CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

In this chapter, literature in the following main areas is reviewed:

1) Geology and Geotechnical study of Old Alluvium (OA)
2) Research on the shear strength of sand and sand mixed with fine material
3) Research on the $K_o$ and in-situ horizontal earth pressure
4) Research on sampling effects

It may appear all four topics are independent but combining the knowledge from these areas will help to develop a framework for the characterization of OA, as will be clear towards the end of the chapter.

2.2 Geology and Geotechnical Study of Old Alluvium

2.2.1 Distribution of OA

The distribution of OA on the outcrop on Singapore Island is illustrated in Figure 1-1. It covers about 15% of the country’s land area forming undulating hills of up to 32m (Tan et al., 1980). It is also found around the coasts of South China Sea and the Malacca Straits, as well as corresponding offshore regions (Gupta et al., 1987).
2.2.2 Content and Nature of OA

OA is a heterogeneous material and various researchers have described it differently. During the early research work on Kinta Valley deposits, Walker (1956) described the Old Alluvium as consisting mainly of grey to brown sandy clay, with frequent intercalated layers of sand and gravel. According to PWD (1976), OA exposed in Singapore was seen to be a clayey coarse angular sand with stringers of subrounded pebbles up to 4cm in diameter. Fine-grained beds were also present, usually as small lenticular bodies. The pebbles within the OA were dominantly quartz, but rhyolite, chert, and argillite pebbles were also found. No granite pebbles were found in Singapore OA.

Table 2-1 A summary of characteristics of different types in Old Alluvium (after Gupta et al., 1987)

<table>
<thead>
<tr>
<th>Type</th>
<th>Common Structure</th>
<th>Classification</th>
<th>Possible Origin</th>
<th>Frequency</th>
<th>No. of beds measured</th>
<th>Average bed thickness (m)</th>
<th>Maximum bed thickness (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Coarse sand with fine pebbles</td>
<td>Massive or with faint horizontal lamination</td>
<td>Sh</td>
<td>Flood deposited channel fill; upper flow regime</td>
<td>Common 60-70 %</td>
<td>107</td>
<td>0.56</td>
<td>4.75+</td>
</tr>
<tr>
<td></td>
<td>Planar cross bedding</td>
<td>Sp</td>
<td>Transverse channel bar</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Medium to coarse sand</td>
<td>Massive or coarsening/fining upwards</td>
<td>Sh</td>
<td>Flood deposited channel fill; upper flow regime</td>
<td>Common 20-30 %</td>
<td>77</td>
<td>0.40</td>
<td>2.50+</td>
</tr>
<tr>
<td></td>
<td>Planar cross bedding</td>
<td>Sp</td>
<td>Transverse channel bar</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Clay and silt</td>
<td>Massive (Sheared/fissured)</td>
<td>Fi</td>
<td>Pockets of lenticles or small channel fill</td>
<td>Low</td>
<td>12 (not including chute)</td>
<td>0.24 (not including chute)</td>
<td>1.16 for chute deposit 0.40 for other beds</td>
</tr>
<tr>
<td></td>
<td>Alternate beds or laminated</td>
<td>A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4. Pebble beds</td>
<td>Massive or indistinct imbrication</td>
<td>Gm</td>
<td>Longitudinal channel bar</td>
<td>Low</td>
<td>33</td>
<td>0.15</td>
<td>0.50</td>
</tr>
</tbody>
</table>
Gupta et al. (1987) gave the most complete description of OA content and nature. A cross section of OA is shown in Figure 2-1. It was noticed that a repeated sequence of a limited number of beds in truncated condition characterizes the Old Alluvium. The beds could be grouped into the following four textural classes: (1) pebbles, (2) coarse sand with fine pebbles, (3) medium to coarse sand, and (4) clay and silt (Table 2-1). Each of the four classes also carried distinctive sedimentary structures and could be identified as definite morphological features.

Pitts & Gupta (1992) and Gupta et al. (1987) reported that the two sandy lithologies (coarse sand with fine pebbles, and medium to coarse sand mixed with clay and silt) form about 90 percent of the deposit. The lithology was usually structureless or possesses a faint horizontal lamination.

### 2.2.3 Mineralogy and Colours of OA

The pebbles were mostly quartz, vein quartz, cryptocrystalline silica and quartzite. Individual pebble-size grains of fresh alkali feldspar were occasionally found. The sand grains are of the same mineralogy, mainly made of quartz and feldspars. XRD tests showed that the clay mineralogy was a mixture of kaolinite (average 49%), illite (average 18%) and smectite (average 33%) (Gupta et al., 1987). The silt faction is almost entirely quartz (Pitts and Gupta, 1992).

The mineralogy of OA is mainly quartz and feldspar throughout. Colours of OA are mainly derived from the different colours of the fines, since quartz has no colour and feldspar is white. According to Walker (1956), OA in Kinta Valley is grey to brown, often with red or yellow bands from iron staining. Peart et al. (2001) classified the cemented OA
sandstones as ‘brown’ and ‘blue’ sandstones. These colours may be a reflection of the different chemical processes went through, which resulted in some different minerals in the fines.

### 2.2.4 Provenance and Age of OA

The mineralogy of the grains forming the Old Alluvium suggested a mixed provenance of granitic and low-grade metamorphic origin. The freshness of the feldspars indicated that erosion of unweathered rocks with subsequent transportation over a short distance followed by rapid burial had occurred (Gupta et al., 1987). However, the rarity of granite pebbles may preclude a nearby local source, which is the Bukit Timah Granite Formation to the west of the OA outcrop. Since such pebbles are more common just to the north in Johore Bahru, the source is likely to be from the this area to the north of Singapore (Tai, 1972).

The absolute age of Singapore Old Alluvium is difficult to determine because of the lack of fossils in this soil. However, since this fluvial deposit continued extensively on a regional scale under the present sea level, and there had been considerable changes in the level of the South China Sea during the Pleistocene (Biswas, 1973), one tends to attribute Pleistocene age to this formation (Gupta et al., 1987).

### 2.2.5 Deposition Environment and Field Relations of OA

Old Alluvium’s continuation from the land to below the present sea level, and its regional similarity, indicated that the environment and processes responsible for its deposition persisted over a large area. However, environmental reconstruction is difficult because of
the scarce knowledge of the Pleistocene era in Southeast Asia. Biswas (1973) investigated the Quaternary cycles of eustatic changes in sea-level and their extent in pre-Holocene times by using lithologic, foraminiferal, and spore-pollen evidence and through this, tried to establish the environment of the OA deposition. The sea level drop estimated was about 144~236 feet (43.89~71.93m) and the youngest eustatic drop of sea-level occurred around 11,000 years ago. As water depths of South China Sea do not exceed 200m and are less than 100m over large areas, relatively minor fluctuation of the sea-level could result in major lateral shifts in sedimentary facies. The shrinkage of the sea was estimated by Biswas to be about 750,000km$^2$. That is to say, a large part of the sea bed was exposed (Figure 2-2).

OA was deposited by a braided river system when the sea level was low. The most plausible explanation behind the braided river system lay in seasonality of flow and presence of abundant coarse sediment in the channel, which were derived from intense erosion in the basin. The entire river channel was never seen in a cross-section, yet a river was estimated to be at least 100m wide and about 2-5m deep at bankfull stage. Minor channels could be 1 to 2m deep and the biggest one measured so far is 30m wide. These channels were separated by transverse or longitudinal bars of coarse sediments. OA is widespread in the region and suggested a high rate of erosion at that time. It was probably deposited at a time of low sea level and later it had been incised, inundated and infilled with the Holocene sediments. The general absence of organics was probably due to the high-energy environment of deposition and the post depositional oxidation. Regarding post-depositional deformation, no unequivocal signs of tilting of the beds had been found in Singapore Old Alluvium (adapted from Gupta et al., 1987).
The contact between the OA and older formations was buried or obscured by deep weathering. The general texture of the OA as exposed in Singapore was consistent with that of an alluvial fan or piedmont plain type deposit. No evidence of marine incursion could be seen (PWD, 1976).

Ingham & Bradford (1960) also mentioned that commonly slumped and distorted nature of these deposits was observed. This was due to potholes created by the solution of underlying limestone bedrock. This was also pointed out by Scrivenor (1919, 1931) previously.

2.2.6 Thickness and Stress History of OA

The present thickness of OA is also highly variable. According to PWD (1976), in Singapore, the recorded maximum depth of OA was –149m below sea level and the recorded maximum height of OA was 45m above sea level, thus giving a total thickness of around 195m. However, Gupta et al. (1987) questioned the 149m depth reported by PWD (1976), saying that it was difficult to distinguish OA and the weathered product of the bed rock so it might be a problem with the identification. Gupta et al. believed that OA was at least 50m thick and the top was eroded. Given the evidence that OA was found up to 32m elevation on local hills, the maximum thickness of OA would still have been over 100m thick.

Pitts (1986) used an approximate method to determine the maximum preconsolidation pressure in Singapore Old Alluvium because the normal method of measuring preconsolidation pressure using an oedometer is very difficult (as the sample is usually sandy). The ratio $c/p'$, which was the ratio of undrained cohesive strength ($c$) to the
effective consolidation pressure \( (p') \), was used in conjunction with the undrained shear strength. Values of \( c/p' \) were determined using empirical correlation with Atterberg Limits. The \( c \) value was measured in a series of undrained direct shear tests, then the \( p' \) was back calculated from \( c \) and \( c/p' \) values. This is then used to calculate an overburden thickness by assuming a value for the unit weight. It was suggested that a minimum of 50 to 60 m of erosion had occurred.

On Singapore island, OA forms undulating hills of up to 32 m (Tan et al., 1980). PWD (1976) reported OA at 45 m above sea level. In Malaysia, Walker (1956) pointed out that the Old Alluvium constituted most of the valley fill in the Kinta Valley and was found at elevations of up to 70 m above sea level. Based on these pieces of information, it seems OA in Singapore was eroded at the top and has an over consolidation stress history.

There is a great scattering in the measured \( K_o \) (coefficient of earth pressure at rest) values. Dames and Moore (1983) recommend a lower bound value for \( K_o=0.75 \) and an upper bound \( K_o=1.0 \). Li and Wong (2001) determined \( K_o \) from the lift-off pressure \( (P_o) \) in pressuremeter tests and the results were highly scattered, varying from 0.5 to 2.5.

### 2.2.7 Cementation of OA

Researchers have differing opinions on whether OA is cemented or not. PWD (1976) reported OA was usually uncemented but quite dense and with a low permeability. Zones of cementation were found but the cemented rock often disintegrates after a few days of exposure. Some areas of cemented OA have been exposed for several years and yet have remained cemented. No studies of the cement in these areas had been made.
However, based on the site investigation results on the MRT Changi Airport Line, Peart *et al.* (2001) argued that OA was, for its greater part, sandstone with strengths in the ‘very weak’ range. However, it was also noticed that despite its compact nature, when the material came into contact with free water there was a marked and often rapid breakdown into its constituent parts. In addition the material also broke down rapidly under mechanical handling once in an unconfined state.

Dames & Moore (1983) noticed the great variability of OA both vertically and horizontally, together with the fact that significant increases in shear strength and reductions in compressibility occur on drying out of the soil. Using undrained triaxial compression tests and direct shear tests, effective cohesion \( (c') \) was found in OA samples, which was believed to be due to cementation. However, the cementation is either extremely variable or tends to break down during sampling and test preparation because the test results are so erratic, particularly those of the direct shear tests.

### 2.2.8 Index Properties of OA

Tan *et al.* (1980) found the soil consistency was generally medium dense to very dense for the sands and stiff to hard for the clayey soils. Water content of the sandy and clayey soils generally ranges from 15%~25% and 20%~40% respectively. It was also noticed that weathering did not seem to have altered the nature of the clays and surprisingly that there was no significant difference between a population of moisture content values of sandy soils randomly sampled from above the water table and a population sampled from below the water table. There was no correlation between randomly selected relative density values and their sampling depths plotted in relation to mean sea level.
Li and Wong (2001) studied the profile of Old Alluvium along the North-East line and subdivided OA into three zones. They were the residual soil zone (OAI), weathered zone (OAII), and cemented zone (OAIII). The Unified Soil Classification System (USCS) was used to classify a total of 774 OA samples and it was shown that the SC and SM soils made up 71% of OA. CL and CH soils covered 21% and soils with fines <12% made up the remaining 8%. Figure 2-3 shows the composition of all OA as well as those in each zone. It was also noticed that very little CL soil (4%) and no CH soil were encountered in OAIII and it was concluded that a decrease in clay content with depth indicated a decrease in the degree of weathering. The OA covered a wide range of particle sizes and the fines content of the SC&SM soils was typically between 20% and 30%. The average fines content for CH was 84 %, for CL was 67 %, for SC&SM was 24 % and “sandy soils with less than 12% fines” was 8.3 %. The unit weight is consistent, with an average of 20.3 kN/m³ and the water content varies between 10% and 40%. The specific gravity ($G_s$) generally lies between 2.6 and 2.7 with an average of 2.65. Atterberg limit test results varied widely. It was concluded that OA was heavily overconsolidated and it was more clayey than silty.

2.2.9 Permeability of OA

There is great scattering in the measured values of Old Alluvium permeability. Dames & Moore (1983) suggested the following values of coefficient of permeability ($k$): upper bound $10^{-6}$m/s, lower bound $10^{-10}$m/s and overall design value $10^{-9}$m/s. Li and Wong (2001) also found that there was a big discrepancy between the results of oedometer and in-situ permeability tests. The $k$ values obtained from oedometer tests varied from $10^{-10}$
m/s to $10^8$ m/s whereas those measured in the field ranged from $10^9$ to $10^6$ m/s. It is believed that the in-situ tests are more credible, but there is no clear trend observed between $k$ value and fine content.

2.2.10 Shear Strength and Stiffness Parameters of OA

The shear strength of OA seems to have no relationship with depth (Tan et al., 1980). Orihara and Khoo (1998) proposed the relationship of shear strength and SPT N-values of OA.

The common practice of SPT in Singapore is as follows: The SPT sampler was driven 450 mm into the soil by a 63.5 kg free falling hammer over a height of 760 mm. The first 150 mm penetration was regarded as the seating drive, hence the number of blow to achieve this penetration was not included in the SPT N-value. The total cumulative numbers of the blow counts required for each 100 mm of the last 300 mm penetration was recorded as the SPT N-value. If 300 mm penetration was not achieved at 100 blow, the driving is stopped and the N-value was reported to be 100 (Kiso Jiban, 2002).

According to Orihara and Khoo (1998), the data of shear strength $Cu$ and SPT N-values falls very broadly into a band and could be approximated as

$$Cu = 6N \text{ (kPa)} \quad (2.1)$$

Li and Wong (2001) determined the undrained shear strength ($Cu$) from Unconsolidated Undrained (UU) Triaxial Compression tests and $Cu$ was found to be correlated to SPT N value and liquidity index (LI) by the following equations:

$$Cu = 5.4N \text{ (kPa)} \quad (2.2)$$

$$Cu = 172.21 e^{-4.6LI} \text{ (kPa)} \quad (2.3)$$
Consolidated Isotropic Undrained (CIU) and Consolidated Isotropic Drained (CID) triaxial tests were done and the summarized friction angle was within the range of 35 ~ 36.5°. It was also stated that OAIII in CIU tests shows an average effective cohesion intercept (\(c'\)) of 30.3kPa (Li and Wong, 2001, Figure 2-4).

The deformation modulus \(E\) obtained by pressuremeter tests and plate load tests were compared with SPT N-values. The following approximate relationships were derived by Orihara and Khoo (1998) based on this comparison:

\[
E = 1 \, N \text{ (MPa)} \quad \text{for 1\textsuperscript{st} cycle of pressuremeter test} \quad (2.4)
\]
\[
E = 2 \, N \text{ (MPa)} \quad \text{for 2\textsuperscript{nd} cycle of pressuremeter test} \quad (2.5)
\]
\[
E = 3 \, N \text{ (MPa)} \quad \text{for 1\textsuperscript{st} cycle of plate load test} \quad (2.6)
\]
\[
E = 6 \, N \text{ (MPa)} \quad \text{for 2\textsuperscript{nd} cycle of plate load test} \quad (2.7)
\]

Li and Wong (2001) also studied pressuremeter test results in OA and several parameters were shown to have a rough correlation with SPT N-values. Such parameters include \(E_{PMT}\), the pressuremeter modulus from the first cycle of test, \(E_r\), the unloading-reloading modulus of the second cycle, lift-off pressure \(P_0\), yield pressure \(P_y\) and limit pressure \(P_L\). They all increase with SPT N value:

\[
E_{PMT} = 0.74 \, N \text{ (MPa)} \quad (2.8)
\]
\[
E_r = 3.72 \, N \text{ (MPa)} \quad (2.9)
\]
\[
P_0 = 8.6 \, N \text{ (kPa)} \quad (N < 100) \quad (2.10)
\]
\[
P_y = 41 \, N \text{ (kPa)} \quad (N < 100) \quad (2.11)
\]
\[
P_L = 106 \, N \text{ (kPa)} \quad (N < 100) \quad (2.12)
\]
2.2.11 Construction Problems in OA

Though OA is generally considered an excellent engineering soil, there were some major construction problems which had resulted in failures. One such problem was reported by Knight-Hassele et al. (2001). It occurred during the construction of Cross Passage 5 (CP5) which was one part of the North East Line (subway) Project. The tunnel excavation was 28~31.5m below the ground level. After several hours of excavation in what was essentially a weak bedded rock, a small water seepage was noticed at the base of the 1.5m high tunnel face. The excavation continued but within 30 minutes a sudden inundation occurred with an estimated water flow of some 300 litres per minute. Large quantities of sand were washed into the tunnel.

The Old Alluvium there was dominantly a yellow grey/light grey silty fine sand with fine, medium and coarse quartz gravel. The SPT>100 material was present to a depth of 30m below ground level where the SPT suddenly drops to 33. The authors concluded that OA contained confined aquifers with water under sub-artesian pressures. The behaviour of the ground during the excavation of the shafts, dewatering wells and the cross passage itself did strongly suggest the presence of a pressurized, less dense, more permeable horizon rather than a layer of very dense material that is being disturbed on the release of sub-artesian water pressures.

2.3 Shear Strength of Sand and Sand Mixture

2.3.1 Undrained Shearing Behaviour of Sand

The undrained behaviour of clean sand has been investigated extensively and a schematic diagram is shown in Figure 2-5. Pattern (1) is shown by loose, saturated clean sand and
there are various terms for it, like contractive behaviour (Thevanayagam and Mohan, 2000), strain-softening (Pitman et al., 1994), pre-failure strain-softening (Chu et al., 1992) or flow liquefaction (Yang, 2002). Pattern (2) is shown by medium dense sand, called contraction followed by dilation (Thevanayagam & Mohan, 2000), strain-softening followed by strain-hardening (Pitman et al., 1994) or limited liquefaction (Vaid et al., 1990). The ‘elbow’ in the $p’q$ plot of pattern (2) is also defined as the Quasi-Steady State (Ishihara, 1993). Dilative behaviour pattern (3) is shown by very dense sand.

It is generally agreed that clean sand in undrained shearing will approach a Steady State Line (SSL) or Critical State Line (CSL) at large strains.

2.3.2 Shear Strength of Clean Sand

In the 33rd Rankine lecture paper by Ishihara (1993), results of extensive laboratory tests on the Japanese standard sand were presented and new index parameters were proposed to quantify undrained sand behaviour better. It was established that for the Toyoura sand, the undrained shear strength at steady state ($S_u$), is determined by the void ratio alone as shown in Figure 2-6. For sand subjected to undrained shearing, the pore water pressure increases or decreases, depending on the initial confining stress, so as to bring the effective confining stress to a unique value inherent to that void ratio.

Bolton (1986) investigated the angle of shearing resistance of sand. Extensive data of the strength and dilatancy of 17 sands in axisymmetric or plane strain at different densities and confining pressures were collated. It was stated the critical state angle of shearing resistance of soil which is shearing at constant volume is principally a function of mineralogy and can be readily be determined experimentally within a margin of about 1°,
being roughly 33° for quartz and 40° for feldspar. The extra angle of shearing of ‘dense’ soil is correlated to its rate of dilation and hence to its relative density and mean effective stress, combined in a new relative dilatancy index.

The effect of grading and anisotropy on sand was examined by Dunstan (1972), who used a mixing apparatus to form sand samples of different grading and then sheared the sand samples in a direct shear box in different directions. It was found that the strength of various gradings of sands at a similar relative density was fairly constant. Interactions of particles of differing sizes have little effect on the difference in strength caused by the anisotropic packing. It seems likely an anisotropic strength component will exist whatever the grading. It was also shown that the difference in shear strength because of anisotropy tends to decrease as the normal stress increases.

2.3.3 Steady State of Liquefiable Sand

The effect of stress path on the steady state lines of a liquefiable sand was investigated by Vaid et al. (1990). Results from undrained triaxial compression and extension tests on water-deposited sands show that steady state line of a given sand, though unique in the effective stress space, is not so in the void ratio-effective stress space. The sand is contractive over a much larger range of void ratios in extension than in compression. While a single steady state line emerges for compression loading, extension loading yields several lines, each characteristic to a given deposition void ratio. All these extension lines lie to the left of the compression line in void ratio-effective stress space. Thus at a given void ratio, steady state strength is smaller in extension than in compression, the difference increasing as the sand becomes looser.
2.3.4 Void Ratio Distribution in Non-cohesive Soils

Åberg (1992) presented a theory for calculation of the void ratio of non-cohesive soils and similar materials. The most important variable is the grain-size distribution of the soil, but the grain shape and the degree of densification were also considered by means of two empirical coefficients. It was stated that the void sizes in a graded soil are mainly determined by the fine grains, which successfully fill the space between the large grains.

2.3.5 Concept of Granular Void Ratio $e_g$ for Sand Mixtures

In the past, most laboratory studies were focused either on clay, or clean sands. Thus these two soils are the best understood. The void ratio, $e$, has long been recognized as the one of the key parameters for clean sand and clay. According to Ishihara (1993), $e$ alone decided the shear strength of sand. However, researchers have also noticed that in sand mixtures like clayey sand or silty sand, with varying fine contents, a unique relationship of shear strength and void ratio can no longer be found. To deal with this problem, the concept of granular void ratio, $e_g$ (Mitchell, 1976; Kenney, 1977) was developed. The granular void ratio is defined as:

$$e_g = \frac{\text{volume of voids} + \text{volume of clay}}{\text{volume of granular phase}} \quad (2.13)$$

The calculation of $e_g$ was illustrated by Wood (1990) in Figure 2-7. The main idea behind this granular void ratio concept is that in a mixture of sand with fines (silt or clay), the force-carrying skeleton is formed by the sand particles. The fines do not contribute to the shear strength so it can be treated as void. This concept has been widely accepted by researchers working on clayey sand and silty sand. However, it also has its limitations. As can be seen later in this thesis, this concept needs to be further improved to deal with OA.
2.3.6 Undrained behaviour of Clayey Sand

The undrained behaviour of clayey sands in triaxial compression and extension was studied by Georgiannou et al. (1990). It was found that the effect of the clay in a clayey sand will depend on several factors, including the grading of the sand, the mineralogy of the clay and the chemistry of the pore water, the clay content and the distribution of the clay within the soil.

In their research, the specimens were prepared by sedimenting Ham river sand into a kaolin suspension and then subject the samples prepared to anisotropic consolidation. Since the clayey sand had a higher granular void ratio than the clean sand, it exhibited undrained brittleness during shear in triaxial compression. Increase in clay fraction at constant \( e_g \) also reduced the stability of the fabric. However, for clay fractions up to 20\%, the clay did not significantly reduce the angle of shearing resistance of the granular component.

Undrained brittleness in triaxial compression reduced as the granular void ratio decreased and the material becomes non-brittle at granular void ratios lower than 0.75. Overconsolidation of a sedimented clayey sand reduced the undrained brittleness and increased the strains to peak in triaxial compression. Ageing of a normally consolidated sedimented clayey sand led to an increase in both peak strength and small-strain stiffness.

Ovando-Shelley and Pérez (1997) also studied the undrained behaviour of clayey sands (tamped Mexico sand with Kaolin) by using load controlled triaxial tests. The test results corroborated the Georgiannou et al. (1990)’s findings that small amounts of fine particles increase the potential for generating excess pore pressure during shearing and reduce strength and stiffness. The intergranular void ratio and the normalized stress state at
the onset of structural collapse and at the point of minimum strength were used to interpret the results.

Georgiannou et al. (1991) also compared the behaviour of clayey sands under undrained cyclic triaxial loading to the behaviour under monotonic loading. Effective stress paths for the normally consolidated soils loaded monotonically in triaxial compression and extension are shown to form a bounding envelope which determines the pattern of behaviour under cyclic loading.

2.3.7 Undrained Behaviour of Silty Sand

Zlatović and Ishihara (1995) studied the influence of non-plastic silt on undrained behaviour of silty sand. Toyoura sand was milled into a non-plastic silt and then the series of mixtures was made from the two materials, with increasing content of silt, from 0% to 100%. The specimens were prepared into the loosest state using three methods of preparation: moist placement, dry deposition and water sedimentation. Before saturation, with an increase in the silt content, the maximum void ratio increased, but after saturation the tendency changed. However, the contractiveness increased, the peak and residual strength decreased and the quasi-steady state line in the ln $p'\cdot e$ space moved downwards with the increase in the content up to 30%. With further increase in the silt content, contractiveness decreased, strength increased and the quasi-steady state line moved upwards.

Thevanayagam & Mohan (2000) presented a careful reanalysis of the basic concepts of critical state soil mechanics and its extention to silty sands. It was suggested that once a silty sand sample is under an external force, some of the grains, by virtue of
their size, heterogeneous positions, and availability of void space to adjust their positions without significantly affecting the adjacent particles, may not actively participate in transferring the normal forces or sustain significant shear forces. Thus the void ratio is an approximate index of active contacts at the ultimate steady state only.

A new set of intergranular and interfine state variables \((\psi_s, e_s\) and \(\psi_f, e_f)\) were introduced to characterize the stress-strain behaviour of silty soils. The \(e_s\) and \(e_f\) can be calculated from void ratio and fine content (FC):

\[
e_s = \frac{[e+(FC/100)]}{[1-(FC/100)]} \quad (2.14)
\]

\[
e_f = \frac{e}{(FC/100)} \quad (2.15)
\]

and \(\psi_s, \psi_f\) are intergranular state parameter and interfine state parameter respectively, which resemble the idea of the traditional state parameter, \(\psi\). This traditional state parameter for sands was proposed by Been and Jefferies (1985). It is defined as \(\psi = e - e_{ss}\), in which \(e\) is the void ratio of the sand, \(e_{ss}\) is the void ratio on the steady state line of the sand at the same normal stress.

Figure 2-8 shows the matrix effects on the behaviour of silty sands prepared at different void ratios and at different fines contents.

At low fines content:

As the void ratio increases, both \(e_s\) and \(e_f\) increase and it can be categorized into three groups according to the relationship between \(e_s\) and the maximum void ratio of the host sand, \(e_{max\_HS}\).

1) At low void ratios where \(e_s < e_{max\_HS}\), the active contacts sustaining shear forces are expected to be those of the coarser grains. A unique SSL (steady state line) may be
observed for all specimens when considered in terms of intergranular void ratio and
the parameters ($\psi_s$, $e_s$) may be used to describe the stress-strain characteristics.

2) At intermediate void ratios where $e_s$ is close to $e_{\text{max}, HS}$, whether or not the fines
actually lie within the intergranular voids or play the role of the separator between
some of the coarser grains dictates the anticipated soil behaviour. If the fines lie
within the intergranular voids, the soil may behave like case 1; otherwise it may
behave like case 3.

3) At high void ratios corresponding to $e_s > e_{\text{max}, HS}$, the fines may actively play the
role of separators between most of the coarser grains. Hence, the soil behaviour
may resemble that of the finer grains. A unique SSL may not be found due to
pressure sensitivity of the fine-grained contacts separating the coarser grains.

**At high fines content:**

As the fines content increases further, $e_s$ becomes very high and therefore the
influence of coarser grains diminishes and the soil is primarily governed by the
compressibility of the fines and shearing along the fines (shown as case 4 in Figure 2-8).

Thevanayagam *et al.* (2002) further developed this idea to recognize the
contribution of fines. It is postulated that fines either provide a beneficial secondary
cushioning effect or contribute to fragility, depending on the nature of the soil’s matrix
structure and the magnitude of $e_s$. At the same $e_s$, the fines that fall within the intergranular
voids provide a cushioning effect and slightly reduce the fragility. When fines fall between
some of the coarse grains and partially support the coarse grain skeleton, the soil is very
fragile.
2.3.8 Effect of Fine Material on Liquefaction Potential of Sand Mixture

Chameau and Sutterer (1994) summarized experimental observation and studies on the effects of fine materials on liquefaction potential. Empirical correlations suggest that sands containing fines should be generally characterized as less susceptible to liquefaction. However, the experimental data available to date, including static steady state considerations, suggest that the actual influence of fines is still difficult to predict and is a function of the fines composition, plasticity, content, and fabric. It is also concluded that the wide variability in the observed effect of fines on liquefaction potential suggests that SPT based procedures, while often conservative for design, do not adequately account for the effect of fines since fabric, plasticity, and clay content are not considered. Undisturbed specimens should be obtained to determine the in-situ void ratio, and to permit monotonic laboratory testing for steady state characteristics and generation of a bounding surface.

2.4 Sampling Effects

2.4.1 Research on ‘perfect sampling’

A ‘perfect sampling’ is an idealized process, which stands for the total removal of the deviator stress $q'$ under undrained condition. Stress changes during ‘perfect sampling’ were analyzed by Skempton and Sowa (1963) and the effect was studied using triaxial apparatus. Experimental investigation leads to the conclusion that if two identical specimens of saturated clay are subjected to different changes in total stress without alteration in water content, and if the strains consequent upon these stress changes cause little alteration in micro-structure, then the undrained strengths of the two specimens will be practically identical.
Santagata and Germaine (2002) also carried out ‘perfect sampling’ tests on Boston blue clay and the effect of sampling disturbance on the compression and undrained shear behaviour of the soil are quantified by comparison with the intact behaviour. The results indicate that release of shear stress associated with ‘perfect sampling’ only causes a modest change in the engineering properties of the soil.

### 2.4.2 Research on ‘ideal tube sampling’

Baligh (1988) computed the strains in tube sampling using the strain path method and found that at the center line of a sample tube, only triaxial compression and extension strain paths are involved. The magnitude of the compression and extension strain depends on width to thickness ratio. Such strain path tests are named ‘ideal tube sampling’ and can be performed in triaxial cell.

Santagata and Germaine (2002) found the effects of ‘ideal tube sampling’ are very significant and increase systematically with the amplitude of the strain imposed. An increase in disturbance causes a decrease in the compression ratio, a decrease in the undrained strength, and an increase in the strain at failure and the recompression ratio, but has a minor effect on the preconsolidation pressure. These effects derive from the decrease in effective stress and from the damage to the soil fabric that occurs as a result of sampling.

Clayton et al. (1998) also investigated the effect of ideal tube sampling and reconsolidation. Triaxial stress and strain path tests have been carried out on high-quality Laval and Sherbrooke samples of lightly overconsolidated Bothkennar clay. The specimens were instrumented with local axial and radial strain and mid-plane pore pressure measurement, with precautions being taken to retain the existing pore water chemistry. The
test programme imposed shear and volumetric strains of various magnitudes to assess the reduction in strength and stiffness caused by sampling and laboratory reconsolidation procedures. It was concluded that for Bothkennar clay, even very high quality tube samples will cause a significant loss in mean effective stress, and some loss of structure during sampling, as a result of the imposed undrained shear strain cycle. Re-establishment of the initial effective stress level, by an appropriate stress path, will recover a proportion of the undisturbed undrained compressive strength that depends on the amount of destructuring: stiffness can not be fully recovered. Reconsolidation procedures risk taking the specimen through the current yield surface, in which case, large volumetric strains will occur and be accompanied by significant destructuring and, hence, irrecoverable loss of strength and stiffness. For natural clays, such as Bothkennar clay, the variability of structure within the deposit means that normalization solely with respect to the effective stress cannot be used to allow for the disturbance caused by tube sampling, or to recover in-situ soil properties.

Hight and Georgiannou (1995) carried out researches on the effects of tube sampling on the undrained behaviour of clayey sands. Triaxial strain path tests, which follow the sequence of strains imposed by a sampling tube on elements on its centre line (Baligh, 1985) were described. Major reductions in effective stress are shown to occur as a result of sampling. Reconsolidation to in situ stresses causes significant reductions in granular void ratio and these, together with fabric changes, modify the behaviour of the sample from that of the in situ soil: undrained brittleness and the effects of ageing in-situ may be eliminated.
2.4.3 Research on Real Sampling Disturbance

Tan et al. (2002) examined results from triaxial unconfined compression tests and undrained compression tests on reconsolidated samples of a Singapore marine clay retrieved using two sampling methods that offer differing quality of samples. One method used soil samples obtained by thick wall sampler that is commonly used in Singapore and the other method used thin-walled samplers with very sharp ends that is used in Japan. Local strain measurements were made using a Hall-effect transducer in the triaxial tests. Bender elements were embedded in some of the samples to establish the maximum shear modulus. If the samples are not reconsolidated, the shear strength and stiffness determined from triaxial tests are found to be sensitive to the quality of the samples, and generally lower than that determined by in-situ tests. However, if the samples are subjected to isotropic or $K_o$ consolidation to the estimated in-situ condition, there is little difference between the shear strengths of samples retrieved using different samplers, and also consistent with results from vane shear tests. However, for the maximum shear modulus, even with reconsolidation, there is still a 10% difference between the results from samples retrieved using different samplers. Further, the laboratory determined maximum shear moduli are about 10% lower than the value determined in an in-situ seismic cone test.

2.4.4 Research on Laboratory Sample Preparation Procedures

Atkinson et al. (1992) investigated the disturbance caused by laboratory sample preparation. Undrained triaxial compression tests were carried out on samples of intact Bothkennar soil recovered using a Laval sampler. The laboratory test samples were cut from the Laval sample using different techniques including pushing a thin-walled tube,
trimming with a wire saw and various methods. The results show that the method of sample trimming had a significant influence on the strengths and stiffness measured. They indicate that, for this soil, sample preparation using wire saw trimming causes the least disturbance.

2.5 $K_o$ Values and In-situ Horizontal Stress

2.5.1 Research on $K_o$

Most of the research done on the coefficient of earth pressure at rest ($K_o$) were based on tests carried out in laboratory, using $K_o$ consolidation apparatus with lateral stress measurement. The soils tested were clean sand or clay. It is generally agreed that $K_o$ is a function of soil type and stress history and the findings can be summarized as the following:

1) If the soil is a normally consolidated clay or a loose sand, in first time (virgin) consolidation without lateral deformation (one dimensional $K_o$ consolidation), $K_o$ is constant and appears to be consistent with the empirical Jaky’s equation $K_o=1-\sin\phi'$ (Jaky, 1948; Bishop, 1958; Youd and Craven, 1975; Abdelhamid and Krizek, 1976; Mersi and Hayat, 1993). As shown in Figure 2-9, in first time $K_o$ consolidation from point O to A, the stress path is a straight line with a constant slope of $\sigma'_h/\sigma'_v$. According to Wood (1990), the nature of the angle $\phi'$ used in Jaky’s equation is somewhat uncertain; however, $K_o$ value depends on the in situ structure of the soil and can be expected to correlate with the peak angle of shearing resistance measured in triaxial compression. For normally consolidated clay or loose sand,
Since the peak angle of shearing resistance equals to the critical state friction angle, \( \phi' \) of the critical state can be used.

2) During unloading the \( K_o \) value increases and does not conform Jaky’s equation. Ladd (1965) reported that as a clay deposit is rebounded, \( K_o \) increases and becomes greater than unity for OCR values exceeding about 3.5±1; in other words the horizontal pressure becomes larger than the vertical pressure. Though \( K_o \) can be greater than 1, it cannot exceed \( K_p \), the passive coefficient of earth pressure. In the case of Hydrite 10 Georgia kaolinite, \( K_o \) in unloading increase with the increase in OCR and at a OCR of about 8, \( K_o \) approaches \( K_p \) (Abdelhamid and Krizek, 1976). As shown in Figure 2-9, the unloading from point A to O is a curved line above the first time loading line but \( K_o \) is less than \( K_p \) most of the time, only approaching \( K_p \) at big OCR values.

3) Various relationships of \( K_o \) and OCR in unloading were proposed. Wroth (1975) interpreted the rebound process by assuming a elastic behaviour, then got the following equation:

\[
K_o = OCRK_{onc} - (\nu'/1 - \nu')(OCR-1) \tag{2.16}
\]

in which \( K_{onc} \) is the \( K_o \) value in virgin compression, and \( \nu' \) is the poisson ratio of the soil.

Schmidt (1967) analyzed the data by Hendron (1963) for sands and proposed:

\[
K_o = bOCR^a \tag{2.17}
\]
Schmidt stated that parameter $a$ is in the range of 0.3 to 0.5, and it is independent of the initial density of sand. However, Mayne and Kulhawy (1982) proposed that for clay and sand, $a = \sin \phi'$. Mesri and Hayat (1993) reexamined that data and concluded that it should be

$$K_o = K_{om} OCR^{\sin \phi'_{cv}}$$

(2.18)

in which $K_{om}$ is the coefficient of earth pressure at rest at the point when unloading starts. Parameter $a$ in Equation 2.17 becomes $\sin \phi'_{cv}$, and $\phi'_{cv}$ is the critical state friction angle, which is independent of the initial density.

There were also other proposed relationships, like Parry’s empirical relation:

$$K_o = K_{om} OCR^{\phi'}$$

(2.19)

in which $\phi'$ is the friction angle expressed in radians.

4) During reloading, as noticed by Wroth (1975), the value of $K_o$ rapidly falls as there is little increase in $\sigma'_h$ for a substantial increase in $\sigma'_v$. The $K_o$ value can be less than $K_o$ in first time loading, but greater than $K_a$, the active coefficient of earth pressure. Shown in Figure 2-9, the reloading line from point O to A is a curved line below the first time loading line. $K_o$ in reloading gradually approaches a normally consolidated $K_o$ value (Mersi and Hayat, 1993). According to Dyvik et al. (1985), the normally consolidated $K_o$ value is reached well before OCR=1. This also indicates that for oedometer and direct shear tests on over-consolidated soils, the desired in-situ $K_o$ will not be attained by simply reloading the specimen back to the
in-situ effective vertical overburden stress. It is important to know the stress path experienced by the soil sample.

5) No significant difference in $K_o$ was observed for clay samples with different fabric in both loading and unloading (Abdelhamid and Krizek, 1976).

6) When a pre-sheared soil is subjected to the laterally constrained consolidation condition, it does not return to the first time $K_o$ loading line. The reason is that during preshearing, the soil’s structure suffers a permanent change, which persists even during the subsequent $K_o$ loading (Mesri and Hayat, 1993).

7) During repeated shear straining, $K_o$ increase with shear strain amplitude and cycle number (Youd and Craven, 1975).

### 2.5.2 Research on Pressuremeter Tests

Ménard invented a pre-boring pressuremeter to measure the in-situ soil stress and deformation in 1955 in France and published the first paper on pressuremeter tests (Ménard, 1957). The problem of deducing the in-situ soil parameter was studied by Palmer (1972) by interpreting the results of Ménard pressuremeter tests on in-situ soils. In the test, a cylindrical cavity was expanded by internal pressure, and the relation between applied pressure and cavity volume change is measured. It is shown that the results of a single test are enough to determine a complete stress-strain relationship in plane strain, using a simple graphical procedure. The only restrictive assumption necessary was that the deformation occurred under undrained conditions; there is no restriction to infinitesimal strain or to elastic perfectly-plastic soils. Gambin (1979) also analyzed Ménard pressuremeter test and provided ways to obtain limit pressure and pressuremeter modulus.
Viana da Fonseca and Almeida e Sousa (2001) investigated the coefficient of earth pressure at rest $K_o$ in saprolitic soils from granite using both Pre-Bored Pressuremeters (PMTs) and self-boring pressuremeters. The conclusion is that the horizontal stress measured by PMTs is not precise because of two reasons. One is the pre-boring process; the other is due to the design of the PMT, it cannot measure $K_o$ values lower than 0.5. However, Due to the sandy and hard nature of OA, it is difficult to use self-boring pressuremeters. Since OA has an over-consolidation stress history, the $K_o$ to be measured is probably over 0.5. Therefore, the PMT is still adopted used in this research on Singapore OA.

2.6 Discussion

The work of previous researchers sheds light on understanding the constituents and engineering behaviour of OA soil. However, there are still many issues, which need to be solved, and these are discussed below.

2.6.1 Heterogeneity of Singapore Old Alluvium

Great variability exists in Singapore OA both in vertical and horizontal direction, which is repetitively described by previous researchers and clearly seen in Figure 2-1. As stated by Gupta et al. (1987), the Old Alluvium of Singapore is a proximal facies of an extensive braided river deposit. The river flow was seasonal in nature, and large floods occurred on a regular basis. The violent and irregular deposition environment resulted in the heterogeneous OA and the heterogeneity was further built up by the post-deposition
lithification and weathering process. Thus, from places to places OA shows various density, cementation, particle size distribution and shear strength.

**Density:**

The measured density of OA varies between 18kN/m$^3$ and 22kN/m$^3$, with a mean value of 20.3kN/m$^3$. (Li and Wong, 2001). It was also noticed by Tan *et al.* (1980) that there was no correlation between randomly selected relative density values and their sampling depths plotted in relation to mean sea level.

The random distribution of density partly explains the non-relevancy between depth and OA shear strength. However, the range of density itself cannot solely account for the great difference in OA shear strength, which may vary in orders.

**Cementation:**

There is still no agreement on whether the material is cemented or not. PWD (1976) stated that the deposit is ‘usually uncemented’ and Tan *et al.* (1980) described the soil as ‘lightly cemented’. According to Dames & Moore, it is ‘with a trace of residual cementation’. The ultimate pile load tests of Chin *et al.*, (1985) also showed that the OA is not strongly cemented. Li & Wong (2001) reported that the soil is slightly cemented but the bond is fragile and can be easily broken.

On the other hand, Peart *et al.* (2001) reported that on the Singapore MRT Changi Airport Line, OA is entirely cemented and belongs to sandstone. However, the term cement was not clearly defined and no attempt was made to identify other effects such as suction. At the same time, Peart *et al.* (2001) admitted despite the soil’s compact nature, when the material comes into contact with free water there is a marked and often rapid
breakdown into its constituent part. In addition the material also breaks down rapidly under mechanical handling once in an unconfined state. This phenomenon clearly indicates that the cementation, of any, is weak. It can even be merely due to the maintained confining stress by suction in the soil. Once the suction is gone and the confining stress is zero, the soil is quite weak and different from a real strongly cemented soil. According to Li & Wong (2001), OA III is cemented. However, in undrained triaxial tests, the effective cohesion intercept value is only 30.3 kPa and therefore cementation is still in question.

As noticed by quite a number of researchers, free water has a great influence on the strength of Old Alluvium. Researchers on Kinta Valley deposits found that OA is plastic when wet, although rather hard when dry. Shirlaw et al. (2000) also reported that some Old Alluvium was hard and stable on first exposure, but deteriorated over night and required chemical grouting to stabilize it.

Singapore Old Alluvium is a material developed under the tropical weather and it shares many common features with other tropical soils. For example, Ho & Fredlund (1980) stated that a lot of the natural, steep slopes in Hong Kong stand well in dry seasons but collapse when there is a continuing precipitation; so slope failures are common in Hong Kong. It was explained that on the surface, residual soils are unsaturated in-situ, having negative pore-water pressures which contribute to their strength. Once saturated, the contribution of the suction is lost and the shear strength decreases.

In this thesis, cementation refers to the strong bonding between particles, not from the suction created by pore fluid. Thus, those materials that will lose strength when come in contact with free water is deemed to have no cementation. The effective cohesion ($c'$)
found by Dames & Moore (1983) can be regarded as due to cementation but it is also described to be extremely variable.

Based on the above discussion, it is concluded that both cemented and uncemented OA exist, and can be easily distinguished according to the behaviour once in contact with free water. So a dispersion test is designed to classify cemented and uncemented OA, which will be presented in the next chapter. In the present study, most of the research was carried out on uncemented OA, which from the review early on, is more dominant in Singapore.

Presence of Fines:

Even in the uncemented OA category, the soil is still heterogeneous, as can be seen in Figure 2-10. The uncemented OA can be seen as a sand mixture covering a wide range of particle size distribution, with particles ranging from gravel, sand, silt to clay. The pebbles were mostly quartz, vein quartz, cryptocrystalline silica and quartzite. Individual pebble-size grains of fresh alkali feldspar were occasionally found. The sand grains are of the same mineralogy, mainly made of quartz and feldspars. XRD tests showed that the clay mineralogy was a mixture of kaolinite, illite and smectite (Gupta et al., 1987). The silt faction is almost entirely quartz (Pitts and Gupta, 1992).

Researchers (Mitchell, 1976; Kenny, 1977) have long realized that fines (silt and clay) play an important role in the behaviour of sand mixture. Shown in Figure 2-8, there is a transition zone of fines content. If the fines content is higher than the transition zone, the influence of coarser grains diminished and the soil mixture’s behaviour is governed by the fines alone. If the fines content is lower than the transition zone, the mixture still behaves like sand and can be studied by treating the fines as void and using granular void ratio, $e_g$. 
Classification:

Due to the heterogeneity, shear strength of Old Alluvium is known to be highly variable. As shown in Figure 1-2, the shear strengths ranges from almost 0 to 700kPa and so far, no relationship of shear strength to depth, or void ratio, or confining stress can be found. However, most of the previous research works on OA didn’t recognize the importance of this heterogeneous nature. Though test results were reported, the reason behind the variability in the test results was not probed. Several researchers proposed practical equations relating the shear strength of OA and the SPT N-values, but these equations can only offer rough estimations and they do not help to understand the reasons for variation in strength.

Since OA has a natural heterogeneity which shows in cementation, density and fine presence, it is not appropriate to report the shear strength of OA indiscriminately. If the OA soil is first to be classified into different subgroups, it is expected that factors regulating the shear strength can be found. In the present research, OA samples are first classified using a dispersion test, which sort them into ‘cemented OA’ and ‘uncemented OA’ group. Then the ‘uncemented OA’ group can be further divided using PSD test results. According to the PSD data of Li and Wong (2001), low-fine-content OA makes up the majority and this thesis is focused on this representative soil. From the literature review, it seems this uncemented, low-fine-content OA can be studied using the granular void ratio, $e_g$. 
2.6.2 Limitation of Concept $e_g$

Though the concept of granular void ratio is insightful and plenty of research on this idea were carried out, there is still some limitations in the current research on clayey sand and silty sand. For example, research on natural sand mixture is rare and all the researches on sand mixture reviewed in this chapter were done on remoulded samples, which were often deliberately made gap-graded using selected grains and fines. When forming the remoulded sand, usually only one type of fines, either silt or clay, was added. No research was done on sand mixture containing different kind of fines, both silt and clay. Even for these remoulded sand mixtures with one fines, research was often done with only one fines content, paying little attention to whether the concept of $e_g$ can still hold with the same host sand but varying fines contents.

Thus, when using this idea to characterize a natural soil such as the uncemented OA, since the soil is continuously graded and contains different fines at different content, the concept of $e_g$ needs to be verified, and modified if necessary. In Chapter 6 of this thesis, $e_g$ is developed and a new parameter $e_{ge}$, the equivalent granular void ratio is proposed. The main difference between $e_{ge}$ and $e_g$ is that $e_{ge}$ acknowledges differing contribution of clay and silt, thus making it possible to be used in OA, a natural sand mixture containing both silt and clay of varying amount.

2.6.3 Sampling and In-situ Stress State

The soil samples used in the present research are mostly natural soil samples retrieved from the ground. During the sampling process, inevitably some changes in stress and strain are imposed on the soil. Laboratory tests are performed on these sampled soil specimens
and thus may not reflect the in-situ properties directly. Understanding of the OA soil is not complete without a knowledge of the stress and strain changes in the sampling process.

To understand the sampling process, two closely related aspects need to be investigated. One is the initial in-situ stress state, the starting point from which the sampling process takes place. For natural soils, the vertical stress state is easier to estimate but the in-situ horizontal stress (or $K_o$) is one of the most difficult problems in geotechnical research. As can be seen from the above literature review, $K_o$ is a function of soil type and stress history. Because of the heterogeneous nature of the soil and the over consolidation stress history caused by erosion, the wide range of measured OA field $K_o$ data is a reasonable consequence. Again, most of the research data on $K_o$ in literature are laboratory studies on reconstituted soil samples. In this research, both laboratory and field tests will be carried out. First, the $K_o$ values of reconstituted clean sand and clayey sand are measured in first time loading, unloading, and reloading. It is hoped that such knowledge will help to predict the in-situ stress state. Then, in-situ horizontal stress studies using pressuremeter tests were carried out at Kim Chuan Site, to measure the in-situ horizontal stress in Singapore OA. The predicted and measured $K_o$ values will be compared.

The other important aspect in the sampling process is the stress and strain disturbance imposed on the in-situ soil, which will also be studied using remoulded and intact OA soil samples. The process of ‘perfect sampling’ or ‘ideal tube sampling’ can be simulated in triaxial tests using reconstituted samples. However, real sampling practice is much more complex than the theoretical approach. Many activities in real sampling may cause additional disturbance and need to be studied. The best way to study sampling disturbance is to compare samples taken with different sampling techniques. OA Samples
from Kim Chuan site will be taken with different techniques and the behaviour will be evaluated to reveal sample quality and sampling disturbance.

2.6.4 Summary

The goal of this research is to investigate the geotechnical properties of Singapore OA and to provide some guidance to engineering practice in this soil. Through literature review, it is found that the previous researches on OA neglect one important aspect of OA, the inherent heterogeneity which shows in cementation, density and particle size distribution. The previous practice of reporting the test results indiscriminately added to the confusion about OA properties. Thus, it is proposed in this thesis to develop a classification framework to sort OA into different groups. Since the uncemented, low-fines-content OA forms the majority, the present research will focus on this OA group.

When working with this uncemented, low-fines-content OA, the idea of granular void ratio $e_g$ seems promising. However, the concept of $e_g$ also has several deficiencies and has not been tested against a natural soil yet. This concept need to be further developed before it is used to characterize OA soil.

Needless to say, proper understanding of natural stress state and sampling disturbance is indispensable to a proper soil characterization. Both laboratory and field tests will be used to investigate these topics. Currently, information of in-situ stress state and sampling disturbance about a natural soil is limited in literature and it is hoped the present research will provide valuable data in this field.
Figure 2-1  A Face section through the Old Alluvium with a selection of morphological features identified (after Gupta et al., 1987)
Figure 2-2  Views of Sunderland during the Pleistocene showing the shoreline estimated by Biswas (1973) (after Gupta et al., 1987)
Figure 2-3  Classification of the Old Alluvium: a) all OA; b) OAI; c) OAIi; d) OAIii. (after Li and Wong, 2001)
Figure 2-4 CIU strength for Old Alluvium: a) OAI; b) OAII c) OAIII (after Li & Wong, 2001)

Figure 2-5 Undrained stress-strain behaviour of clean sand (a) $p' - q$ plot (b) $\varepsilon_d - q$ plot
Figure 2-6  Undrained behaviour of Toyoura Sand at same void ratio and different confining stress (adapted from Ishihara, 1993)

Figure 2-7  Concept of granular void ratio $e_g$ (adapted from Wood, 1990)
Figure 2-8 Intergranular matrix phase diagram: (a) cases 1-4; (b) effect of fines on soil matrix at constant \( e \) (adapted from Thevanayagam and Mohan, 2000)
Figure 2-9  Stress path of soil in $K_o$ consolidation (after Mersi and Hayat, 1993)

Figure 2-10  Particle size distribution of uncemented OA from Kim Chuan, BH-1