissipated during the 1$^{st}$ cycle

$\lambda$ is an empirical parameter

For the test hereby presented, the values of the different parameters are the following:

$d_{\text{liq, 1}} = 0.70$ (measured value = 0.64)

$W_{\text{tot, 1}} = 0.0046 \text{ MJ/m}^3$ (measured value = 0.0034 MJ/m$^3$)

$\lambda = 0.496$

Based on Eq 3-34, the total energy needed to reach a degree of liquefaction equal to 1 is 0.0169 MJ/m$^3$

Fig. 3-27 presents the secant shear modulus as a function of the energy dissipated during each cycle. An exponential analytical equation can be used to describe this relationship:

$$G_{s_n} = A \cdot W_n^B$$

(Eq. 3-35)

where $G_{s_n}$ is the secant shear modulus of the $n^{th}$ cycle

$W_n$ is the energy dissipated during the $n^{th}$ cycle

$A$ and $B$ are empirical parameters.

The values of the parameters $A$ and $B$ are respectively 1200 and 0.8.

The exponential curve fits very well the experimental data until the 15$^{th}$ cycle, when the neck appears in the specimen. As mentioned before, the apparition of the neck changes the strains and stresses distribution. This observation can explain the deviation of the measurements from the proposed analytical curve.
III.3.3 Influence of the loading parameters on the results of cyclic triaxial testing

A. Introduction

The present research aims principally to investigate the influence of the shear strain amplitude $\gamma_a$, the relative density $D_r$ and the consolidation stress $P_c'$ on the strength degradation during a cyclic triaxial test. The conclusions deduced from each triaxial test are the results of a long experimental procedure where some simplifications and assumptions were made. In order to evaluate the scientific value of these conclusions, the following questions have to be investigated:

- Are the cyclic triaxial tests repeatable?
- Does the specimen stay homogenous during the cyclic triaxial test?
- What is the influence of the stress path ($P=\text{cst}$ or $\sigma_3=\text{cst}$)?
- What is the influence of the starting direction (extension or compression)?

The present paragraph is going to address these different questions. The objective is to validate the experimental setup followed for the cyclic triaxial tests or to find the limitations to the conclusions deduced from the triaxial tests that have to be taken into account due to the choice of the experimental procedure.
B. Cyclic triaxial test repeatability

In order to verify that the results of the cyclic triaxial tests are repeatable, different tests with the same parameters were performed. Three sets of parameters were chosen investigating 2 relative densities, 2 shear strain amplitudes and 2 total mean stresses. The comparison of the evolution of the secant shear modulus degradation and the pore pressure build-up between the different tests (Fig. 3-28) shows the results of the cyclic triaxial tests can be considered repeatable. This good repeatability is probably the consequence of the precise procedure followed carefully during the test execution. It also results from large-size specimen. Indeed, a small error during the specimen preparation, saturation or consolidation has a lower impact on a large specimen than the same error on a small specimen.

Fig. 3-28: Analysis of the cyclic triaxial tests repeatability.
III. Cyclic Triaxial Test

C. Investigation of the homogeneity of the specimens

The test processing and analysis assume the specimen is homogenous during the cyclic triaxial tests. However, on one side, some authors (Sheng, 1997) have shown the stress distribution is not homogenous during a triaxial test, and stress concentrations must exist close to the specimen extremities. On the other side, a neck appeared at the end of each cyclic triaxial tests performed during this research. Therefore, in order to make sure the soil behaviour deduced from the cyclic triaxial test is really representative of the soil, the homogeneity of the specimen was investigated further.

The apparition of the neck at the top of the specimen indicates the specimen is no longer homogenous at the end of the test. Indeed, it seems the axial strain is concentrated in the vicinity of the neck, and no longer evenly distributed along the specimen. Since the neck always appeared at the end of the test when the specimens were close to the liquefaction, it can be considered that the resulting heterogeneity does not influence the analysis of the evolution of the soil degradation before the apparition of the neck.

In order to verify the specimen is homogenous during the degradation, a cyclic triaxial test was stopped at the middle of the degradation and the specimen was frozen and analysed. Fig. 3-29 compares the evolution of the secant shear modulus of this test with an other similar test. When the shear strain was equal to zero at the end of the 4th cycles, the test was stopped and the triaxial cell was placed in a cold chamber (with a temperature of −10°C). During the freezing, the lateral pressure was kept constant. To avoid the freezing of the fluid in triaxial cell, oil was used to fill it. After 3 days, the cell was dismounted and the frozen specimen was cut in 10 slices of 2cm (Fig. 3-30).
Each slice was cut in 4 quarters. The water content and the density of each quarter was measured (Fig. 3-31).

The distribution of the water content is relatively constant along the specimen. The average value is equal to 23.09%. This value is close to the value of 23.84% deduced from the specimen preparation (error of 3%). The average dispersion of the water content is equal to 4.42% with a maximum for the top layer where the deviation reaches 13%. The range of possible values (min to max) is equal to 17% of the average water content.

The dry density $\gamma_d$ has an average value of 15.15kN/m³ instead of 15.5kN/m³ (difference of 2%) that is deduced from the specimen preparation. The average deviation around this mean dry density is 1.5% with a maximum value of 4.4% at the specimen top. The range of possible values is equal to 6.5% of the average dry density.

Fig. 3-31: (a) Water content and (b) density distributions in the specimen frozen after 4 cycles of cyclic triaxial test.
These results show that the specimen can be considered homogenous during a cyclic triaxial test, excepted at its extremities. Indeed, the principal deviations of the water content and of the dry density are located at the specimen top and base. These deviations can result from the cyclic test or from the frost action. Indeed, it is well known that during the freezing of soil, water in the specimen is redistributed due to the cryogenic suction. Laboratory investigations performed by Coté (2000) on sand specimens concludes that a difference of the water content of about 1 to 2% is expected between the extremities and the centre of a triaxial specimen during the freezing process.

In conclusion, it can be considered that the central area of the specimen remains homogenous during the cyclic test. The weaknesses observed at the specimen extremities result in a part from the cyclic test and an other part from the freezing action. The heterogeneity induced by the cyclic test at the specimen extremities probably increases as a function of the soil degradation and is responsible of the apparition of a neck at the specimen top when the sample is close to the liquefaction. To confirm these conclusions and separate the action of the freezing process from the influence of the cyclic test, more analysis are needed on samples frozen after different number of cycles (e.g. 1, 2, 6, …).

D. Influence of the stress path

The cyclic triaxial tests performed in the framework of this investigation follow a stress path where the total mean stress \( P = (\sigma_1 + 2\sigma_3)/3 \) is maintained constant. This is carried out by regulation of the total lateral stress \( \sigma_3 \) as a function of the measured total axial stress \( \sigma_1 \). The objective of this paragraph is to study the influence of the choice of the stress path on the cyclic degradation of soil.

Fig. 3-33 presents the comparison of 8 cyclic triaxial tests performed with 2 different stress paths (\( \sigma_3 \) const or \( P \) constant - Fig. 3-32), 2 relative densities Dr (70 % and 86%) and 2 shear strain amplitudes \( \gamma_a \) (0.75% and 1.5%).

Fig. 3-32: Stress paths during triaxial test.
Fig. 3-33: Stress path influence on (a) the secant shear modulus degradation, (b) the build-up of pore pressure and (c) the evolution of the accumulated dissipated energy during cyclic triaxial tests.
Based on the available test results, the influence of the stress path on the test results can not be determined easily. However, it seems that the choice of the stress path has low influence on the number of cycles required to reach the cyclic liquefaction during a cyclic triaxial test regarding the other test parameters (Dr, γa,...) specially for test performed with a low relative density. The tests performed on the looser specimen (Dr=70%) can be considered identical. At the other hand, the tests carried out on the denser sand show a soil behaviour more sensitive to the stress path.

E. Influence of the starting direction

The cyclic triaxial tests performed during this investigation start generally by compressing the specimen. Since the soil behaviour is different in extension and in compression, there is a need to investigate in greater detail the influence of the starting direction. This paragraph compares the results of 4 tests starting in opposite directions for two relative densities (Fig. 3-34).

The comparison between the cyclic triaxial tests starting in extension and in compression (Fig. 3-34) shows that the starting direction has no significant influence on the secant shear modulus degradation and on the pore pressure build-up. Only the secant shear modulus of the first cycles of the tests performed on loose specimen are different. This observation results from the smaller resistance of soil specimen under triaxial extension, compared to triaxial compression. During the first cycle of the test starting in extension, the specimen reached the critical state. This results also shows that the granular structure of a soil specimen sheared under large cyclic strain, is modified during each cycle in such a way that the specimen looses the memory of the previous cycle.
Fig. 3-34: Influence of the starting direction on (a) the secant shear modulus degradation and (b) the build-up of pore pressure during cyclic triaxial tests.

**F. Conclusion**

The present research principally focused the analysis on the influence of the shear strain amplitude, the relative density and the consolidation stress on the soil degradation. However, to validate this analysis or to find its limitations, it was necessary to study the influence of the loading parameters on the cyclic triaxial tests results.

The comparison of similar tests showed that the cyclic triaxial tests are very repeatable. The results of these tests can be trusted and are representative of the soil behaviour during triaxial shearing, for the corresponding shear strain amplitude, relative density and the consolidation stress.

The analysis of specimen homogeneity during cyclic triaxial tests has shown that the specimen can be considered homogenous until the soil resistance becomes low. When the specimen is close to the liquefaction (cyclic mobility), a neck appears near the top of the specimen. At that time, the strain distribution is no longer homogenous.

Tests performed with different stress paths concluded that the stress path has almost no influence on the soil degradation during cyclic triaxial tests. The starting direction (extension versus compression) has no influence on the soil degradation.
III.3.4 Comparison between the different tests

The following paragraphs analyse the influence of the shear strain amplitude $\gamma_a$, the relative density $D_r$ and the initial effective stress $P_c'$ on the specimen degradation. Based on comparisons between the different tests, the objective is to identify the mechanics and the key parameters that lead the soil strength degradation during cyclic triaxial test.

A. Influence of the strain amplitude

The degradation of the soil during a cyclic triaxial test is strongly influenced by the shear strain amplitude (Fig. 3-35). The higher the shear strain, the faster the degradation.

The degradation of the secant shear modulus (Fig. 3-35-a) during large strain amplitudes tests (>1.5%) follows a semi logarithmic degradation and can be described using the same equation as presented in the previous paragraph (Eq 3-29). The evolution is not similar for small amplitudes. This could result from the combination of two phenomena: the first related to the experimental setup and the second related to the soil behaviour.

The vertical displacement corresponding to shear strains of 0.075% and 0.15% are respectively 0.1mm and 0.2mm i.e. the same range of the grain diameters ($d_{50} = 0.18\text{mm}$). The sensitivities of the transducers and the load frame were adapted to these amplitudes. The observations do not results from an inaccuracy in the measurements or in the regulation. However, the beginning of the test is characterised by a transition of the grain condition from an isotropic condition to an anisotropic one, that induces a reorganisation of the grain skeleton. This reorganisation occurs rapidly for large strain amplitudes, due to the dilative behaviour of soil that modifies the grain structure more or less independently from the initial stress. The tests performed with a small amplitude, where the behaviour during the first cycle is only contractive and then dependent of the initial stress, require some cycles to adapt the grain structure to the new type of solicitation. Due to this phenomenon, the secant shear modulus can even increase during the first cycle.

The second explanation of the difference between the large and small strain amplitudes tests is the difference between the dilative and the contractive behaviour of the specimen. The test results show the degradation is faster when the specimen follows 2 phases of dilation and 2 phases of contraction during each cycle, than when the specimen has only the tendency to contract. During the first cycles of small amplitudes cyclic tests (about 2 for $\gamma_a=0.15\%$ and 20 for $\gamma_a=0.075\%$), the sample behaves only in contraction. The corresponding degradation rate is low. After some cycles, the behaviour becomes dilative and contractive and an increase of the degradation rate is observed.
The combination of these two phenomena explains the evolution of the secant shear modulus degradation:

1. a small increase of the soil resistance during the adaptation of the soil structure to the new solicitation type
2. a relatively low degradation rate during the cycles when the soil behaviour is only contractive
3. a stronger degradation when the soil behaviour follows different phases of contraction and dilation.

The points 1 and 2 do not exist for cyclic triaxial test whose amplitude is larger than 0.375% because during the first cycle, the soil behaviour is dilative enough to cancel the effects of the first two phenomena.

The degree of liquefaction at the end of each cycle (Fig. 3-35-b) increases with the shear strain amplitude. This observation could be look surprising. Indeed, it is surprising that the pore pressure at the end of each cycle grows systematically whereas the specimen follows 2 strong dilative phases during each cycle. During the large strain tests, the degree of liquefaction measured was very low and even negative in some case at the end of the dilative phases, but the more dilative the specimen, the higher the degree of liquefaction at the end of the cycle. In conclusion, there is no equilibrium for which the dilative behaviour of the soil compensates the increase of pore pressure. The dilative behaviour even favours the degradation phenomenon: the more dilative the soil, the higher the pore pressure build-up at the end of each cycle.

The relationships between the accumulated dissipated energy and the secant shear modulus shown in Fig. 3-35-c is linear for the tests performed with a large strain amplitude ($\gamma_a > 0.375\%$). This linearity is no longer verified for small strain amplitude tests. It seems that the linearity depends principally on the dilative behaviour of the soil. Indeed, this phenomena is observed only during tests where the soil has a strong dilative behaviour, i.e. where the stress path becomes tangent to the critical states lines during the dilative phases in contraction and extension (= the butterfly shape). Moreover, the relationship between the accumulated dissipated energy and the secant shear modulus during a small strain amplitude test becomes linear when the stress path becomes tangent to the critical states lines.

Fig. 3-36 shows the evolution of the secant shear modulus measured during the first cycle of a cyclic triaxial test $G_s1$ as a function of the shear strain amplitude $\gamma_a$. The data were measured during cyclic triaxial tests, excepted the measurement corresponding to a shear strain amplitude of $10^{-3}\%$ that was measured using bender elements in a triaxial cell. The different types of behaviour (see Chapter 2) described in the literature by different authors (e.g. Dobry, 1987; Kokusho, 1980) can be identified on the curve.

As shown on Fig. 3-37, the results are very similar to the measurements proposed by Kokusho (1980) for the Toyoura sand and to the values calculated with the empirical formula proposed by Ishibashi (1993) (see Chapter 2).
III. Cyclic Triaxial Test

Fig. 3-35: Influence of shear strain amplitude on degradation during cyclic triaxial tests ($P_0' = 135\text{kPa}$ and $Dr = 70\%$): (a) secant shear modulus degradation, (b) pore pressure build-up; (c) relationship between normalised secant shear modulus and normalised accumulated dissipated energy.
For strain amplitudes under $10^{-3}\%$ the soil behaves as a linear elastic material. The secant shear modulus is maximum and no degradation is observed. For Brusselian sand, the threshold of non linearity is about $10^{-3}\%$ (Fig. 3-36). The cyclic triaxial tests showed that, for shear strains between $10^{-3}\%$ and $10^{-2}\%$, the relationship between the shear stress and the shear strain presents an hysteresis loop but no degradation is observed. For strain amplitudes larger than $10^{-2}\%$, the soil behaviour is elasto-plastic and the soil resistance decreases as a function of the cycle number. For the Brusselian sand, the threshold of degradation is around $10^{-2}\%$ (Fig. 3-36).

Fig. 3-36: Influence of the shear strain amplitude on secant shear modulus of the first cycle during cyclic triaxial tests ($P'_0 = 135\text{kPa}$ and $\gamma_a = 86\%$)

Fig. 3-37: Comparison of the evolution of the secant shear modulus between the Brusselian sand, the Toyoura sand (Kohusho, 1980) and the formula proposed by Ishibashi (1993).
The relationship between the dissipated energy during each cycle (i.e. the surface of the hysteresis loops) and the secant shear modulus of the corresponding cycle follows an exponential law (Fig. 3-38) that can be described by the following equation:

\[ G_{sn} = A \cdot W_n^B \quad \text{(Eq. 3-36)} \]

where \( G_{sn} \) is the secant shear modulus of the \( n^{\text{th}} \) cycle,
\( W_n \) is the energy dissipated during the \( n^{\text{th}} \) cycle,
\( A \) and \( B \) are empirical parameters.

The analysis of Fig. 3-38 compares all the tests performed in this research (different shear strain amplitudes, initial stresses and different densities). It concludes the value of the exponent \( B \) is nearly constant for all the tests performed, and is equal to 0.8 where \( G_{sn} \) is expressed in MPa and \( W_n \) in MJ/m³. The parameter \( A \) seems to depend only on the strain amplitude \( \gamma_a \) while it is independent from the relative densities and the consolidation stress investigated.

Fig. 3-38: Relationship between secant shear modulus and dissipated energy during cyclic triaxial tests for different shear strain amplitudes, relative densities and initial effective mean stress: (a) relative density \( \text{Dr} = 70\% \); (b) relative density \( \text{Dr} = 86\% \).
The value of parameter A is drawn in Fig. 3-39 as a function of the shear strain amplitude for different relative densities and consolidation stresses. The following exponential equation fits rather well to the experimental results:

\[ A = 2000 \gamma_a^{-1.8} \]  
(Eq. 3-37)

Taking into account Eq 3-36, Eq 3-37 can be rewritten to express the secant shear modulus based on the shear strain amplitude and the energy dissipated during each cycle (Eq. 3-38).

\[ G_s = 2000 \frac{W_n^{0.8}}{\gamma_a^{1.8}}, \text{ or} \]  
(Eq. 3-38)

\[ \Delta \tau_{\text{max}} = \tau_{\text{max}} - \tau_{\text{min}} = 4000 \left( \frac{W_n}{\gamma_a} \right)^{0.8} \]  
(Eq. 3-39)

\( G_s \) in MPa, \( W_n \) in MJ/m³, \( \Delta \tau_{\text{max}} \) in MPa and \( \gamma_a \) in %

This relationship indicates that an unique relationship exists between the extremum points of a hysteresis curve and its area.

Fig. 3-39: Exponential relationship between parameter A and shear strain amplitude during cyclic triaxial tests for different shear strain amplitudes, relative densities and initial effective mean stress.

Fig. 3-40 compares the hysteresis loops and the stress paths coming from different cyclic triaxial tests performed on dense specimens (Dr = 86%) and having the same secant shear modulus. The differences between these hysteresis loops are the corresponding cycle number N and the initial effective mean stress \( P_0' \). When the soil has a dilative behaviour, the shape of the stress path and the hysteresis loops are neither influenced by the cycle number nor by the initial soil condition.
The dilation phases sweep out the soil memory and the resistance is then only function of the current stress state. It can be remarked that the cycles presented on Fig. 3-40 have the same effective mean stress at the beginning of the cycle.

An other indication of the dependency of the degradation phenomena to the effective mean stress is illustrated on Fig. 3-41 where the secant shear modulus of each cycle divided by the secant shear modulus of the first cycle $G_{s,n}/G_{s1}$ is plotted against the degree of liquefaction of the previous cycle $d_{liq,n-1}$. Since, by definition, the degree of liquefaction is representative of the pore pressure at the end of each cycle, the degree of liquefaction of the $n-1^{th}$ cycle corresponds to the pore pressure at the beginning of the $n^{th}$ cycle. Therefore, based on Fig. 3-41, it can be concluded that the value of the secant shear modulus is principally function of the pore pressure at the beginning of the cycle and that the other test parameters seem to have a relatively low influence on this relationship.
In consequence, the research on the degradation law must focus all the attention on the pore pressure build-up between the beginning and the end of each cycle, based on the current stress state. Indeed, based on the different analyses presented above, each stress path and hysteresis loops can be reconstructed starting from the initial stress condition and the shear stress amplitude.

Fig. 3-41: Relationship between the secant shear modulus normalised with the secant shear modulus of the first cycle $G_{n}/G_{1}$ and the degree of liquefaction at the beginning of the cycle $d_{liq\_n-1}$. 
B. Influence of the relative density

The relative density is one of the most important parameters governing soil degradation during cyclic loading. The test results (Fig. 3-42) show that, for a shear strain amplitude of 1.5%, a reduction of the relative density from 85% to 65% results in a reduction of the initial secant shear modulus and of the number of cycles required to liquefy the soil of about a factor 10: the soil structure is reorganised more easily during the cyclic shearing performed on a loose sand than on a dense sand due to the large voids between the grains. There is a limit relative density for which the initial secant shear modulus and the number of cycles needed to liquefy the soil are equal to zero. This state corresponds to the critical void ratio. For this density and looser densities, the soil structure is unstable and a static liquefaction is directly observed upon shearing.

These observations point out the importance to correctly evaluate the soil density when a analysis of the cyclic degradation of a soil is planned. Indeed, an error of 5% on the relative density can generate an error of about 100% on the evaluation of the cyclic resistance.
Fig. 3-42: Influence of relative density on degradation during cyclic triaxial tests ($P_0' = 125$ kPa and $\gamma_a = 0.75\%$ and 1.5\%): (a) secant shear modulus degradation, (b) pore pressure build-up; (c) relationship between normalised secant shear modulus and normalised accumulated dissipated energy.
C. Influence of total mean stress

The influence of the mean stress on the soil behaviour during a cyclic triaxial tests is more important for small amplitudes than for large amplitudes (Fig. 3-43 & Fig. 3-44).

During large amplitude tests, the dilative phase of the first cycle reorganises strongly the soil structure during the first loading. Therefore, the soil specimen looses the memory of the initial condition, and the soil resistance becomes independent of the initial effective mean stress.

At the other hand, during the small amplitude tests, the initial effective mean stress strongly influences the initial secant shear modulus. The higher the initial effective mean stress, the higher the corresponding secant shear modulus. This is because the higher initial confining stress requires to apply an higher strength on the specimen to moves grains relatively to each other. However, the test points out that the initial effective mean stress does not modify significantly the number of cycles required to liquefy the specimen.
Fig. 3-43: Influence of effective initial mean stress on degradation during cyclic triaxial (Dr = 86%): (a) secant shear modulus degradation, (b) pore pressure build-up; (c) relationship between normalised secant shear modulus and normalised accumulated dissipated energy.
Fig. 3-44: Influence of effective initial mean stress on degradation during cyclic triaxial (Dr=70%): (a) secant shear modulus degradation, (b) pore pressure build-up; (c) relationship between normalised secant shear modulus and normalised accumulated dissipated energy.
III.4 Conclusion

The last paragraphs presented the experimental setup and the results of the 45 cyclic triaxial tests performed during this research. These tests were strain controlled and the lateral pressure was regulated to maintain the total effective stress $P$ constant. The tests were performed on medium and dense Brusselian sand and investigated a range of shear strain amplitudes from 0.1% to 9%.

The analysis of the experimental setup showed the good repeatability of the triaxial tests, probably due to the accuracy of the experimental procedure and to the large size of the specimens. The results can be considered independent of the choice of the stress path ($P$ constant or $\sigma_3$ constant) and independent of the starting direction. Measurements performed on frozen specimens showed a relative good homogeneity of the water content distribution and of the density as well.

The main conclusion coming out the test analysis is that the sand degradation is essentially controlled by its tendency to dilate or to contract.

The main observations are the following:

- The pore pressure has a frequency twice that of the shear strain; the maximum pore pressure corresponds to the inflexion point of the shear stress curve. The minimum pore pressure corresponds to the maximum shear strain.

- The shape of the hysteresis loops of the shear stress versus the shear strain is strongly influenced by the evolution of the pore pressure: the shape is convex when the pore pressure increases (contraction phase) and is concave when the pore pressure decreases (dilation phase). The hysteresis loops present also two fixed points. These points correspond to the zero shear stress.

- The shape of the stress path followed during each loop seems to be a homothetic transformation of each other. During each dilative phase, the stress path becomes tangent to the critical state line.

- The degradation of the secant shear modulus as a function of the number of cycles is successfully described with a semi-logarithmic expression. The relationship between the shear strain amplitude and the secant shear modulus of the first cycle $G_s_1$ is similar to the shape proposed in the literature and 3 ranges of shear strain amplitudes can be identified as a function of the soil behaviour: elastic linear behaviour, elasto-plastic behaviour without degradation, and elasto-plastic behaviour with degradation. The evolution of this relationship in the latter range can be separated in two different parts: a semi-logarithmic part and a part corresponding to large strains where the decrease of the initial secant shear modulus is smaller.
• The degradation of the secant shear modulus as a function of the accumulated dissipated energy is linear when the soil has a strong dilative behaviour.

• The secant shear modulus of a cycle can be calculated using an exponential relationship depending on the energy dissipated during the corresponding cycle and the shear strain amplitude. This relationship is independent of the relative density and of the consolidation stress.

• The results of the tests performed with large strain amplitudes showed that even if large dilation phases are observed, the specimen will always reach liquefaction after a few cycles. There is no equilibrium where the dilation phases compensate the contraction phases. An increase of the pore pressure at the end of each cycle is always observed. Furthermore, the dilation accelerates the soil degradation and the number of cycles required to liquefy the soil decreases.

• The relative density is one of the most important parameter influencing soil degradation. The lower the density, the faster the soil degradation. There is a limit density corresponding to the critical density where no cycles are needed to liquefy the soil, and where static liquefaction occurs.

• The initial effective mean stress (or the total mean stress) influences principally the beginning of the tests. This influence depends on the strain amplitude and on the value of the stress. On the other hand, it seems that the number of cycles required to liquefy the soil is independent from the initial effective mean stress.
IV CYCLIC DIRECT SIMPLE SHEAR TEST

IV.1 Introduction

The objective of this section is to present the results of the cyclic direct simple shear (DDS) tests performed within the framework of this research.

The first part presents the experimental setup extending the capabilities of the new (1997) NGI simple shear device to cyclic strain controlled test. This chapter describes also the processing of the measurements and the assumptions done in the analysis of the test. The second part presents the detailed results of a cyclic simple shear test and introduces some of the analytical equations that are able to describe simply the evolution of soil degradation. The last part compares the results of the different cyclic simple shear tests. The problem of the repeatability is analysed. The influence of the amplitude of the cyclic strain, the relative density and the consolidation stress is investigated.

IV.2 Experimental setup

This chapter describes the direct simple shear (DDS) device used in this investigation, the procedure used in the cyclic simple shear tests, the data processing and the assumptions made in the analysis of the test.

IV.2.1 Description of the direct simple shear apparatus

The new NGI direct simple shear apparatus (Fig. 3-45) whose control system has been modified by the author to allow cyclic strain controlled simple shear testing, is used for this research. This device was designed by the Norwegian Geotechnical Institute and is manufactured by Geonor.

This apparatus allows the shearing of a soil specimen in such way that uniform shear strain results throughout the sample. This is accomplished by translating the base of the specimen horizontally relative to a fixed specimen top. The specimen is confined in a wire reinforced rubber membrane that, while permitting specimen shear displacements and axial deformation, does not permit radial strain of the specimen. The cross section of the specimen is remained cylindrical during all the test.

This apparatus allows the control of the horizontal shear strains or the horizontal shear stresses in one horizontal directions, and the control of the axial strain or the axial stress in the vertical direction during the consolidation and during the test. This control is performed using a linear actuator driven by two electrical motors.
All tests are constant volume tests. Since the wire-reinforced membrane tends to prevent radial specimen strain, the specimen volume is held constant by maintaining a constant specimen height. This is accomplished by changing the vertical stress applied on the specimen. This change in the vertical stress is assumed to be equivalent to the excess pore pressure buildup in an undrained test.

Drained shear testing can also be performed using this device by maintaining a constant axial load during the test. However, such tests have not been performed in this investigation.

**IV.2.2 Sample preparation**

The NGI trimming apparatus is used to prepare the specimen for all the tests.

The specimen is prepared by the method of moist tamping in one lift. The initial water content is 8%. For the triaxial tests (discussed previously), the initial water content was 12%. However, this value was found to be too high for obtaining a properly saturated specimen during the simple shear test.

The specimen is tamped directly in a reinforced membrane (Fig. 3-46-a) in two different operations. The first operation (Fig. 3-46-b) consists of tamping the specimen with an apparatus that assures the same compaction of the sand near the membrane.
than in the middle of the specimen. After placing the top cap on the specimen, the second operation (Fig. 3-46-c) consists of tamping the specimen on the entire surface until the height corresponding to the desired relative density is reached. During the second phase of the tamping, the number of hammer blows needed is counted. Usually, the same number of blows was needed for each specimen having the same density.

The specimen height is usually 16mm and the surface area is 35cm$^2$. These sizes correspond to a volume of 56cm$^3$ (i.e. 28 times smaller than the specimen in the triaxial test). The membrane used was reinforced by iron-nickel wire with a diameter of 0.2mm. The wire is wound at 30 turns per centimetre of membrane height (0.3mm centre to centre spacing).

**IV.2.3 Consolidation and saturation**

After preparation, the specimen is placed in the DSS apparatus (Fig. 3-47) and loaded to the axial stress desired for the test. The load is applied progressively with a constant strain rate of 0.03 mm/min. The time to reach the desired stress is about 10 minutes. Following the loading, a flow of CO$_2$ is sent through the specimen, from the bottom to the top for approximately 5 minutes.

The specimen is then saturated by flushing water. Air bubbles are allowed to leave the specimen at the top of the specimen. When no more air bubbles are flushing out, the drainage of the top of the specimen is connected to a burette. The level of water in the burette is placed in order to maintain a pore pressure at the specimen extremities equal to the atmospheric pressure during the consolidation and the cyclic test.
The time of the consolidation is exactly 6 hours during which the axial and horizontal stresses and strains are monitored. This time was chosen to have an axial deformation smaller than 1µm in the last hour, and to minimise the effect of creep of the specimen. The control system of the vertical motor maintains the vertical stress constant with a maximum variation of 0.5%. If a horizontal shear stress is measured, the horizontal strain is corrected to maintain this stress equal to zero.

It is important to note that in the simple shear test, the consolidation of the specimen is anisotropic. The specimen is consolidated under $K_0$ conditions (virtually zero lateral strains).

**IV.2.4 Cyclic test**

After the consolidation, the soil specimen is sheared with a cyclic horizontal strain. The shear strain amplitudes used in this investigation cover the shear strains from 0.25% to 9%. The frequency of this cyclic displacement is 0.0005Hz (2000seconds/cycle), except for the amplitudes up to 7% where the maximum speed of the motor limits the frequency at 0.00025Hz (4000 seconds/cycle).

All tests are constant volume tests. The total vertical stress is assumed constant during the test, and the variation of the axial load needed to keep the height of the specimen constant is assumed to be equal to the build-up of the pore pressure.
After the completion of the test (i.e. when no more resistance is observed), the final characteristics (weight and water content) are measured. Other parameters (initial and final void ratio, degree of saturation,…) are then calculated based on those measurements.

IV.2.5 Data acquisition system and data processing

A. Regulation and Data acquisition

A new data acquisition program was developed by the author to expand the capabilities of the NGI simple shear device, to allow cyclic strain controlled testing using the electrically-driven shear motor. This program was written using LabVIEW software. The program acquires the measurements of the vertical and horizontal load cells and of the horizontal and vertical displacement transducers. It converts the measured voltages in physical values and calculates the corresponding strains and stresses. Based on the horizontal shear strain measurement, the program drives the horizontal motor to obtain a cyclic shear strain of the specimen. An independent system controls the sample height during the test.

Measurements are taken at a frequency of 50Hz (100 000 measurements/cycle). These 50 measurements per second are filtered and reduce to 5 measurements per second (10 000 measurements/cycle). The filtering is needed to eliminate any noise and spikes in the signal and to obtain a signal smooth enough for regulation purposes. All the filtered data is used to regulate the horizontal strain but only 1 measurement per second is recorded (2000 measurements/cycle). After the test, the number of data is reduced from 2000 measurements/cycle to 200 measurements/cycle by averaging each 10 measurements. The processing is based on those 200 measurements per cycle.

B. Data processing

B.1 Measurement corrections

Friction correction

The amplitude of the friction in the horizontal direction during a cyclic simple shear test was found equal to 7N (Δτ = 2kPa). This friction is not important during a monotonic test because it becomes rapidly very small compared to the shear resistance of the soil. However, in a cyclic strain controlled test, the friction has to be taken into account because at the end of the test, the shear resistance is on the same order of magnitude as the friction. So, two friction tests are performed before and after each cyclic test. These friction tests consist of one cycle without any load on the specimen. A comparison of all the friction tests shows good agreement. The correction of the horizontal load of the cyclic simple shear test is made using an analytical equation that fits the experimental results of the two friction tests. An example of this correction on the first and the last cycle is shown on Fig. 3-48.
Membrane correction

A membrane correction is also applied to the horizontal load. This correction takes into account the resistance of the membrane during shearing. The calibration curves made by NGI were used. These curves are a function of the type of membrane, the membrane thickness and the shear strain. An example of this correction on the first and last cycles of a cyclic test is shown on Fig. 3-48.

Zero correction

The first zero correction is applied to the measured horizontal load. This correction is needed because it is not possible to know the value of the friction at the beginning of the test. However, using precautions during the preparation of the test, this correction becomes less than 1N ($\Delta \tau = 0.3$ kPa).

The second zero correction is applied on the measurement of the vertical load applied during the consolidation (axial total stress). This correction takes into account the weight of the top cap, the friction in the vertical direction and the membrane resistance. This correction is based on the measurement of the last cycle when a complete liquefaction is assumed (i.e. measured axial load equal to 0). This correction does not exceed 5N ($\Delta \sigma_v = 1.5$ kPa i.e. < 2.5% of the desired value for the tests reported herein).

B.2 Stress and strain computations

The strains are calculated by dividing the measured vertical and horizontal displacement by the height of the specimen after the consolidation (Eq 3-40 and 3-41). It is assumed that the strains are homogenously distributed and that the sections of the specimen stay circular along the specimen height.
\[ \gamma_H = \frac{\text{HD}}{h_0 - \Delta h_{\text{cons.}}} \times 100 \]  
\[ (\text{Eq. 3-40}) \]

\[ \varepsilon_V = \frac{\text{VD}}{h_0 - \Delta h_{\text{cons.}}} \times 100 \]  
\[ (\text{Eq. 3-41}) \]

where \( \gamma_H \) is the horizontal shear strain
- HD is the measured horizontal displacement
- VD is the measured vertical displacement
- \( h_0 \) is the initial height of the specimen
- \( \Delta h_{\text{cons.}} \) is the deformation observed during consolidation

The horizontal shear stress \( \tau_H \) is calculated by dividing the measured horizontal load - corrected with the 3 corrections presented in the previous paragraph, i.e. friction correction, membrane correction and zero correction - by the surface area of the specimen. The specimen section is assumed to be constant on all the specimen height.

\[ \tau_H = \frac{\text{HL} + \Delta \text{HL}_{\text{friction}} + \Delta \text{HL}_{\text{membrane}} + \Delta \text{HL}_{\text{zero}}}{A_0} \]  
\[ (\text{Eq. 3-42}) \]

where \( \tau_H \) is the horizontal shear stress
- HL is the measured horizontal load
- \( \Delta \text{HL}_{\text{friction}} \) is the friction correction
- \( \Delta \text{HL}_{\text{membrane}} \) is the membrane correction
- \( \Delta \text{HL}_{\text{zero}} \) is the horizontal zero correction
- \( A_0 \) is the cross-sectional area of the specimen

The total vertical stress is assumed to stay constant during the constant volume simple shear tests. This stress is not measured during the shearing, and is deduced from the initial load applied on the specimen during the consolidation (Eq 3-43). The vertical load measured during the consolidation is corrected based on the zero correction presented in the previous paragraph.

\[ \sigma_V = \frac{\text{VL}_{\text{cons.}} + \Delta \text{VL}_{\text{zero}}}{A_0} = \text{constant during the test} \]  
\[ (\text{Eq. 3-43}) \]

where \( \sigma_V \) is total vertical stress
- \( \text{VL}_{\text{cons.}} \) is the measured vertical load applied during consolidation
- \( \Delta h_{\text{cons.}} \) is the deformation observed during consolidation
- \( A_0 \) is the cross-sectional area of the specimen
- \( \Delta \text{VL}_{\text{zero}} \) is the vertical zero correction

The effective vertical stress is calculated by dividing the measured vertical load corrected with the zero correction, by the surface area of the specimen (Eq 3-44).

\[ \sigma_V' = \frac{\text{VL} + \Delta \text{VL}_{\text{zero}}}{A_0} \]  
\[ (\text{Eq. 3-44}) \]

where \( \sigma_V' \) is the vertical effective stress
- VL is the measured vertical load
- \( A_0 \) is the cross-sectional area of the specimen
IV. Cyclic Direct Simple Shear Test

\[ \Delta V_{L_{\text{zero}}} \text{ is the vertical zero correction} \]

Assuming the total vertical stress stays constant during the test, the excess pore pressure \( u \) is calculated by the difference between the total vertical stress and the effective vertical stress (Eq 3-45).

\[ u = \sigma_v - \sigma_v' = \frac{V_L + \Delta V_{L_{\text{zero}}}}{A_0} - \frac{V_L}{A_0} = \frac{V_{\text{cons}} - V_L}{A_0} \]  
\[ \text{(Eq. 3-45)} \]

where \( u \) is the deduced pore pressure
\( V_L \) is the measured vertical load
\( A_0 \) is the cross-sectional area of the specimen
\( V_{\text{cons}} \) is the vertical load applied during consolidation
\( \Delta V_{L_{\text{zero}}} \) is the vertical zero correction

C. Post processing

Similarly to the cyclic triaxial tests, the specimen degradation is analysed based on the evolution of the hysteresis loops of the horizontal shear stress, the horizontal shear strain, and the build-up of pore pressure at the end of each cycle. The same parameters as for the triaxial tests are used to characterise each hysteresis loop: the secant shear modulus \( G_s \), the dissipated energy \( W_n \), and the accumulated dissipated energy \( W_{\text{tot}} \). The build-up of pore pressure is characterised by the degree of liquefaction \( d_{\text{liq}} \). The different equations used to calculate these parameters are reminded below:

\[ G_s = \frac{\tau_{\text{max}}}{\gamma_{\text{max}}} \times 100 \]  
\[ \text{(Eq. 3-46)} \]

\[ W_n = \text{surface of the hysteresis loops } \gamma - \tau \]  
\[ \text{(Eq. 3-47)} \]

\[ W_{\text{tot}} = \sum_{i=1}^{n} W_i \]  
\[ \text{(Eq. 3-48)} \]

\[ d_{\text{liq}} = \frac{u_{\text{end of n-th cycle}}}{\sigma_v} \]  
\[ \text{(Eq. 3-49)} \]

where \( \tau_{\text{max}} \) and \( \tau_{\text{min}} \) are the maximum and minimum horizontal shear stresses measured during the \( n^{\text{th}} \) cycle
\( \gamma_{\text{max}} \) and \( \gamma_{\text{min}} \) are the maximum and minimum horizontal shear strains measured during the \( n^{\text{th}} \) cycle
\( u_{\text{end of n-th cycle}} \) is the deduced pore pressure at the end of the \( n^{\text{th}} \) cycle
\( \sigma_v \) is the axial consolidation stress
IV.3 Cyclic Simple Shear test results

IV.3.1 Introduction

Sixty-three cyclic strain-controlled cyclic tests\(^5\) were performed on the Brusselian sand. The purpose of these tests was principally to investigate the influence of the shear strain amplitude \(\gamma_a\), the relative density \(D_r\), and the initial effective vertical normal stress \(\sigma_{v'}\) (or the total vertical normal stress \(\sigma_v\)) on the sand degradation. The ranges covered by these parameters are the following:

- a range of shear strains \(\gamma_a\) from 0.25\% to 9\%
- a range of relative densities \(D_r\) from 75\% to 85\%, and
- a range of initial effective vertical normal stress \(\sigma_{v'}\) from 75kPa to 200kPa.

Table 3-2 summarizes the choices of parameters for the different tests performed, and the number of tests performed with the same choice of parameters in order to investigate the influence of the experimental setup on the test results.

<table>
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<tr>
<th>Relative Density (D_r = 85%)</th>
<th>(\gamma_a) [%]</th>
<th>0.25</th>
<th>0.50</th>
<th>0.75</th>
<th>1</th>
<th>1.25</th>
<th>1.5</th>
<th>3</th>
<th>5.25</th>
<th>7</th>
<th>9</th>
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<tr>
<td>(P_0' = 75\text{kPa})</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(P_0' = 200\text{kPa})</td>
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<td>1</td>
<td>5</td>
<td>3</td>
<td>3</td>
<td>7</td>
<td>8</td>
<td>4</td>
<td>1</td>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Relative Density (D_r = 75%)</th>
<th>(\gamma_a) [%]</th>
<th>0.25</th>
<th>0.50</th>
<th>0.75</th>
<th>1</th>
<th>1.25</th>
<th>1.5</th>
<th>3</th>
<th>5.25</th>
<th>7</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>(P_0' = 75\text{kPa})</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
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<tr>
<td>(P_0' = 200\text{kPa})</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3-4: Summary of cyclic direct simple shear tests

The first paragraph describes in detail the results of a typical cyclic direct simple shear test. The next paragraph analyses the influence of the experimental setup on the test results (repeatability, specimen preparation, preshearing, …). The last paragraph compares the test results and proposes some explanations of the soil degradation during cyclic direct simple shear tests.

IV.3.2 Typical result of a strain controlled cyclic simple shear test

A. Introduction

The purpose of the following paragraphs is to present detailed results of a cyclic direct simple shear test. In the first part, the soil behaviour is described as a function of time. The second part discusses aspects of the hysteresis curve and of the stress path. In the last paragraph, the degradation is presented as a function of the number of cycles and as a function of the dissipated energy.

\(^5\) A summary of the results of the tests is available in the appendixes on the enclosed CD-rom.
The initial characteristics of the cyclic test are:

- Initial void ratio = 0.618 (Dr=85%)
- Initial height = 15.88mm and area =35cm²
- Consolidation axial stress = 202 kPa
- Vertical deformation during consolidation = 1.92%
- Void ratio after consolidation = 0.587 (Dr=90%)
- Degree of saturation measured after the test completion = 102%
- Amplitude of the cyclic shear strain = 3%
- Frequency of the cyclic shear strain = 0.0005Hz

B. Soil Behaviour as a function of time

During the direct simple shear test, the four following parameters are measured continuously:

- The axial effective normal stress $\sigma_V$
- The axial normal strain $\varepsilon_V$
- The horizontal shear stress $\tau_H$
- The horizontal shear strain $\gamma_H$

The strain values are controlled during the test. The control system of the vertical motor maintains the specimen height constant during the test. The allowed variation is 0.5µm (300 times smaller than the mean diameter $d_{50}$ of the sand). The horizontal shear strain follows a sine function with a period of 2000 seconds and an amplitude of 3%.

Fig. 3-49 compares the measurements performed during the first three cycles. The shape of the shear stress curve is similar to the shear stress curve observed during the cyclic triaxial tests, except that in the simple shear test the curve is symmetric. The pore pressure observed in the simple shear test is also similar to that of the triaxial test: it has a double frequency. During each cycle, two periods of dilatation and two periods of contraction are observed. The end of each dilatation phase corresponds nearly with the maximum shear strain, whereas the end of each contraction phase corresponds with the inflexion point of the shear stress curve. Except for the first cycle, the moment when the pore pressure exceeds the maximum pore pressure observed during the previous cycle corresponds to the zero shear stress.
C. Analysis of the hysteresis and of the stress path

An analysis of the hysteresis loops of the horizontal shear stress as a function of shear strain (Fig. 3-50) shows two fixed points. The strains and stresses corresponding to those points are:

- shear strain $\gamma_H = 1.88\%$ and shear stress $\tau_H = -0.17\text{kPa}$
- shear strain $\gamma_H = -2.02\%$ and shear stress $\tau_H = 0.07\text{kPa}$

This phenomenon is observed during all the cyclic simple shear tests performed in this investigation and all the cyclic triaxial tests. All results found in the literature present the same particularity. In each case, these fixed points roughly correspond to zero shear stress and occur always when the specimen has a contractive behaviour. In the previous paragraph, it was shown that the zero shear stress corresponds to the maximum pore pressure observed during the previous cycle. Therefore, this point and the fixed point may be assumed to occur simultaneously. However, a detailed analysis (Fig. 3-51) shows that the fixed point usually precedes the point where the pore pressure is equal to the maximum pore pressure observed during the previous cycle.

The points of maximum pore pressure during this cycle are also shown on Fig. 3-51. These points correspond to the inflexion points of each curve of the hysteresis. That means more energy has to be spent to shear the specimen when it has a dilative behaviour than a contractive one. The granular explanation is that it is more difficult to move the grain out of the soil matrix than push it in the hole where it used to be.
Fig. 3-50: Hysteresis loops during a simple shear test.

Fig. 3-51: Hysteresis loops of the 3rd cycle and comparison with the different states of the pore pressure

It is not possible to draw a conventional stress path (deviator q versus mean effective stress P' as for triaxial test) for the cyclic simple shear test due to the horizontal normal stress being unknown (strain and not stress boundary condition). As an alternative, the stress path is defined as the shear stress versus the effective axial stress, $\sigma_v' - \tau_H$ (Fig. 3-52). During each dilative phase, the stress path follows a straight line. The slope of this line stays constant during the test (33°), but the intersection of this line with the horizontal axis moves progressively to 0. This movement could be due to the influence of the horizontal normal stress in the specimen.

Indeed, Dyvik (1983) showed that, during a cyclic test, the K factor ($=\sigma_h'/\sigma_v'$) increases progressively from $k_0$ to 1. Based on those results, we could expect that the stress path of the first cycles would shift to the left in a graph of $p'-q$ and that, during
the last cycles, the conventional stress path p’-q and the stress path σv’-τH correspond. Therefore, it would be possible to obtain a unique straight line for all the cycles in a regular stress path p’-q.

![Stress path](image)

**Fig. 3-52: “Stress path” during a cyclic simple shear test.**

**D. Sample degradation**

As for the cyclic triaxial test, two parameters are used to characterise the evolution of the sample degradation:

- number of cycles, N
- dissipated energy, W

This paragraph presents the results of the tests in function of these two parameters and introduces some analytical equations to describe the curves.

**D.1 Sample degradation as a function of cycle number.**

Fig. 3-53 presents the secant shear modulus as a function of number of cycles. During the first 8 cycles, this curve follows a semi-logarithmic law that can be described with the following analytical expression:

\[
G_{s_n} = \frac{\Delta \tau_{\text{max}}}{\Delta \gamma_{\text{max}}} = G_{s_1} \left(1 - \log_{10}(N^\alpha)\right) \\
\text{(Eq. 3-50)}
\]

where \(G_{s_n}\) is the secant shear modulus at the \(n^{th}\) cycle

- \(G_{s_1}\) is the secant shear modulus at the 1\(^{st}\) cycle
- \(N\) is the cycle number
- \(\alpha\) is an empirical parameter

This equation is identical to the expression used to describe the degradation of secant shear modulus during a cyclic triaxial test.
The values of the parameters for the analysed test are:

\[ G_s = 2.9 \text{MPa} \quad \text{(measured value } = 2.96 \text{MPa}) \]
\[ \alpha = 1 \]

As a function of the parameter \( \alpha \), it is possible to calculate the number of cycles needed to reach a secant shear modulus equal to 0 using the following expression:

\[ N_{G_s=0} = \frac{1}{\alpha} \]

(Eq. 3-51)

The result of the regression is also drawn on Fig. 3-53. The regression is very good for the first cycles, but when the secant shear modulus becomes small the measurements become slowly tangent to the horizontal axis. Using this equation, 10 cycles are required to obtain a secant shear modulus equal to 0.

Similarly to the cyclic triaxial test, different definitions can be considered to calculate the degree of liquefaction during each cycle of a cyclic direct simple shear test. For example, the calculation can be function of the average deduced pore pressure, the maximum deduced pore pressure or other values of the pore pressure characteristic of particular points of the cycle (maximum strain, minimum strain, …). The evolution of the degree of liquefaction for six possible definitions are shown on Fig. 3-54. Based on the selected definition, large differences in the evolution are observed but the different degrees of liquefaction grows progressively to be all equal to 1 when the soil is completely liquefied.

In the present research, the degree of liquefaction \( d_{\text{liq}} \) is representative of the deduced pore pressure and the effective stresses at the end of the cycle. This degree of liquefaction correspond more or less with the maximum possible degree of liquefaction.
Fig. 3-54: Comparison between the different possible definitions of the degree of liquefaction during a cyclic direct simple shear test.

Fig. 3-55 presents the selected degree of liquefaction as a function of the number of cycles. Similarly to the cyclic triaxial tests, the same equation can be used to find the number of cycles needed to reach the liquefaction ($d_{liq}=1$):

$$d_{liq} = \frac{\Delta u_{\text{end of the n\text{th} cycle}}}{\sigma_v \text{consolidation}} = d_{liq1} \cdot \left(1 + \frac{D \cdot (N-1)}{1+E \cdot (N-1)}\right)$$  \hspace{1cm} (Eq. 3-52)

where $d_{liq\ n}$ is the mobilised liquefaction pore pressure at the $N^{th}$ cycle
$d_{liq\ 1}$ is the mobilised liquefaction pore pressure at the 1$^{st}$ cycle
$D$ and $E$ are empirical parameters

The values of these parameters for the analysed test are $d_{liq\ 1} = 0.87$ (measured value=0.87), $D = 0.144$ and $E = 0.981$. 
IV. Cyclic Direct Simple Shear Test

Fig. 3-55: Build-up of the pore pressure during cyclic direct simple shear test.

The result of the regression is presented on Fig. 3-55. With this equation, the number of cycles needed to obtain the liquefaction \( (d_{\text{liq}} = 1) \) is 9.5. This value is comparable with the value found with the model for describing the degradation of the secant shear modulus.

\[
d_{\text{liq n}} = \frac{u_{\text{end of n cycle}}}{P_c} = d_{\text{liq}} + \frac{D_e(N-1)}{1+E_e(N-1)}
\]

\[
\lambda = \frac{1}{100 - d_{\text{liq}}}
\]

\[
\begin{align*}
\text{Initial void ratio} &= 0.636 \\
\text{Degree of saturation} &= 100 \% \\
\text{Total axial stress} &= 202 \text{ kPa} \\
\text{Final void ratio} &= 0.605 \\
\text{Amplitude of the cyclic strain} &= 3\% \\
\text{Frequency of the cyclic strain} &= 0.0005 \text{Hz}
\end{align*}
\]

D.2 Sample degradation as a function of the dissipated energy

Fig. 3-56 and Fig. 3-57 present the degradation of the secant shear modulus \( G_{s n} \) and the build-up of the pore pressure \( d_{\text{liq}} \) as a function of the accumulated dissipated energy \( W_{\text{tot}} \).

Fig. 3-56: Degradation of the secant shear modulus as a function of the dissipated energy during cyclic direct simple shear test.
As observed in the cyclic triaxial tests, there exists a linear relationship between the secant shear modulus and the accumulated dissipated energy. This relationship can be described using the same following equation:

\[ G_{sn} = G_{s1} \left( 1 - \beta \left( \frac{W_{tot \ n}}{W_{tot \ 1}} - 1 \right) \right) \]  

(Eq. 3-53)

where \( G_{sn} \) is the secant shear modulus of the \( n \)th cycle

\( G_{s1} \) is the secant shear modulus of the 1st cycle

\( W_{tot \ n} \) is the energy dissipated during the \( n \) first cycles

\( W_{tot \ 1} \) is the energy dissipated during the 1st cycle

\( \beta \) is an empirical parameter.

The values of those parameters for the analysed test are:

\( G_{s1} = 2.96 \text{ MPa} \)  
(measured value = 2.96 MPa)

\( W_{tot \ 1} = 0.0031 \text{ MJ/m}^3 \)  
(measured value = 0.0032 MJ/m\(^3\))

\( \beta = 0.545 \)

This phenomenon was observed during the cyclic simple shear tests where the specimen had a strong dilative behaviour; i.e. for the tests where the cyclic strain amplitude was over 1.5\%, or when the specimen had a dilative behaviour on a part of the stress path.

The following semi-logarithmic equation is used to represent the build-up of the pore pressure.

\[ d_{liq \ n} = \frac{\Delta u}{\sigma_{v \ 0}} = d_{liq \ 1} + LOG_{10} \left( \frac{W_n}{W_1} \right)^{\lambda} \]  

(Eq. 3-54)

where \( d_{liq \ n} \) is the mobilised liquefaction pore pressure after the \( n \)th cycle

\( d_{liq \ 1} \) is the mobilised liquefaction pore pressure after the 1st cycle

\( \lambda \) is an empirical parameter.
The values of the parameters corresponding to the analysed test are:

\[ d_{\text{liq}1} = 0.86 \quad \text{(measured value = 0.87)} \]
\[ W_{\text{tot}} = 0.0029 \text{ MJ/m}^3 \quad \text{(measured value = 0.0032 MJ/m}^3) \]
\[ \lambda = 0.328 \]

Fig. 3-58 presents the secant shear modulus as a function of the energy dissipated during each cycle. An exponential analytical equation can be used to describe this relationship:

\[ G_{s_n} = A \cdot W_n^B \]  
\[ \text{(Eq. 3-55)} \]

where \( G_{s_n} \) is the secant shear modulus of the \( n^{th} \) cycle
\( W_n \) is the energy dissipated during the \( n^{th} \) cycle
\( A \) and \( B \) are empirical parameters.

The values of the parameters \( A \) and \( B \) are respectively 300 and 0.8. This equation implies there exists a unique relationship between the shape of the hysteresis loop and the minimum and maximum values of the shear stress. The hysteresis loop is completely described by the value of the secant shear modulus.

Fig. 3-58: Degradation of the secant shear modulus as a function of the energy dissipated during each cycle during cyclic direct simple shear test.

IV.3.3 Influence of the experimental setup on the cyclic simple shear tests results

A. Introduction

The next coming paragraphs investigate the influence of some parameters of the experimental procedure on the results of the cyclic direct simple shear tests. The first paragraph discusses the repeatability of the tests, whereas the next paragraphs analyse the influence of the specimen preparation method, the importance of the saturation and the impact of preshearing.
**B. Repeatability**

In order to investigate the repeatability of the cyclic direct simple shear test, four cyclic tests were performed with the same parameters (Fig. 3-59):

- Strain amplitude = 3%
- Relative density = 87.5%
- Axial consolidation stress = 200 kPa

The tests results demonstrated a low repeatability of the cyclic DSS tests on dense sand. Indeed, a variation of about 20% is observed on Fig. 3-59.

In order to analyse the influence of the consolidation time on the tests repeatability, tests 3 and 4 (Fig. 3-59) were consolidated during a period of 26 hours (instead of 6 hours). Although, in this case, the results show a smaller resistance when the specimen is consolidated for longer, other tests performed with different strain amplitudes (0.75%, 1.5% and 5.25%) showed a consolidation time of 6 hours is long enough not to influence the results.

![Fig. 3-59: Repeatability of the cyclic simple shear test.](image)

**C. Influence of sample preparation**

Another sample preparation was tested on two specimens (Fig. 3-60). The specimens were prepared in three layers. Each layer was tamped to the relative density calculated according to the undercompaction method (Ladd, 1978). With this new preparation, the repeatability of the test was not better. However, a strong difference was observed in the cyclic resistance. The number of cycles to reach a complete degradation were respectively 5 and 6 for these two tests, instead of 18 for the regular test. It was decided, however, to continue with the previous sample preparation technique (sample compacting in one layer) because it is the generally accepted sample preparation for sand in a simple shear test.
**D. Influence of the saturation**

Two tests were performed on unsaturated specimens (Fig. 3-61). The degree of saturation revealed a strong influence on the cyclic resistance of the sand specimen. Probably, the presence of two phases in the specimen creates suction that artificially increase the effective stresses. A lot of care (good initial water content, use of CO2,…) was spent in the subsequent tests to assure a perfect saturation of the specimen. The degree of saturation was determined at the end of each test by measuring the final water content.
E. Influence of preshearing

Preshearing is often used during the consolidation of a sand specimen before the direct simple shear test. It consists in applying a cyclic shear stress during the specimen consolidation. The stress amplitude depends on the applied vertical stress during the consolidation. In the tests performed in this research, the stress amplitude was about 20kPa and 400 cycles with a frequency of 0.1Hz were applied. During this operation, the specimen condition is drained.

The objective is twofold. At one hand, the preshearing tries to reproduce the history of the specimen and to cancel the influence of the specimen remoulding. At the other hand, the preshearing aims to homogenise the specimen. Indeed, Dyvik (1981) pointed out that some areas close to the membrane are not loaded during the application of the vertical consolidation load on the specimen, and that the consolidation stress is concentrated in the middle of the specimen.

Two cyclic direct simple shear tests were carried out using the preshearing technique during the consolidation (Fig. 3-62). The main objective was to find out if preshearing improved the repeatability of the cyclic DSS tests. The preshearing tends to increase lightly the sand resistance during the cyclic test, probably consecutive to a grain reorganisation during the preshearing. However, preshearing does not seem to improve the repeatability of the cyclic tests. Therefore, it was decided to continue to perform the tests without preshearing.

F. Conclusion

A large number of cyclic tests were performed to investigate the repeatability of the cyclic simple shear test on dense sand. The results of this investigation showed that a perfect repeatability (like that observed with the triaxial tests) is not possible during a simple shear test on dense sand. A possible explanation could be that the small size of
the specimen induces a very important sensitivity of the sample preparation. Some modifications in the sample preparation (saturation, preshearing and other tamping method) were investigated in order to improve this repeatability. However, no significant results were observed. Therefore, it was decided to perform a large number of tests and to validate the results of each test based on the comparison with the results of the other tests.

**IV.3.4 Comparison of the different tests**

This section presents the comparison of Sixty-three cyclic simple shear tests. These tests investigate the influence of the strain amplitude, the relative density and the consolidation stress, on the cyclic resistance of the Brusselian sand. 10 strain amplitudes (0.25%, 0.5%, 0.75%, 1%, 1.25%, 1.5%, 3%, 5.25%, 7% and 9%), 2 axial consolidation stresses (75kPa and 200 kPa) and 2 relative densities (70% and 86%) were investigated.

**A. Influence of the strain amplitude**

Fig. 3-63 compares the evolution of the shear stress and the effective axial stress as a function of the shear strain during the first cycle for different strain amplitudes.

Fig. 3-64 compares the degradation of the secant shear modulus and the build-up of the pore pressure for different shear amplitudes (Dr=86% and \(\sigma_v=200\text{kPa}\)). In the paragraph IV.3.2, it was proposed to describe the secant shear modulus degradation with a semi-logarithmic expression (Eq 3-50). The analysis of Fig. 3-64 validates the utilisation of this equation, except for the test where the amplitude was 0.25%. Similarly to the triaxial tests, the difference of behaviour at the beginning of the low amplitude tests is probably due to the grains reorganisation consecutive to the transition from consolidation to shearing.

For large strain amplitude (higher than 3%), the initial secant shear modulus and the sample degradation seem to become independent of strain amplitude. This phenomenon is probably the consequence of the large dilative phases observed for large strain amplitudes. During large strain amplitude tests, the grain reorganization consecutive to the dilative phases reaches an equilibrium, and the potential of tendency of volume reduction during the unloading remains constant.

The build-up of the pore pressure during each test (Fig. 3-64-b) is similar to the build-up observed during the cyclic triaxial tests. While large dilation phases are observed during each cycle of large strain tests, a continuous increase of the pore pressure is observed at the end of each cycle. Similarly to triaxial tests, no equilibrium is observed where the dilative phases compensates the contraction phases. The larger the strain amplitude, the faster the build-up of pore pressure. However, for the same
reasons as explained above, it seems that for large strain amplitude, the pore pressure build-up becomes independent of the strain amplitude.

The degradation of the secant shear modulus is presented as a function of the accumulated dissipated energy (normalised by the energy dissipated during the first cycle) on Fig. 3-64c. In paragraph IV.3.2, a linear equation was used to describe the degradation of the secant shear modulus as a function of the accumulated dissipated energy. However, the analysis of Fig. 3-64c shows that this relationship can not be used for all shear strain amplitudes. While the curves of the tests with amplitude larger than 1.5% are linear, this can only be observed for a part of the curves of the tests with amplitude smaller than 1.5%. A detailed analysis of the results shows that the relationship becomes linear when a part of the stress path becomes tangent the steady states line; i.e. when the shape of the hysteresis loops becomes concave instead of convex.
Fig. 3-63: Comparison of the hysteresis loops, the effective vertical normal stress and the stress path of the first cycle (Dr=86% and $\sigma_v'=200$ kPa).
Fig. 3-64: Influence of the shear strain amplitude on the sand degradation during cyclic DDS tests (Dr=86% and σ_v=200 kPa): (a) secant shear modulus degradation, (b) pore pressure build-up; (c) relationship between normalised secant shear modulus and normalised accumulated dissipated energy
Fig. 3-65 compares the relationship between the secant shear modulus and the energy dissipated during each cycle for different shear strains. An exponential analytical equation was used to describe this relationship:

\[ G_{sn} = A . W_n^B \] (Eq. 3-56)

where \( G_{sn} \) is the secant shear modulus of the \( n^{th} \) cycle
\( W_n \) is the energy dissipated during the \( n^{th} \) cycle
\( A \) and \( B \) are empirical parameters.

A numerical analysis of parameters \( A \) and \( B \) shows that parameter \( B \) is constant for all cyclic direct simple shear DSS tests and that parameter \( A \) is only dependent on the shear strain amplitude (Fig. 3-66). An exponential relationship has been found between parameter \( A \) and the shear strain amplitude. The resulting equation (Eq 3-57) that links the secant shear modulus of cycle \( G_{sn} \), the energy dissipated during the corresponding cycle \( W_n \) and the strain amplitude \( \gamma_a \) during a cyclic DSS test is identical to the equation used to describe that relationship during triaxial tests.

\[ G_{sn} = 2000 \frac{W_n^{0.8}}{\gamma_a^{1.8}} \] (Eq. 3-57)

In conclusion, there exists an unique relationship between the development of the area of the hysteresis loop and the minimum and maximum values of the shear stress. The development of the area of the hysteresis loops is independent of the test type (DSS or Triaxial). Taken into account in addition the influence of the soil behaviour (contractive or dilative) on the curvature of loading and unloading stress-strain curve, it can be considered that the hysteresis loops are completely described by the value of the secant shear modulus.

![Graph showing comparison of energy dissipated and secant shear modulus](image-url)

Fig. 3-65: Comparison of the relationship between the energy dissipated during each cycle and the secant shear modulus during cyclic DDS tests
B. Influence of the relative density

In the range of high densities where the cyclic DSS tests were carried out, even if a lower cyclic resistance was observed on looser specimens, the relative density seemed to have a low influence on the soil degradation (Fig. 3-67). The same observation was made during the analysis of the triaxial tests: a small variation in the density of dense specimens (Dr > 75%) showed little influence on the soil degradation. However, the triaxial tests also pointed out that the relative density becomes predominant on the soil degradation during cyclic tests on looser soil. The results of DSS confirm the observations based on the triaxial test, but the range of relative densities is too small to deduce the influence of this parameter on the soil degradation.

C. Influence of the initial effective vertical stress

As for the triaxial test, the initial stress state influences strongly the initial secant shear modulus, but has less influence on the number of cycles required to liquefy the soil (Fig. 3-68). This influence seems to be more important for small strain amplitudes than for large strain amplitudes. This observation results from the dilative behaviour of the Brusselian sand. Although the triaxial test results showed that large dilation phases cancel the effects of the initial stress state, the soil degradation during DSS tests seems to be more dependent of the initial consolidation stress.
Fig. 3-67: Influence of relative density on degradation during cyclic DSS tests (σ₀' = 200 kPa): (a) secant shear modulus degradation, (b) pore pressure build-up; (c) relationship between normalised secant shear modulus and normalised accumulated dissipated energy.
Fig. 3-68: Influence of the initial effective vertical stress on degradation during cyclic DSS tests (Dr=85%): (a) secant shear modulus degradation, (b) pore pressure build-up; (c) relationship between normalised secant shear modulus and normalised accumulated dissipated energy.
IV.4 Conclusion

In the description of the experimental setup of the cyclic simple shear (DSS) tests presented in the previous paragraphs, the following principal characteristics can be pointed out:

- The tests were performed on the new NGI simple shear device, with a modified control system to allow cyclic strain controlled simple shear testing.

- All tests were constant volume tests. The total axial stress was assumed constant during the shearing, and the change of the axial stress needed to keep the height of the specimen constant was assumed equivalent to the excess pore pressure measured during an undrained test.

- Based on 20 cyclic simple shear tests investigating the repeatability of the cyclic simple shear test, it is difficult to obtain a perfect repeatability of the cyclic simple shear tests on a dense sand. Therefore, each parameter had to be investigated with a large number of tests in order to confirm the results of each test based on the results of the other tests.

The cyclic simple shear tests covered a large range of shear strain amplitudes (from 0.25% to 9%) for two axial consolidation stresses and two relative densities. The analysis of the test results led to the following observations:

- The comparison of the shear stress curve and the build-up of the pore pressure measured during a cyclic simple shear test shows the same characteristics as observed during the cyclic triaxial tests: (a) the pore pressure has a double frequency; (b) the maximum pore pressure corresponds to the inflexion point of the shear stress curve; (c) the minimum pore pressure corresponds to the maximum strain; and (d) the zero shear stress corresponds roughly to the maximum pore pressure observed during the previous cycle.

- The hysteresis of the shear stress versus the shear strain presents two fixed points. These points correspond to the zero shear stress.

- Due to the uncertainties in the horizontal normal stress, it is not possible to draw a conventional stress path (deviator q versus mean effective stress \(P'\) as for triaxial tests) for the cyclic simple shear test. As an alternative, the stress path was defined as the shear stress versus the effective axial stress. During each dilative phases, this stress path follows a straight line whose slope is independent of the number of cycles, of the shear strain amplitude, and of the consolidation axial stress. The intersection of this line with the horizontal axis moves progressively toward the origin. This movement could be explained by the evolution of the horizontal normal stress.
• The degradation of the secant shear modulus as a function of the number of cycles is described successfully with a semi-logarithmic equation. It was found that the relationship between the shear strain amplitude and the secant shear modulus of the first cycle $G_{s1}$ can be separated in two different semi-logarithmic parts. The first part (in the strain amplitude range from 0.25 to 1.5%) is characterised by a strong decrease of the initial secant shear modulus when the strain amplitude increases. On the other hand, in the second part, a very low (almost constant) decrease of the initial shear modulus is observed. The separation between these two parts corresponds to the shear strain when the soil enters in a dilative phase during a monotonic test.

The number of cycles needed to reach cyclic failure was described successfully by a negative exponential relationship of the shear strain amplitude. However, it seems that, for large amplitudes, the number of cycles needed to reach the cyclic failure becomes independent of the shear strain amplitude. This observation means that the influence of grain reorganisation during each dilative phase has limited impact on the soil degradation.

• The degradation of the secant shear modulus as a function of the accumulated dissipated energy is linear when the soil has a strong dilative behaviour.

• There is an exponential relationship between the secant shear modulus of a cycle and the energy dissipated during this cycle. This relationship was successfully described using the same equation as used for the triaxial to express the same relationship.

Based these observations, the difference between the dilative and the contractive behaviour seems to lead the evolution of all the parameters analysed during a cyclic simple shear test.
V CONCLUSION OF THE EXPERIMENTAL INVESTIGATION

The present chapter has presented the results of the experimental investigation of sand strength degradation of Brusselian sand under cyclic deformation. This degradation was analysed based on the results of two different kinds of tests: cyclic triaxial tests and cyclic simple shear tests.

The presentation of the experimental setup pointed out many differences between the sample preparation and the test conditions in these two types of test. Table 3-5 summarises the principal differences. The main difference is that of shearing: during triaxial tests, the specimen is loaded in the principal directions whereas, during DSS test, the principal directions are not constant and rotate during the shearing. An other limitation of the DSS test stems from the uncertainties in the horizontal stress, making impossible the determination of the stress state of the specimen.

<table>
<thead>
<tr>
<th>Cyclic Triaxial Test</th>
<th>Cyclic Direct Simple Shear Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample preparation by moist tamping.</td>
<td>Sample preparation by moist tamping.</td>
</tr>
<tr>
<td>Isotropic consolidation</td>
<td>Anisotropic consolidation</td>
</tr>
<tr>
<td>Total mean stress P constant</td>
<td>Total vertical normal stress constant</td>
</tr>
<tr>
<td>Pore pressure measured</td>
<td>Pore pressure deduced from measurements</td>
</tr>
<tr>
<td>Stress state fully determined</td>
<td>Principal stresses are unknown</td>
</tr>
<tr>
<td>No rotation of the principal axis</td>
<td>Rotation of the principal axis</td>
</tr>
<tr>
<td>Good repeatability</td>
<td>Low repeatability</td>
</tr>
</tbody>
</table>

Table 3-5: Comparison between the cyclic triaxial test and the cyclic DSS test

Despite these differences, the same qualitative behaviour was observed in both test types. The principal conclusion from the analysis of the tests results is the importance of the dilation behaviour of the sand during the cyclic test. The dilation phases have a strong influence on the shape of the hysteresis loops. When the soil dilates, the shape of the hysteresis loops changes from concave shape to convex shape (similar to a “banana” shape). Even if this modification of hysteresis loops tends to increase the corresponding secant shear modulus, the pore pressure at the end of each cycle continuously increases. During each large strain amplitude cyclic tests performed in this investigation, the specimen always reached liquefaction after a certain number of cycles. No equilibrium where the dilation phases would compensate the contraction phases was observed. Even more, small strain amplitude tests showed that the degradation rate is higher when the soil has dilative phases during the cycle, compared to when it only has a contractive behaviour during all the cycle.

When a dilative phase is observed during the cycles, the stress path of both test type presented the same “butterfly” shape. The analysis of the stress path showed that the “butterfly” shape is homothetic from cycle to cycle. It results from this observation that the stress ratio in a triaxial test is independent of the cycle number.
The relationship between the secant shear modulus of a cycle, the energy dissipated during this cycle, and the shear strain amplitude can be described with the same equation for the triaxial and the direct simple shear tests. This relationship implies the existence of a unique relationship between the development of the areas of the hysteresis and the minimum and maximum values of the shear stress. The development of the areas of the hysteresis loops is independent of the test type (DSS or Triaxial). Taken into account in addition the influence of the soil behaviour (contractive or dilative) on the curvature of loading and unloading stress-strain curve, it can be considered that the hysteresis loops are completely described by the value of the secant shear modulus.

Both the triaxial and DSS cyclic tests presented a linear relationship between the secant shear modulus of a cycle and the total energy dissipated since the beginning of the test. This observation was already made by Liang (1995), Kern (1996) and Figueroa (1994, 1997) during cyclic torsion tests performed on hollow cylinder. This linearity seems to be an important characteristic of dilative cohesionless materials.

It is difficult directly to compare quantitatively soil degradation during a cyclic triaxial test and a cyclic DSS test due to the many differences between these test types. The comparison will then be made possible in the next chapter using a constitutive model able to integrate the differences between the two shearing modes.