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EFFECT OF SHEARING RATE ON THE BEHAVIOR OF SAND

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**A thesis submitted to the Nanyang Technological University
in partial fulfillment of the requirement for the degree of
Master of Engineering**

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Summary

The stress-strain behavior of cohesionless soils is known to be affected significantly by time-effects such as shearing rate effect. Shearing rate may affect the shear strength and pore water pressure. However, the mechanisms behind the rate-effects are not well understood.

An experimental study of time-effect on cohesionless soils was carried out. A standard Ottawa Sand was used in the study. All the tests were carried out on isotropically consolidated specimens of sand. The tests conducted included, undrained and drained compression tests at various shearing rates. Different sample preparation methods were adopted to form samples with different structures. To identify whether chemical effect could be a factor, distilled water with and without silicic acid gel were used. To identify whether the aging effect could be a factor, tests with samples submerged for different aging time before shearing were carried out. The mechanisms for the strain rate effect were discussed. Cone penetration tests were also conducted to verify the chemical effect and aging effect on the resistance of penetration.

The tests results showed that shearing rate does affect the stress-strain behavior of Ottawa Sand significantly, including shearing strength, pore water pressure and so on. Chemical effect and aging effect were two important factors of shearing strain rate effect. The resistance of penetration of sand was also affected by chemical effect and aging effect.

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LIST OF SYMBOLS AND ABBREVIATIONS

CS	Critical state
CU	Coefficient of uniformity
CPT	Cone penetration test
CSD	Constant shear drained tests
CSL	Critical state line
CSRL	Constant stress ratio line
c_c	Compression index
c_u	Coefficient of uniformity
c_v	Coefficient of consolidation
DPVC	Digital pressure/ volume controller
D_{50}	Mean particle size
e	Void ratio
e_c	Void ratio after consolidation
e_f	Void ratio at the end of shearing
e_s	Inter-granular void ratio
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
FC	Fines content
G_s	Specific gravity
K	Coefficient of permeability
LVDT	Linear variable displacement transducer
M	Gradient of critical state line
M_F	Gradient of failure line
M_{PSL}	Gradient of peak stress line
M_{CSRL}	Gradient of constant stress ratio line
MT	Moist tamping method
m_v	Coefficient of compressibility
PT	Phase transformation state
PSL	Peak stress line
PTS	Phase transformation state
p'	Mean effective stress (kPa)
q	Deviator stress (kPa)

q_{peak}	Peak deviator stress
SEM	Scanning electron microscopy
SSL	Steady state line
WS	Water sedimentation methos
ϵ_a	Axial strain(%)
ϵ_v	Volumetric strain(%)
σ_3'	Effective confining pressure(kPa)
σ_{ij}'	Stress increment

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Soil is subjected to different loading rates and loading modes for different geotechnical problems. The loading rates can vary from very fast as in the case of pile driving to very slow as in the case of land reclamation. The stress-strain behavior of soils can be affected significantly by time-effect such as shearing rate effect and creep. If these time-effects are not taken into consideration, the geotechnical design can be affected and sometimes lead to unsafe design.

It has been reported that time-effect plays an important role in the test procedure and measured results for in-situ cone penetrometer test (CPT) in sand. The cone resistance in sand hydraulically placed granular fills and freshly densified deposits increases with time (Mitchell and Solymar, 1984; Ng et al., 1996). However, the mechanisms of this phenomenon are still not well understood.

When designing pile foundations, static design equations, pile driving formula, static loading tests and stress wave analyses can be employed to estimate the axial capacity of single pile. Both laboratory and field tests show that soil exhibits time-dependent behavior. An important experimental observation is that soil gains additional strength and stiffness with time due to time-dependent processes such as aging. Similarly, results show that the capacity of piles increases, to a certain extent, with time after installation due to time-dependent processes in the soil (Wendel, 1900; Long et al., 1999). However, most design methods do not take this into account. This implies that the design methods used today do not in general make use of the full capacity of piles. This time-effect on the bearing capacity of piles needs to be studied further.

In the analysis of the stability of soil slopes, static liquefaction and instability of sand under both undrained and drained conditions are often considered to be the triggering factors for the failure of the slope. Several studies showed that the shear strength of soil was affected by the strain rate when being sheared under both undrained and drained conditions (Casagrande and Shannon, 1948; Yamamuro and Lade, 1993, 1998). Since some of the study results are in contradiction with each other, further experimental studies are needed.

1.2 OBJECTIVES OF RESEARCH

The objectives of this study are:

1. To investigate the influence of strain rate on the stress-strain behavior of sand.
2. To study the possible chemical or physical effect of silicic acid gel on the strain rate effect of the stress-strain behavior of sand.
3. To study the fundamental mechanisms of the strain rate effect on sand.

1.3 SCOPE OF STUDY

The main focus of this research is to investigate the strain rate effect on the behavior of sands under drained and undrained conditions. For easy comparison, standard sand (Ottawa sand) will be used. Different sample preparation methods will be adopted to form samples with different structures. To identify whether chemical effect could be a factor, different pore fluids will be used. The mechanisms for the strain rate effect will also be discussed.

1.4 OUTLINE OF REPORT

This report consists of 6 chapters including introduction. A review of

previous work is presented in Chapter 2. Chapter 3 introduces the testing arrangement and methodologies adopted in the study. The effects of strain rate on the behavior of sand are presented in Chapter 4. The chemical effect of the silicic acid gel and its coupling with strain rate effect on the stress-strain behavior of sand is discussed in Chapter 5. Finally, the conclusions, practical implications and recommendations for future study are discussed in Chapter 6.

CHAPTER 2

Review of Previous Work

2.1 SHEARING RATE EFFECT ON THE BEHAVIOR OF COHESIONLESS SOILS

The effect of strain rate on the behavior of cohesionless soils has been studied by several researchers in the past.

Casagrande and Shannon (1948) were among the first to study the effect of strain rate on the strength of soil. They performed drained triaxial compression tests at low confining pressures (about 30 to 90 kPa) with times to failure varying from 0.3 to 2100s on dense sand. They found that the compressive strength increased by about 10%, but that the initial modulus of deformation did not change. They performed transient loading tests on samples of clay and dry sand, and showed that for clay soils, an increased strain rate from 1 to approximately 8000 percent per minute caused the strength to increase by about 50%, and the modulus of deformation to increase by 100%. For dry sand the strength increase was only about 10 to 15 percent and the modulus of deformation increase was about 30%.

Seed and Lundgren (1954) performed drained and undrained tests on dense sand at low confining pressures (about 200 kPa) with times to failure varying from 0.02 to 900s. They discovered that drained tests on saturated sands at high strain rates did not have sufficient time to adequately drain and consequently approached undrained conditions. This resulted in compressive strength increases of 15% to 20%, as well as increases in the initial modulus of deformation. The transient strengths of dense saturated samples were found to be about 40% greater than the strength under normal rates of loading. Although no pore pressures were measured, it was inferred that a strength increase of 25 to 30 percent was due to the pore pressure decrease caused by the tendency for dilation, leaving a strength increase of 10% to

15% due to the effect of rate of loading.

Nash and Dixon (1961) performed undrained tests on saturated samples of sand with pore pressure measurements at low consolidation pressures (about 10 kPa) using strain rates varying from 2%/min to 8000%/min. Their results showed decreasing maximum deviator stresses and lower modulus of deformation with increasing strain rates. In transient tests the pore pressure changes occurred in a smooth regular path, whereas in the slow tests the pore pressure changed by sudden jerks. It was reasoned that these jerks corresponded to fracturing of particles. It was suggested that there was no time for particles to fracture in the transient tests, and therefore there would be more tendency toward dilation under rapid loading conditions.

Whitman and Healy (1962) performed drained and undrained tests on dense and loose sands at low confining pressures (about 70 kPa) with times to failure varying from 0.005s to 300s. At high strain rates and drained conditions they found that the compressive strength increased about 10%. However, at lower strain rates and drained conditions, they found that the compressive strengths actually decreased slightly with increasing strain rate. They suggested that the increased strength was probably mostly due to an increased tendency to dilation which led to lower pore pressures at high rates of strain. They found that the peak compressive strength of dry sands and dense saturated sands showed no time dependence, while the strength of saturated loose sands was found to be dependent on the strain rate. For Ottawa sand, the compressive strength increased by approximately 40% between failure time of 5s and 0.025s. They also found that saturated specimens of loose Ottawa sand exhibited a well-defined yield point at a low strain, and the yield point stress was found to decrease by 25% with

decreasing failure time as shown in Fig.2.1.

Fig.2.1 Stress-strain curves for loose saturated Ottawa sand
(after Whitman and Healy, 1962)

Lee et al. (1969) performed drained and undrained tests on dense and loose sands at low to moderate confining pressures (about 100 to 1500 kPa) and axial strain rates ranging from 0.02 to 15000 percent per minute. These tests showed that an increase in strain rate produced an increase in strength, an increase in the initial tangent modulus and a decrease in the strain at failure. They found that an increase in strain rate from 0.1 to 10000 percent per minute produced about 7 percent increase in strength for loose sands at all confining pressures. However for dense sands, a similar increase in strain

rate caused about 20 percent strength increase for tests at 1500kPa confining pressure but only about 7 to 10 percent strength increase for tests at 100kPa confining pressure as shown in Fig.2.2.

Fig.2.2 Stress strain curves for normal and transient loading tests in loose and dense sand conditions (after Lee et al., 1969)

Drained and undrained triaxial compression tests on dense Cambria sand at a high confining pressure at 34MPa were also conducted by Yamamuro and Lade (1993) to study the effect of strain rate on the instability of granular soils. They found that as strain rate increased in drained conditions, friction-angles, maximum deviator stresses and axial strain to failure increased slightly, but volumetric compression decreased slightly. While as

strain rate increased in undrained conditions, maximum deviator stresses and axial strain to maximum deviator stresses increased, but pore pressures, friction-angles and effective confining pressure at failure decreased significantly as shown in Fig.2.3a, Fig.2.3b and Fig.2.3c.

Fig.2.3a

Fig.2.3b

Fig.2.3c

Fig.2.3 (a) Deviator stress. (b) Pore pressures. (c) Effective stress paths for undrained tests sheared at different strain rates. (after Yamamuro and Lade, 1993)

Lade et al (2009) observed that the time effect behavior in sand is quite different from that observed in clays and most other materials. Drained triaxial compression experiments have been performed with a constant effective confining pressure of 200 kPa on crushed coral sand to study time effects in granular materials.

The test series included experiments with specimens loaded at five different strain rates with a 256-fold ratio between the extreme rates, and they all produced the same stress-strain and volume change behavior, thus showing no effect of strain rate (Fig.2.4). Tests with sudden changes in strain rate from slow to fast and vice versa showed only very temporary effects before the stress-strain and volume change reverted to the common relations.

The above review indicates that although some of the results contradict with each other, most of the observations show that the undrained behavior

of sand is affected by strain rate. Hence, a series of triaxial tests were conducted with different strain rate under a confining pressure of 150 kPa to study the effects of strain rate on the behavior of sand.

Fig. 2.4 Comparison of stress-strain and volume change relations for drained triaxial compression tests performed with five different strain rates (after Lade et al, 1993)

2.2 AGING EFFECT OF SOIL

Different strain rates lead to different shearing time before failure of the soil. Therefore, aging may be one factor contributing to the strain rate effect. Aging effect has been studied by several researchers in the past (Mesri et al,

1990; Schmertmann, 1991; Kuwano, 1999). Some researchers indicated that there were mechanical processes that could lead to the aging effect, while others believed that it was the chemical processes that could have resulted in the aging phenomenon.

2.2.1 Aging Due To Mechanical Processes

Mesri et al. (1990) proposed that aging effects in sands are due to an increased frictional resistance that develops during secondary compression. According to the authors, this increased resistance does not occur solely from the change in density that occurs during drained secondary compression. Rather, it is due to a continued rearrangement of particles resulting in increased macro interlocking of particles and micro interlocking of surface roughness. These mechanisms cause an increase in both stiffness and horizontal effective stress, which, in turn, cause increases in properties such as the small strain shear modulus and the cone penetrometer resistance.

Mesri et al. (1990) argued against cementation as mechanisms by using the drained triaxial test data from Daramola (1980). Specimens of Ham River sand that underwent secondary compression for up to 152 days had an increased modulus up to approximately 3% axial strain. According to the authors, any mechanism involving bonding or cementation would result in a brittle stress-strain response and any property increase would be destroyed at strains less than 3%. Thus it was postulated that another less brittle mechanism must be responsible for the increase in stiffness, and the frictional mechanism was proposed. This is supported by the existence of locked sands (Richards and Barton, 1999), which exhibit a tensile strength even without the presence of binding cement.

Although no increase in stiffness was detected when glass balls were loaded isotropically, sand has been found to increase in strength and stiffness under isotropic stress conditions (Daramola, 1980). These increases develop even under isotropic confinement because the angular particles can lock together in an anisotropic fabric. It has been shown that more angular particles produce materials more susceptible to creeping deformations.

Schmertmann (1991) stated that aging effects are the result of mechanical mechanisms as opposed to chemical bonding or cementation. Schmertmann (1991) considered the mechanisms responsible to be the same for both sands and clays and believed that aging effects in sands are the result of the interdependent effects of dispersive particle movements, increased interlocking, and internal stress arching. Schmertmann (1991) also believes that, during aging, small particle movements occur which lead to a more stable soil structure. These movements result in an increase in stiffness and a decrease in the compressibility of the soil. The dispersive particle movements would result in an increased micro interlocking at the particle contacts, which might also contribute to the property changes during aging. Even smooth particles at small scales have an irregular and rough shape. Small particle movements or grain slippage could cause the two particles to fit together in a more stable arrangement, thus providing an increased resistance to shear. The strength due to the micro interlocking would be a function of the normal stress at the contacts and would cause an increase in the frictional component of strength. Schmertmann (1991) also hypothesized that aging effects might be caused by uneven stress distributions within the soil skeleton. There are stiffer elements, or zones, within the soil that carry more stresses than adjacent weaker elements, creating an arching effect which may increase the soil strength.

Isotropic compression tests carried out by Kuwano (1999) showed that radial creep strains were greater than axial strains in soils with angular particles than in soils with rounded particles due to a more anisotropic initial fabric. Angular particles can result in longer duration of creep and a greater aging effect since they have a larger range of stable contacts and the particles can interlock. As spherical particles rearrange more easily than elongated ones, rounder particles initially creep at a higher rate before settling into a stable state. Hence, any aging effect on rounded particles tends to disappear quickly when the soil is subjected to new stress state.

2.2.2 Aging Due To Chemical Processes

Chemical processes maybe a cause of time-dependent strengthening. Historically, the most widespread theory used to explain time-dependent strengthening effects in sand has involved inter-particle bonding. Terzaghi originally referred to “bond strength” in connection with the presence of a quasi-pre-consolidation pressure in the field (Schmertman, 1991). Generally, this mechanism has been thought of as type of cementation, which would increase the cohesion of a soil without affecting its friction angle.

Denisov and Reltov (1961) showed that quartz sand grains adhered to a glass plate over time. They placed individual sand grains on a vibrating quartz or glass plate and measured the force necessary to move the grains as shown in Figure.2.5 and Figure.2.6. The dry grains were allowed to sit on the plate for varying times and then the plate was submerged, also for varying times, before vibrating began. It was found that the force required moving the sand grains continued to increase up to about 15 days of immersion in water. The cementating agent was thought to be silica-acid gel, which has an amorphous structure and would form a precipitate at particle contacts (Mitchell and Solymer, 1984)? The increased strength was derived

from crystal overgrowths caused by pressure solution and compaction.

Fig.2.5 Apparatus for determination of adhesion of sand grains
(after Denisov and Reltov, 1961)

Fig 2.6 Variations with the duration of the effect of water and
period of contact between sand grains and a quartz plate
(after Denisov and Reltov, 1961)

Strong evidence of a chemical mechanism being responsible for some aging was obtained by Joshi et al. (1995). A laboratory study was performed

specifically to study the effect of time on the penetration resistance. The influences of both sand type and pore fluid composition on the magnitude of aging effects were investigated. Two different sands were tested: a local river sand and Beaufort Sea sand. The sands were tested dry and in distilled and seawater.

The sand was placed in 36 cm diameter by 37 cm high PVC cells and subjected to a static vertical stress of 100 kPa. Each specimen was prepared by vibrating the specimen under the static load until the desired density was achieved. After loading, the specimens were aged for two years, and values of penetration resistance were obtained at various times in each specimen. Penetration tests in each specimen were performed with a series of four, 1 cm diameter penetrometer so that redundant data could be obtained at each time. Aging effects were observed in all cases. A typical plot of load vs. displacement at various times for one of the penetrometer is shown in Figure 2.7 and appears to clearly show an aging effect.

Fig.2.7 Load-displacement curves at various times for river sand in sea water.
(after Joshi et al., 1995)

In addition to the penetration tests, detailed mineralogical studies by Joshi et al. (1995) using X-ray diffraction and electron microscopy showed the presence of precipitates on the grains of the submerged samples after aging. For the samples of River sand submerged in distilled water, the composition of the precipitates was found to be calcium and possibly silica. In the case of the samples submerged in seawater, sodium and chlorine were also found as precipitates. Because aging was observed in dry sand, the authors concluded that, for sands in the dry state, aging effects were caused by rearrangement of particles. In the submerged state, it was hypothesized that the aging effects were the result of a combination of rearrangement of particles and precipitation of the soluble fractions of the sand.

However, there are several reported cases in which cementation was an unlikely mechanism of aging, at least in the short-term. For example, dry granular soils can show an increase in stiffness and strength with time (Joshi et al., 1995; Losert et al., 2000). Cementation in dry sand is unlikely, as moisture is required to drive solution and precipitation reactions involving silica or other cementation agents. Mesri et al. (1990) used the triaxial test data from Daramola (1980) to argue against a chemical mechanism responsible for aging effects in sands.

Fig. 2.8 shows the effects of aging on both the stiffness and shear strength of Ham River sand. Four consolidated drained triaxial tests were performed on samples with the same relative density and confining pressure (400 kPa), but consolidated for different periods of time (0, 10, 30 and 152 days) prior to the start of the triaxial tests. The results showed that the stiffness increased and the strain to failure decreased with increasing time of consolidation. Although increased values of modulus were observed, the strain at failure is

approximately 3%. Mesri et al. (1990) argued that this large strain would destroy any cementation, and therefore another less brittle mechanism must be responsible for the increase in stiffness.

Fig 2.8 Effects of aging on both the stiffness and shear strength of Ham River sand (after Daramola, 1980)

2.4 SUMMARY

This chapter presented a literature review of general stress-strain behavior and time dependent behavior of cohesionless soils. The behavior of cohesionless soils under different strain rates was examined.

The time-dependent strength behavior has been studied by many researchers in the past. Some researchers indicated that some mechanical

processes lead to time-dependent strengthening (Mesri et al, 1990; Schmertmann, 1991; Richards and Barton, 1999; Dowding and Hryciw, 1986; Howie et al., 2001), while others believed that some chemical processes result in this phenomenon (Schmertman, 1991; Joshi et al., 1995). However, what has to be studied further is the mechanism of the rate effect and the factors that control it. The objectives of this project are to clarify some of the doubts and investigate further that how the rate effect is affected by the chemical process.

CHAPTER 3

Testing Materials and Methodologies

3.1 INTRODUCTION

This chapter describes the material tested, the apparatus and program that was deployed and developed for the research and the methods used to prepare the specimens for testing.

3.2 MATERIAL TESTED

The material tested in this research was standard sand (Ottawa Sand). It will be called Ottawa sand in this thesis. The particle size distribution is shown in Fig. 3.1. As seen from Fig.3.1, the sand is uniformly graded and has a mean particle size, D_{50} , of approximately 0.3mm. Observations from the scanning electron microscopy also revealed that the individual grains of the sand are mainly sub-angular in shape, as shown in Fig.3.2. A summary of the basic properties of the granular fill material is presented in Table 3.1 below.

Fig.3.1 Particle size distribution curve for the material.

Fig.3.2 SEM photograph of Ottawa Sand

Table 3.1 Basic properties of Ottawa Sand

	Mean Size(mm)	Coefficient of Uniformity (CU)	Coefficient of Curvature (CC)	Maximum Void Ratio (e_{max})	Minimum Void Ratio (e_{min})	Specific Gravity(G_s)
Ottawa Sand	0.4	2	0.85	0.85	0.50	2.64

3.3 SPICEMEN PREPARATION

Specimens of diameter 100 mm and height 200 mm were used for all the triaxial tests. The specimens tested were reconstituted by using either the water sedimentation method (WS) or the moist tamping method (MT), as outlined and illustrated in sections 3.3.4 and 3.3.5.

3.3.1 Free-ends

For any laboratory test to be considered a true element test, it is

important to ensure that the measured material properties are virtually independent of the test boundary conditions and specimen dimensions. However, the use of standard rough porous stones in triaxial tests often results in the non-uniformity of both stress and strains due to the effect of end restraints (Rowe & Barden, 1964). To minimize the bedding restrictions and achieve as uniform strain distribution as possible during the test, free-ends with enlarged platens were used. The diameter of the enlarged top and base platens is 115mm and each platen has a 38mm diameter porous stone at the center.

The “free ends” consist of a layer of high vacuum grease and latex rubber disc (Fig 3.3). The thickness of the latex rubber disc was 0.6 to 0.7mm. The free-ends were pre-loaded under a dead weight of 30kg overnight. This was to remove any air bubbles that might be trapped within the lubrication layer. Free-ends prepared in this way have been proven to be effective on reducing end frictions (Chu & Lo, 1993).

Fig 3.3 Enlarged platen for free-end

3.3.2 Apparatus for specimen preparation

Fig. 3.4 shows the basic equipments that were used in the preparation of all the specimens. Before setting up the mould on the base pedestal, high vacuum grease was applied around the sides of the pedestal as well as the top platen to prevent leakage. One porous stone each of diameter 38mm was then placed on the base pedestal and top platen.

Fig 3.4 Apparatus used for specimen preparation

Fig 3.5 shows a typical example of how the three-part split mould is prepared before the placement of sand. The membrane was stretched taut against the mould by means of vacuum. The sand was then placed into the mould using either the WS or the MT method described. Fig 3.6 shows a specimen before assembling the triaxial cell. After the cell was assembled, the cell was filled with water and the saturation of the specimen begins (Fig 3.7). The saturation was carried out under a small water head of 0.5m.

Fig 3.5 Preparation of 3-part split mould prior to sand placement

Fig 3.6 A test specimen

Fig 3.7 Saturation of specimen

3.3.3 Water Sedimentation (WS) Method

A membrane was first stretched taut from the inside face of a three-part split mould, as shown in Fig.3.5 above. Some pre-weighed, oven-dried sand and some de-aired water were prepared (Fig.3.8). Great care was taken to ensure that the sand was continuously deposited below the water level, without causing any appreciable segregation of the material. Compaction to achieve denser specimens was done by tapping the side of the mould. The porous stone was then mounted onto the specimen upon the completion of sand placement. The top loading platen was then seated onto the porous stone and the membrane was secured to it with O-rings. A low suction pressure

Fig.3.8 Water sedimentation method

was applied to the specimen using a burette before the mould was dismantled. Direct measurement of the specimen dimensions was then made before the cell was assembled. The initial void ratio, e_0 was then predicted from the specimen dimensions taken. The assembled cell was then filled with water, after which a small cell pressure of approximately 10kPa to 20kPa was applied before the specimen was flushed with de-aired water for saturation under a small water head of about 0.5m (Fig.3.7).

3.3.4 Moist Tamping (MT) Method

The procedure for the preparation of specimen using the MT method was similar to that of the WS method, except that the oven-dried sand was now mixed with 5% of de-aired water prior to placement in the split mould. After mixing, the slightly moist sand was poured into mould in four lifts (Fig 3.9). For each lift, slight tamping was applied with a small, flat bottomed tamper. The amount of energy used in the tamping process was carefully controlled and different amount of tamping energy controlled by different times of tamping was used to achieve specimens of different density. Although the capillary suction effect between the particles was able to stabilize the

Fig.3.9 Moist tamping method

specimen temporarily, a low suction pressure was still applied to the specimen before removing the split mould. This was to ensure the stability of the specimen before saturation. The dimensions of the specimen were then taken and the initial void ratio, e_0 predicted. The cell was assembled and filled with water, after which a small cell pressure of approximately 10kPa to 20kPa was applied before the specimen was flushed with de-aired water for saturation under a small water head of about 0.5m.

3.4 TESTING ARRANGEMENT

The triaxial testing system used in this research project is shown schematically in Fig 3.10. The testing system comprises a computer, a triaxial machine, a hydraulic actuator, two Digital Pressure-Volume Controllers (DPVCs) and an eight channel data logger. The hydraulic actuator was used to control the load and the data-logger was used to convert analogue data to digital format. DPVC#1 was used to control the cell pressure.

1. Internal LVDTs 2. Pressure Transducers 3. Specimen 4. Load Cell 5. External LVDT
Fig.3.10 Schematic diagram of testing arrangement

DPVC#2 was used to measure the pore water pressure change in the specimen. The DPVCs were controlled by the computer via the standard IEEE-488 computer interface. The resolution was 1kPa for pressure and 1mm^3 for volumetric change. Data during the test from triaxial machine and DPVCs were recorded by the data-logger and stored in the computer. A submersible load cell of 10kN capacity was used to measure the vertical force. An external Linear Variable Displacement Transducer (LVDT) of 50mm travel

was mounted directly on top of the triaxial chamber for the purpose of measuring large strains.

3.5 VOID RATIO MEASUREMENT FOR SOILS

The accuracy in the determination of the void ratio of the sand specimen plays an important role in the analysis of the stress-strain behavior of sand. Thus it was necessary to adopt a method, which involved the least uncertainty and errors, for the determination of the void ratio of the specimen. The usual method used for the determination of void ratio for a specimen prepared by the MT method was to measure the dimensions of the specimen after consolidation by disassembling the triaxial cell. However, this method was found to be not only tedious but at the same time, cause unnecessary disturbances to the specimen. Alternative methods had been proposed by Konrad (1990) and Chu et al. (1993), but these methods measured the void ratio after saturation instead of void ratio after consolidation.

An alternative method was proposed by Verdugo and Ishihara (1996) to address this particular problem. The proposed method involved the usage of the water content of the specimen for the void ratio calculation. The void ratio of the specimen at the end of the test, e_f was computed using the following formula:

$$e_f = \frac{(M_{\text{porefluid}}/M_{\text{dry}})G_s}{G_s - 1} \quad (3.1)$$

Where e_f was the void ratio of the specimen at the end of shearing, $M_{\text{porefluid}}$ was the mass of pore fluid in the specimen, M_{dry} was the dry weight of the specimen and G_s the specific gravity of the Ottawa sand.

Eq.3.1 was applicable to an undrained test, where $e_c = e_f$. For other types of

tests, the void ratio of the specimen after consolidation, e could be computed using the volumetric change measured during shearing. Comparative studies using the conventional method and the proposed procedure had been carried out by Verdugo and Ishihara (1996). They suggested that the void ratio estimated using this proposed procedure was less error-prone. Therefore, this method was also adopted in the present study.

CHAPTER 4

Effect of Strain Rate on the Behavior of Sand

4.1 TESTING PROGRAMME

To investigate the influence of strain rate on the behavior of loose and medium loose sands, a series of undrained triaxial compression tests were conducted under different axial strain rates of 0.004mm/min, 0.04mm/min, 0.4mm/min and 4mm/min. At least two tests were carried out for each testing condition to ensure that the results obtained were consistent. The loose specimens were prepared using the MT method and the medium loose sand was prepared using both MT method and WS method. To eliminate the influence of cell pressure, all the specimens were consolidated isotropically under an effective confining pressure of 150 kPa before shearing. The testing conditions of the tests are summarized in Table 4.1 below.

Table.4.1 Summary of testing conditions

Test No.	Test Type	Preparation method	P_c' (kPa)	e_c	Strain rate (mm/min)	q_{peak} (kPa)	ϵ_a at q_{peak} (%)
unm01	Undrained	Water Sedimentation	150	0.698	0.04	—	—
unm02				0.693	0.4		
unl01	Undrained	Moist Tamping	150	0.714	0.004	163.89	1.45
unl02				0.710	0.04	123.01	1.04
unl03				0.713	0.4	96.47	0.86
unl04				0.711	4	84.21	0.51
dl01	Drained	Moist Tamping	150	0.704	0.04	—	—
dl02				0.705	0.4		
dl03				0.707	4		

4.2 EFFECT OF STRAIN RATE ON THE BEHAVIOUR OF VERY LOOSE SAND

4.2.1 Undrained Behavior

Fig. 4.1 presents the results of four undrained tests for very loose sand conducted using strain rates 0.004mm/min, 0.04mm/min, 0.4mm/min and 4mm/min. It can be seen clearly from Fig. 4.1a that all the tests exhibited a

flow type of behavior with the peak strength (q_{peak}) showing dependency on the strain rate. As can be seen from the graph, the lower the strain rate, the higher the value of q_{peak} .

Fig. 4.1a

Fig. 4.1b

Fig.4.1.Effect of strain rate on the undrained behavior

(a) Effective stress paths. (b) Stress-strain behavior. (c) Excess pore water pressure generated during shearing. (d) Logarithmic relationship of peak stress obtained in very loose sand with strain rate.

This is also seen in Fig. 4.1b where the stress-strain curves of the four undrained tests for very loose sand are presented that the peak value of deviator stress occurs at different axial strains in the four tests. Peak strength

(q_{peak}) of test unl04 occurs at the axial strain of 0.51%. When the strain rate is lower compared with test unl04 in test unl03, the axial strain of the peak strength (q_{peak}) becomes higher with 0.86%. When the strain rate is lower compared with test unl03 in test unl02, the axial strain of the peak strength (q_{peak}) becomes higher with 1.04%. When the strain rate is lower compared with test unl02 in test unl01, the axial strain of the peak strength (q_{peak}) becomes higher with 1.45%. The peak stress obtained from the test sheared at the lowest rate occurs at the largest axial strain.

The pore water pressures were measured at the top and bottom of the specimens in all the tests. They were used to compare the pore pressure responses in different tests, as shown in Fig. 4.1c. It is important to mention here that the "change in pore pressure", as plotted on vertical axis in Fig. 4.1c and subsequent figures in this thesis, refers to the excess pore pressure developed during shearing, i.e. the total pore pressure inside the specimen minus the applied back pressure. It can be seen from Fig. 4.1c, higher pore water pressures were generated when the specimen was sheared at a faster rate. This in turn leads to a lower peak stress (q_{peak}).

The peak deviator stresses obtained in the four deformation-controlled tests are plotted versus the axial strain rate in Fig. 4.1d. The peak deviator stress increased with the decrease in axial strain rate. The values of peak deviator stress in the four tests are given in Table 4.1. The increase in axial strain rate by 10 times (0.04mm/min), 100 times (0.4mm/min) and 1000 times (4mm/min) over the slowest strain rate (0.004mm/min) caused a decrease of 24.9%, 41.1% and 48.6% respectively in the peak deviator stress. The peak deviator stresses obtained in the four deformation-controlled tests are plotted versus the shearing time in Fig. 4.1e. The peak deviator stress increased with the decrease in shearing rate.

Similar results were observed before on Changi Sand in the thesis by Loke, (2003) which were shown in Fig.4.2. It can be seen that similar strain rate effect was observed on both Changi Sand and Ottawa Sand. The peak deviator stress (q_{peak}) was found to decrease with increasing strain rate. The peak stress obtained from the test sheared at the lowest rate occurs at the largest axial strain. Excess pore water pressure increased with increasing strain rate.

Fig.4.2a

Fig.4.2b

Fig.4.2c

Fig.4.2d

Fig.4.2.Effect of strain rate on the undrained behavior on Changi Sand
(a) Effective stress paths. (b) Stress-strain behavior. (c) Excess pore water pressure generated during shearing. (d) Logarithmic relationship of yield stress obtained in very loose sand with strain rate (after Loke,2003)

4.2.2 Drained Behavior

The results of the undrained tests on very loose sand shows a negative rate effect, which is opposite to what has been established by most of the

previous studies. To verify if the undrained results presented in section 4.2.1 reflected a real material behavior, a series of drained tests were also conducted using different strain rates. Three drained tests were conducted on loose specimens at three different strain rates of 0.04, 0.4 and 4mm/min. the experimental results are presented in Fig. 4.3.

Fig.4.3a

Fig.4.3b

Fig.4.3 Effect of strain rate on the drained behavior of very loose sand (a) Stress-strain behavior. (b) Volumetric strain vs. axial strain curves.

The results showed that the strain rate has some effect on the stress-strain behavior of the drained tests (Fig.4.3a). The peak deviator stress increased with the decrease in shearing rate. Meanwhile, it does affect the volumetric behavior of the specimens, as shown in Fig.4.3b. The specimen experiences more volumetric compression when sheared at a faster rate. This observation is consistent with the pore water pressure change behavior shown in Fig.4.3c. Similar results were observed before on Changi Sand in the thesis by Loke, (2003) which were shown in Fig.4.4.

Fig.4.4 Effect of strain rate on the drained behavior of very loose sand. Volumetric strain vs. axial strain curves. (after Loke, 2003)

4.3 DISCUSSION

A strain rate effect on the peak deviator stress (q_{peak}) was observed from the experiment results above. As can be seen from Fig.4.1a and 4.1b, the peak deviator stresses (q_{peak}) decreased as strain rate increased. It can also be observed that the peak deviator stresses (q_{peak}) in all the four undrained tests occurred at a small axial strain, ranging from 0.51% to 1.45%. It can be seen from Fig.4.1b that the axial strain at which q_{peak} is reached increases with decreasing strain rate. An increase in pore water pressure was

observed with increasing strain rate. In the drained tests, different volumetric strains are observed under different strain rates. The results show that the higher strain rate results in lower deviator stress of the drained tests and higher volumetric strains.

However, strain rate effect observed here had in fact been reported by a few researchers in the past (Nash and Dixon, 1961; Whitman and Healy, 1962) as well. Whitman and Healy (1962) in their studies on the effect of strain rate on the strength of dry and saturated sand had reported the observation of distinct peak stress decreased with increasing strain rate in saturated Ottawa sand ($e_c=0.65$ to 0.68) at low strains ($\approx 0.5\%$). A convincing explanation for this phenomenon and identification of the cause of it were not provided. However a few probable reasons had been suggested by the Whitman and Healy (1962): (1) the effect of flexible membrane on the pore water pressure generation within the specimen and (2) vibrations induced by rapid loading. Both were unlikely in the present tests because of the liquid rubber technique used to reduce membrane penetration and the lack of vibrations during the tests.

The experiment results presented here suggest that this negative strain rate effect is a valid phenomenon. Since the amount of time taken to conduct the four undrained tests were 0.167hrs (4mm/min), 1.67hrs (0.4mm/min), 16hrs (0.04mm/min), 160hrs (0.004mm/min). Aging may be one of the factors of this phenomenon. Aging has been studied by several researchers in the past. Some researchers indicated that some mechanical processes lead to aging, while others believed that some chemical processes result in this phenomenon.

Mesri et al. (1990) and Schmertmann (1991) hypothesized that the dominant mechanism of aging of granular systems on an engineering time-scale is creep. Increased strength and stiffness are due to a continued rearrangement of particles resulting in increased macro-interlocking of particles and increased micro-interlocking of surface roughness. This rearrangement is time-dependent. The lower the strain rate is, the longer the shearing time, the more the rearrangement happen. More rearrangement result in more macro-interlocking of particles and more micro-interlocking of surface roughness which lead to higher strength of soils. Richards and Barton (1999) found the existence of locked sands which exhibit a tensile strength even without the presence of binding cement. This is one support to the explanation above.

Besides mechanical processes, chemical processes related to time may be another possible cause for the increase in deviator stress with decreased strain rate. Historically, the most widespread theory used to describe chemical processes in sand is related to inter-particle bonding. Terzaghi originally referred to "bond strength" in connection with the presence of a quasi-pre-consolidation pressure in the field (Schmertman, 1992). Generally, this mechanism has been thought of as type of cementation, which would increase the cohesion of a soil without affecting its friction angle.

Denisov and Reltov (1961) showed that quartz sand grains adhered to a glass plate over time and the force required to move the sand grains continued to increase up to about 15 days of immersion in water. The cementating agent was thought to be silica-acid gel, which has an amorphous structure and would form a precipitate at particle contacts

(Mitchell and Solymer, 1984). They have noted that the formation of this silicic acid gel plays an essential role in the cementation of soil particles and experiments by Reltov have further shown that the gradual strengthening of hydraulically placed sand embankments is due to the formation of silicic acid gel films on the surface of quartz grains. Further studies by Denisov & Reltov (1961) have concluded that the bonding strength between the sand particles due to the formation of silicic acid gel films increase with the increase in contact time between the sand grain particles and water. It is possible that during rapid loading, where the soil particles are only in contact with the water for a short period of time, there is insufficient time for the formation of silicic-acid gel film on the surface of the grain particles to create inter-particle bonding. Hence, the peak stress (q_{peak}) achievable by the specimen increased with decreasing strain rate of shearing.

4.4 EFFECT OF STRAIN RATE ON THE BEHAVIOR OF MEDIUM LOOSE SAND

The strain rate may affect the undrained behavior of not only very loose sand but also medium loose sand. To investigate the effect of strain rate on medium loose sand, undrained tests on dilative specimens prepared by WS method were conducted.

4.4.1 Undrained behavior

Fig. 4.5a shows the results of the two undrained tests on medium loose sand conducted with strain rates of 0.04 and 0.4 mm/min. The stress-strain behavior is affected by the strain rate. As shown in Fig. 4.5b, the higher the strain rate, the higher the deviator stress. As can be seen from Fig.4.5c, the test with a slower rate (0.04mm/min) generated more excess pore water pressure. This trend is opposite to that observed in very loose sand.

Fig.4.5a

Fig.4.5b

Fig.4.5c

Fig.4.5 Effect of strain rate on the undrained behavior:

(a) Effective stress paths (b) Stress-strain behavior (c) Excess pore water pressure generated during shearing

4.4.2 Discussion

The results presented in Fig.4.5 appear to indicate a strain rate effect phenomenon. However, more data are required to confirm this observation. It can be seen from Fig.4.5c that an increase in strain rate from 0.04 mm/min to 0.4 mm/min produces an increase of approximately 3.5% in the soil strength for medium loose sand.

This increase in strength maybe related to an increased tendency to reduce pore water pressure (or dilate). This phenomenon of increasing dilatancy with strain rate had been observed by Healy (1963) and Lee et al. (1969) in Ottawa sand and Antioch sand. This result of changi sand was also observed in the thesis of Loke, W.L. (2003) which was shown in Fig.4.6 below.

Fig.4.6a

Fig.4.6b

Fig.4.6c

Fig.4.6 Effect of strain rate on the undrained behavior of medium loose sand. (a) Effective stress paths. (b) Stress strain behavior. (c) Excess pore water pressure generated during shearing.

4.5 SUMMARY AND CONCLUSIONS

Triaxial tests on very loose and medium loose sand were conducted under drained and undrained conditions at strain rates from 0.004 mm/min to 4.0 mm/min. The following observations can be made from the test results.

1. For very loose sand, the peak deviator stress, q_{peak} was found to decrease with increasing strain rate. Excess pore water pressure from undrained tests and volumetric strains from drained tests were observed to increase with increasing strain rate. Reasons of this strain rate effect will be discussed in the following chapter.
2. Contrary to the effect of strain rate on very loose sand, the deviator stress, q seems to increase with increasing strain rate for medium loose sand. A small increase of in deviator stress was observed when the strain rate increased from 0.04 mm/min to 0.4 mm/min. This increase is related to the increase in dilatancy as the rate of shearing increased. However, because of the two results are quite similar, more data are required to confirm this observation.

CHAPTER 5

Chemical Effect of The Silicic Acid Gel

5.1 INTRODUCTION

As mentioned previously, chemical processes as a function of time may be one possible cause for the increase in deviator stress with decreased strain rate. As discussed in Chapter 2. The cement agent was thought to be silica-acid gel, which has an amorphous structure and would form a precipitate at particle contacts (Mitchell and Solymer, 1984). Further studies by Denisov & Reltov (1961) have concluded that the bonding strength between the sand particles due to the formation of silicic acid gel films increased with the increase in contact time between the sand grain particles and water.

This chapter shows a series of undrained triaxial compression tests which are to investigate the influence of silicic acid gel on the behavior of cohesionless sands.

5.2 TESTING PROGRAME

The silicic acid was white odorless powder under room temperature. Its chemical formula is $\text{SiO}_2 \cdot n\text{H}_2\text{O}$. When the silicic acid powder is dissolved in water, we get the silicic acid gel solution which is achromatous and odorless. After placing for a period of time or under illumination, there will be floccules and precipitation in the silicic acid gel solution.

To investigate the influence of silicic acid gel on the behavior of cohesionless sands, a series of undrained triaxial compression tests were conducted. The specimens were prepared using the MT method. To eliminate the influence of cell pressure, all the specimens were consolidated isotropically under an effective confining pressure of 150 kPa before shearing. Before tamping, the silicic acid powder was dissolved in de-aired water to achieve the targeted concentration. Because the mass of chemical

solution could not be measured accurately, the concentration of the silicic acid gel was calculated as the percentage of dry chemical powder to dry sand. After saturation, the sample was placed for different periods of time before consolidation and shearing. The testing conditions of the tests are summarized in Table 5.1 below.

Table.5.1 Summary of testing conditions

	Test No.	Percentage of silicic acid gel by weight(%)	Placement time (hrs)	Strain rate (mm/min)	q_{peak} (kPa)	e_c
Ottawa Sand	vlc01	1	15	0.4	128.73	0.711
	vlc02		3		114.64	0.708
	vlc03		1		98.06	0.710
	vlc04	0.5	15		120.82	0.712
	vlc05		3		109.35	0.706
	vlc06		1		94.78	0.705
	vlc07	0	15		117.96	0.703
	vlc08		3		95.86	0.703
	vlc09		1		91.48	0.709

5.3 CHEMICAL EFFECT OF THE SILICIC ACID GEL ON THE BEHAVIOR OF VERY LOOSE SAND

Fig. 5.1 presents the results of the three undrained tests conducted using 1% of the silicic acid gel. The samples were placed for 1hr, 3hrs and 15hrs after consolidation and then sheared in a strain rate of 0.4mm/min. It can be seen clearly from Fig. 5.1a that all the tests exhibited a flow type of behavior with the peak strength (q_{peak}) showing dependency on the period of time submerged. As can be seen from the graph, test vlc01 submerged for 15hrs with 1% of the silicic acid gel has a peak strength (q_{peak}) about 128.73kPa. When submerged for shorter time with the same percentage of silicic acid gel in test vlc02, the peak strength (q_{peak}) becomes 114.64kPa. In test vlc03, submerged time is 1hr, the peak strength (q_{peak}) becomes 98.06kPa. The

longer the submerged time was, the higher the value of q_{peak} .

Fig.5.1a

Fig.5.1b

Fig.5.1c

Fig.5.1 Chemical effect on the undrained behavior (1%)

(a) Effective stress paths. (b) Stress-strain behavior. (c) Excess pore water pressure generated during shearing.

This is also seen in Fig. 5.1b where the stress-strain curves of the undrained tests are presented. It can also be observed from Fig. 5.1b that the peak stress occurs at almost the same axial strain in the three tests.

Fig. 5.1c shows the pore water pressure responses to the different strain rates. As can be seen from it, compared to test vlc01, more water pressure is generated when specimen is placed for shorter time in test vlc02 and vlc03. The longer the placement time was, the lower the increase in pore water pressure.

Fig. 5.2 presents the results of the three undrained tests conducted using 0.5% of the silicic acid gel. The samples were placed for 1hr, 3hrs and 15hrs after consolidation and then sheared in a strain rate of 0.4mm/min. It can be

seen clearly from Fig. 5.2a that all the tests exhibited a flow type of behavior with the peak strength (q_{peak}) showing dependency on the period of time the specimen was submerged. Test vlc04 on specimen submerged for 15hours has a peak strength (q_{peak}) of 120.8kPa. When submerged for shorter time with the same percentage of silicic acid gel in test vlc05, the peak strength (q_{peak}) becomes lower to 109.35kPa. In test vlc06, submerged time is lower to 1 hour, the peak strength (q_{peak}) becomes 94.78kPa. The longer the submerged time was, the higher the value of q_{peak} .

This is also seen in Fig. 5.2b where the stress-strain curves of the undrained tests are presented. The peak stress occurs at almost the same axial strain in the three tests. Fig. 5.2c shows the pore water pressure responses to the strain rates. Again it is seen that the longer the placement time, the lower the increase in pore water pressure.

Fig.5.2a

Fig.5.2b

Fig.5.2c

Fig.5.2 Chemical effect on the undrained behavior (0.5%).
(a) Effective stress paths. (b) Stress-strain behavior. (c) Excess pore water pressure generated during shearing.

To confirm whether the increase in peak strength is mainly a result of chemical process, a series of tests on samples which were merely placed for 1hr, 3hrs and 15hrs after consolidation without mixing any chemical and then sheared in a strain rate of 0.4mm/min were conducted. It can be seen from Fig. 5.3a that although no chemicals were added, the peak strength (q_{peak}) still shows dependency on the period of time submerged. Test vlc07 was conducted on sample submerged for 15hours. The peak strength (q_{peak}) was about 117.9kPa. When submerged for shorter time in test vlc08, the peak strength (q_{peak}) becomes lower to 96kPa. In test vlc09, with specimen submerged for 1 hour, the peak strength (q_{peak}) becomes 91.5kPa. The longer the submerged time was, the higher the value of q_{peak} .

This can also be seen from Fig. 5.3b where the stress-strain curves of the undrained tests are presented. The peak stress occurs at almost the same axial strain in the three tests. The pore water pressure responses to the different aging time are shown in Fig. 5.3c. It can be seen that more water pressure is generated when specimen is placed for shorter time. The shorter the placement time is, the higher the change in pore water pressure.

Fig.5.3a

Fig.5.3b

Fig.5.3c

Fig.5.3 Chemical effect on the undrained behavior (0%)

(a) Effective stress paths. (b) Stress-strain behavior. (c) Excess pore water pressure generated during shearing.

Fig.5.4a presents the results of the three undrained tests conducted using 1%, 0.5% and 0% of the silicic acid gel respectively. All the samples were placed for 15hrs after consolidation and then sheared in a strain rate of 0.4mm/min. It can be seen clearly from Fig. 5.4a and 5.4b that peak strength (q_{peak}) is affected by the percentage of the chemical. However, the effect is insignificant. Fig. 5.4c shows the pore water pressure responses are also similar with different percent of chemical.

Fig.5.4a

Fig.5.4b

Fig.5.4c

Fig.5.4 Chemical effect on the undrained behavior (15 hrs).

(a) Effective stress paths. (b) Stress-strain behavior. (c) Excess pore water pressure generated during shearing.

5.4 DISCUSSION

As can be seen from Fig.5.1a, Fig.5.2a, Fig.5.3a and Fig.5.4a, the peak deviator stresses (q_{peak}) are affected by different percentage of chemical and period of time submerged. When mixed with the same percentage of silicic acid gel, the longer the submerged time, the higher the peak deviator stresses (q_{peak}). When placed with the same period of time, test of higher percentage of silicic acid gel shows a higher peak deviator stresses (q_{peak}). Fig.5.1b, Fig.5.2b and Fig.5.3b show that the peak deviator stresses (q_{peak}) in all the undrained tests occurred at a similar axial strain about 0.86%. We can also see from Fig.5.1c, Fig.5.2c and Fig.5.3c that a larger excess pore water pressure developed in test of shorter submerged time with the same percentage of chemicals.

To see how the silicic acid gel works in the samples, some scanning

electron microscopy photos were taken. Fig.5.5a and Fig.5.5b shows the individual grains of Ottawa sand without chemical mixed before shearing; Fig.5.5c and Fig.5.5d shows the individual grains of Ottawa sand with chemical mixed before shearing; Fig.5.5e and Fig.5.5f shows the same grains with chemical mixed after shearing.

From Fig.5.5a and 5.5b we can see that the original sand particles are clean and the surface is smooth. When the sand was mixed with silicic acid gel, the particles were covered with a layer of coating as shown in Fig.5.5c and 5.5d, However, cementation was not obvious. A photo of the sand after shearing was also shown in Fig.5.5e and 5.5f. Some of the coating layer appeared to have broken.

Therefore the existence of silicic acid gel should have affected the undrained behavior of Ottawa Sand to certain extent. The formation of the silicic acid gel films increase with increasing in contact time between the sand particles and water.

Fig.5.5a

Fig.5.5b

Fig.5.5c

Fig.5.5d

Fig.5.5e

Fig.5.5f

Fig.5.5 SEM photograph of Ottawa Sand

- (a) Without chemical mixed before shearing (20). (b) Without chemical mixed before shearing (100). (c) With chemical mixed before shearing (20). (d) With chemical mixed before shearing (100). (e) With chemical mixed after shearing(20). (f) With chemical mixed after shearing(100).

To study whether some chemical reaction took place during tests, XRD

test was conducted on the samples. X-Ray Diffraction (XRD) uses information about the position, intensity, width, and shape of diffraction peaks in a pattern from a polycrystalline sample to indicate any chemical content in the mixture. X-axis means the intensity and Y-axis is regarding to two-theta which is the diffraction angle. Fig.5.6a shows the XRD result of pure Ottawa sand. This diffraction peak position and intensity is almost the same as the standard diffraction peak position and intensity of SiO_2 . This indicated that the main chemical composition of Ottawa sand is SiO_2 . Fig.5.6b shows the XRD result of Ottawa sand mixed with silicic acid gel for two hours before shearing. Compared to the peak property in Fig.5.6a, the new peak position and intensity are similar. This indicated that the chemical composition of the mixture is still SiO_2 , there isn't any other chemical being generated. Although peak intensity is something different, it is because that silicic acid gel film wraps up the sand particles which lead to surface modification. This is supported by the SEM results showed in Fig.5.5c and Fig.5.5d. Fig.5.6c shows the XRD result of sand with silicic acid gel after shearing. It can be observed that the position and intensity is similar to the wave in Fig.5.6b. This means that there is not new chemical being generated during shearing.

Fig.5.6a

Fig.5.6b **Fig.5.6c****Fig.5.6 XRD result of sand**

(a) Pure Ottawa Sand (b) Sand mixed with silicic acid gel before shearing (c) Sand mixed with silicic acid gel after shearing

5.5 AGING PROCESS IN STRAIN RATE EFFECT

Another question that needs to be addressed is whether shearing or stress state affects the aging process. To investigate this, different aging periods during consolidation were also used. A series of undrained tests were conducted as listed in Table.5.2.

Table.5.2. Summary of testing conditions

Sand Type	Test No.	Placement time (hrs)	Strain rate (mm/min)	q _{peak} (kPa)	e _c
Ottawa Sand	vlu01	0	0.004	163.89	0.714
	vlu02	160	0.4	132.96	0.713
	vlu03	0	0.04	122.98	0.709
	vlu04	15	0.4	109.73	0.704

Fig. 5.7 shows two tests prepared by MT method and having similar void ratio. Sample of test vlu01 was sheared in strain rate of 0.004mm/min without aging during consolidation, while sample of test vlu02 was placed for aging time of 160 hrs and was sheared with a strain rate 0.4mm/min.

For strain rate 0.004mm/min, the shearing time before failure is at least 160hours. Therefore, the total contact hour with chemical is about the same. If chemical process is the only reason, Test vlu01 and vlu02 should have similar undrained behavior. However, we can see that test vlu01 has a peak deviator stress of 163.9kPa which is much higher than that in Test vlu02. With the increase in aging time, the deviator stress increases. However, this effect is not as strong as the increase due to strain rate effect. In other word, aging effect under shearing is much more pronounced than it under consolidation.

Fig.5.7a

Fig.5.7b

Fig.5.7c

Fig. 5.7 Comparison between strain rate effect and aging effect on the undrained behavior of Ottawa Sand

(a) Effective stress paths. (b) Stress-strain behavior. (c) Excess pore water pressure generated during shearing.

From Fig. 5.7 we can see that undrained behavior are affected by both strain rate and aging. For strain rate 0.4mm/min, the shearing time is about 3 hrs. For 0.04mm/min and 0.004mm/min, the shearing times are at least 15 hrs and 160 hrs respectively. So if we hypothesis that chemical reaction process effect is the only reason which causes the strain rate effect, test vlu01 and test vlu02, test vlu03 and test vlu04, which having the similar contact time should have similar undrained behavior. However, we can see that test vlu01 has a peak deviator stress of 163.89kPa, while test vlu02 only has 132.96kPa. Test vlu03 has a peak deviator stress of 122.98kPa, while test vlu04 only has 109.73kPa. Samples have similar aging time, but they don't behavior similarly. With the increase of aging time the deviator stress increases, but not as much as the increase in different strain rate

tests, or we can say aging-effect under shearing is much more pronounced than it under consolidation.

To check the test results in vlu01 and vlu02, another two tests were conducted. Sample of test vlu03 was sheared in strain rate of 0.04mm/min without aging during consolidation, while sample of test vlu04 was placed for aging time of 15 hrs and was sheared with a strain rate 0.4mm/min. For strain rate 0.04mm/min, the shearing time before failure is at least 15hrs. Therefore, the total contact hour of these two tests with chemical is about the same. It was shown in Fig.5.8 that there were similar behavior in Test vlu03 and vlu04 as in Test vlu01 and vlu02. The aging effect under shearing is more pronounced than it under consolidation, but the difference was not as big as it in Test vlu01 and vlu02.

Fig.5.8a

Fig. 5.8b

Fig. 5.8c

Fig. 5.8 Comparison between strain rate effect and aging effect on the undrained behavior of Ottawa Sand
(b) Effective stress paths. (b) Stress-strain behavior. (c) Excess pore water pressure generated during shearing.

According to the analysis above, we may consider that physical process such as dispersed movement and inter-particle locking may be the other factors contributing to the aging process. Schmertmann (1991) indicated that dispersive particle movements and increased interlocking were two of causes of aging effects in sands. It was suggested that when movements occurred during aging, platy particles would tend to orient themselves perpendicular to the direction of the major principal effective stress.

Schmertmann postulated that the dispersive particle movements mentioned above would result in an increased micro interlocking at the particle contacts, which might also contribute to the aging effect. The strength due to the micro interlocking would be a function of the normal stress at the contacts and would cause an increase in the frictional component of strength.

Dispersive particle movements and inter-particle locking are both physical process and they are naturally accelerated by slow shearing process. So compared to vlu02 of which sample was just submerged much more dispersive particle movements and inter-particle locking happened in vlu01, which would manifest themselves as an increase in strength and stiffness. That is why aging-effect under shearing is much more pronounced than it is under consolidation.

5.6 CHEMICAL EFFECT ON CONE PENETROMETER TEST

5.6.1 Materials

The material tested in this research was standard sand Ottawa sand and Changi sand. The particle size distribution is shown in Fig.3.1 and Fig. 5.9. A summary of the basic properties of the materials is presented in Table

5.3 below.

Table 5.3 Basic properties of Ottawa Sand and Changi Sand

	Mean Size(mm)	Coefficient of Uniformity (CU)	Coefficient of Curvature (CC)	Maximum Void Ratio (e_{max})	Minimum Void Ratio (e_{min})	Specific Gravity(G_s)
Ottawa Sand	0.4	2.0	0.85	0.85	0.50	2.64
Changi Sand	0.55	3.28	0.53	0.92	0.53	2.65

Fig.5.9 Particle size distribution curve for Changi Sand

5.6.2 Apparatus for test

The detail of the cone penetrometer is shown in Fig.5.10a and Fig.5.10b. The sharpness gauge for cone consists of a small steel plate 1.75 ± 0.1 mm thick with a $1.5 \text{ mm} \pm 0.02 \text{ mm}$ diameter hole accurately drilled and reamed. The metal cups are made of brass or aluminum alloy, 55mm diameter and 40 mm deep as shown in Fig.5.11.

Fig.5.10 Details of cone

Fig.5.11 Picture of metal cup and gauge

5.6.3 Test procedure

Put 200g sand and 200g water into a metal container. Mix and boil them to remove any air bubbles that might be trapped within the sand. Then mix 4g, 2g and 1g (2%, 1% and 0.5% of total mass of sand) of silicic acid respectively into sand. After mixing, cover the container with plastic cloth and tie it with rubber band to avoid evaporation of water. Put the sample into a constant temperature and humidity cabinet for 1 hour, 3 hours and 20 hours respectively before test. After releasing the cone for 5s, it will be

locked and the penetration depth was recorded.

5.6.4 Test results

Tests were conducted on Ottawa Sand at different percentage and different placement time. For comparison, Tests were also conducted on Changi Sand. At least three tests were carried out for each testing condition to ensure that the results obtained were consistent. The testing conditions of the tests are summarized in Table 5.4 below.

Table 5.4 Cone penetrometer test results

	Test No.	Percentage of silicic acid gel by weight (%)	Placement time (hrs)	Penetration depth (mm)		
Ottawa Sand	P1	0	1	13.72	13.80	13.95
			3	13.26	13.39	13.51
			20	12.56	13.00	13.22
	P2	0.5	1	13.33	13.48	13.66
			3	12.55	12.67	12.89
			20	12.13	12.45	12.66
	P3	1	1	12.98	13.11	13.44
			3	12.01	12.20	12.34
			20	11.71	11.89	12.01
	P4	2	1	11.88	12.05	12.52
			3	11.87	11.91	12.01
			20	11.22	11.48	11.66
Changi Sand	P0	0	1	12.34	12.59	12.88
			3	12.11	12.28	12.56
			20	10.88	11.04	11.33
	P0.5	0.5	1	11.75	11.93	12.12
			3	11.31	11.67	12.09
			20	10.45	10.72	10.98
	P1	1	1	11.66	11.77	11.99
			3	10.88	11.06	11.32
			20	9.92	10.05	10.33
	P2	2	1	11.08	11.34	11.67
			3	9.77	9.92	10.12
			20	9.13	9.44	9.66

a. Test results of Ottawa Sand

Fig.5.12 and Fig.5.13 presents the results of penetration depth for

Ottawa Sand. It can be seen clearly that all the tests exhibited some behavior with the penetration depth showing dependency on the percentage of chemical. For every group of tests with the same placement time, the lower the percentage of chemical is, the higher the penetration depth. For every group of tests with the same percentage of chemical, the longer the placement time, the lower the penetration depth.

Fig 5.12 Penetration depth vs. placement time for Ottawa Sand

Fig 5.13 Penetration depth vs. percentage of silicic acid for Ottawa Sand

b. Test results of Changi Sand

Fig.5.14 and Fig.5.15 presents the results of penetration depth for Changi Sand. It can be seen clearly that all the tests behavior are similar to Ottawa Sand. The lower the percentage of chemical is, the higher the penetration depth. The longer the placement time was, the lower the penetration depth.

Fig 5.14 Penetration depth vs. placement time for Changi Sand

Fig 5.15 Penetration depth vs. percentage of silicic acid for Changi Sand

5.6.5 Discussion

We can see that from the test results above the cone penetrometer depth of both Ottawa Sand and Changi Sand show dependence with the chemical percentage and aging time. For every group of tests with the same percentage of chemical, the longer the aging time, the lower the penetration depth. For every group of tests with the same aging time the lower the percentage of chemical is, the higher the penetration depth. Since lower penetration depth usually means higher strength and stiffness of soil. This is consistent with the tests results in section 5.4 and 5.5 which can be seen another evidence of the chemical effect.

5.7 SUMMARY AND CONCLUSIONS

Triaxial tests on sand with and without chemicals were conducted under undrained condition at strain rates ranging from 0.004 mm/min to 4.0 mm/min. The following conclusions can be made from the test results.

1. The stress-strain behavior and the peak deviator stresses (q_{peak}) are affected by the percentage of chemical and the period of time submerged. With the same percentage of silicic acid gel, the longer the submerged time was, the higher the peak deviator stresses (q_{peak}). When the specimens were aged with the same period of time, the higher the percentage of silicic acid gel, the higher the peak deviator stresses (q_{peak}). The peak deviator stresses (q_{peak}) in all the undrained tests occurred at a similar axial strain.

2. When mixed in the sample, the silicic acid gel will form a film which wraps around the sand particles. The film may increase the bonding strength between sand particles. However, it can be partially destroyed under shearing.

3. Aging-effect is one cause of strain rate effect. Aging-effect under shearing is much more pronounced than under consolidation. It is probably because of more dispersive particle movements and inter-particle locking happened during shearing.

4. The depth of penetration by a cone penetrometer into saturated sand samples is affected by the percentage of silicic acid gel and aging time. For every group of tests with the same content of chemical, the longer the aging time, the lower the penetration depth. For every group of tests with the same aging time, the lower the percentage of chemical is, the higher the penetration depth.

CHAPTER 6

Conclusions and Recommendations

6.1 CONCLUSIONS

An experimental study of the effect of shearing rate and the effect of silicic acid gel on the stress-strain behavior of Ottawa sand has been carried out. The main conclusions from this study can be summarized as follows.

1. In the undrained tests for very loose sand, the peak deviator stress (q_{peak}) decreases with increasing strain rate. Excess pore water pressure from undrained tests increases with increasing strain rate. This strain rate effect was attributed partially to the aging effect and the surface modification caused by silicic acid gel.
2. In the drained tests for very loose sand, similar to the undrained tests, the peak deviator stress (q_{peak}) decreases with increasing strain rate. The volumetric strain increases with increasing strain rate.
3. In the undrained tests for medium loose sand, contrary to the very loose sand, the peak deviator stress (q_{peak}) increases with increasing strain rate. Excess pore water pressure from undrained tests decreases with increasing strain rate. However, because of the tests results are quite a few and similar, more data are required to confirm this observation.
4. In the undrained tests for very loose sand mixed with silicic acid gel, the peak deviator stresses (q_{peak}) are becoming higher under higher percentage of silicic acid gel and longer aging time before consolidation and during shearing. When mixed with the same percentage of silicic acid gel, the test on specimens with longer aging time before consolidation shows a higher peak deviator stresses (q_{peak}) and lower excess pore water pressure. When the specimens were aged with the same period of

time before consolidation, test on specimens with higher percentage of silicic acid gel shows higher peak deviator stresses (q_{peak}) and similar excess pore water pressure. The peak deviator stresses (q_{peak}) in all the undrained tests occurs at a similar axial strain.

5. When the silicic acid gel was mixed in the sample, gel particles absorbed on and wrapped around the sand particles. This change of sand surface property will increase the bonding strength between sand particles.
6. The depth of penetration by a cone penetrometer into saturated sand samples mixed with silicic acid gel is affected by the percentage of silicic acid gel and the aging time. For every group of tests with the same percentage of chemical, the longer the aging time, the lower the penetration depth. For every group of tests with the same aging time, the lower the percentage of chemical is, the higher the penetration depth.

6.2 RECOMMENDATIONS FOR FUTURE RESEARCH

Several findings have been made in this thesis. However, due to the time constraints several important factors could not be studied. Following recommendations can be made for further research work.

1. The effects of strain rate should be studied for other types of sand besides Changi sand and Ottawa sand which are found to be affected more by strain rates. Because shape of particles and particle size distribution may significantly affect the strain rate effect, other sands with different shape of particles and particle size distributions should be studied to confirm this effect and make some comparison between different types of sand.

2. Silicic acid gel film was thought to form in the sample after long contact between sand particles and water under high pressure. But this phenomenon was not directly observed in this thesis. Further test and observation should be conducted to verify whether it exists and the mechanisms of how it interacts with sand.
3. Cone penetrometer tests in laboratory were conducted to study the chemical effect and aging effect. To confirm these effects, large scale model test should be carried out.

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