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ENGINEERING PROPERTIES OF OLDER ALLUVIUM

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Dedicated to beloved parents, my lovely wife, my son Elyas, my daughter Taraneem, my grandfathers, my grandmothers, my brothers, my sisters, my sister in law and my family. Thanks for all your love and supports.

Badee Alshameri

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ABSTRACT

Moisture content is one of the most crucial factors influencing soil and rock strength. This paper deals with the effect of moisture content on strength of older alluvium under dry, wet and saturated conditions. Older alluvium is semi cemented eroded deposited and reshaped by water to make non-marine setting. Specimens were tested in for shear strength, hardness and point load index. According to these results, the petrophysical properties of older alluvium decrease with increasing moisture. The strength was extremely reduced after the moisture content increased over the range of natural moisture content i.e. at saturated condition. For soil mechanics and soil engineering projects the shear strength, friction angle and cohesion assess at dry condition in order to give classification for soil strength. While the design parameters (shear strength, friction angle and cohesion) were taken at weak condition i.e. saturated and wet condition. However the difficulties and non reliable preparing regular samples at laboratory, most of samples destroyed during the sample preparation. Point load apparatus and Schmidt (rebound) hammer test did not able to record any reading during test the samples for both wet and dry condition. Older alluvium shows equilibrium between distribution of the clay/silt and gravel with percent finer approximately 38% and 38.5% respectively, and lower presence of sand with percent finer approximately 23.4%. The range of natural moisture content was within range of 17.98 to 19.65%. The results revealed that moisture content have great influence in the reduction of the shear strength τ, friction angle Ø and cohesion c. When the moisture content on older alluvium deposits increased the shear strength reduced to 22.3% and to 75.3% at wet and saturated condition respectively (the shear strength equal to 57.4kPa and 18.3kPa for wet and saturated condition respectively) in comparison to the magnitude of shear strength at dry condition (shear strength at dry condition equal to 74.1kPa). The same as for friction angle, when the moisture content increased the friction angle reduced to 18.6% and 66.9% at wet condition and saturated condition respectively (friction angle equal to 55.19^o and 22.45^o for wet and saturated condition respectively) in comparison to the magnitude at dry condition (at dry condition friction angle equal to 67.83^o). Otherwise the effective of increase the moisture content at cohesion is different i. e. the magnitude of cohesion at dry condition was equal to 21.044 kPa. At wet condition the cohesion increased to 12.7% (cohesion equal to 23.71kPa) in comparison to the magnitude at dry condition. At saturated condition the cohesion value will decreased to 54.6% (cohesion equal to 9.54 kPa) in comparison to the magnitude at dry condition.

ABSTRAK

Kandungan lembapan ialah salah satu faktor penting yang mempengaruhi kekuatan tanah dan batu. Kajian ini dibuat bagi mengkaji kesan kandungan lembapan terhadap kekuatan Alluvium tua dalam keadaan kering, basah dan tepu. Alluvium tua ialah separa tersimen. Spesimen diuji untuk kekuatan ricih, ketahanan dan indeks beban titik. Keputusan uji kaji menunjukkan sifat petrofizikal alluvium yang berkurangan apabila kelembapan meningkat. Kekuatannya menurun dengan mendadak selepas kandungan lembapan meningkat melebihi daripada kadar yang sepatutnya, sebagai contoh ketika dalam keadaan tepu. Kebiasaannya, rekabentuk mekanik tanah dan kejuruteraan tanah, kekuatan ricih, sudut geseran dan kejelikitan dibuat ketika keadaan kering dengan tujuan untuk mengklasifikasikan kekuatan tanah. Walaubagaimanapun, parameter reka bentuk (kekuatan ricih, sudut geseran dan kejelikitan) sangat terubah ketika keadaan tepu dan basah. Kesukaran dan cara pengambilan sampel yang tidak betul menyebabkan kebanyakan sampel musnah. Alat Beban Tumpu dan Ujian Hentakan Schmidt tidak dapat mencatatkan sebarang bacaan ketika uji kaji sampel dilakukan dalam keadaan basah dan kering. Alluvium tua menunjukkan persamaan di antara agihan untuk tanah liat dan batu kerikil, peratus halus di antara 38% dan 38.5%, manakala untuk pasir, peratus lulus ialah 23.4%. Kebiasaannya, bacaan untuk kandungan lembapan yang asal ialah di antara 17.98% ke 19.65%. Keputusan menunjukkan kandungan lembapan memberi kesan kepada pengurangan kekuatan ricih τ, sudut geseran Ø dan kejelikitan c. Apabila kandungan lembapan untuk mendapan alluvium tua ditingkatkan, kekuatan ricih berkurangan kepada 22.3% dan 75.3% dalam keadaan basah dan tepu (kekuatan ricih bersamaan dengan 57.4kPa dan 18.3kPa untuk keadaan basah dan tepu) dengan membandingkan dengan kekuatan ricih dalam keadaan kering (kekuatan ricih ketika kering bersamaan dengan 74.1kPa). Begitu juga dengan sudut geseran, apabila kandungan lembapan meningkat, sudut geseran juga berkurangan kepada 18.6% dan 66.9% dalam keadaan basah dan tepu) dengan membandingkan dengan magnitud dalam keadaan kering (sudut geseran bersamaan dengan 67.83% dalam keadaan kering). Walaubagaimanapun, kandungan lembapan efektif dalam keadaan jelekit adalah berbeza. Sebagai contoh, magnitud kejelikitan dalam keadaan kering adalah bersamaan dengan 21.044kPa. Dalam keadaan basah, kejelikitan telah meningkat kepada 12.7% (kejelikitan bersamaan dengan 23.71kPa) dengan membandingkan magnitud dalam keadaan kering. Dalam keadan tepu, nilai kejelikitan akan berkurangan kepada 54.6% (kejelikitan bersamaan dengan 9.54kPa) dengan membandingkan dengan magnitud ketika keadaan kering.

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SYMBOL		DEFINITION
A	-	Initial area of the specimen
c	-	Cohesion
C_u	-	Uniformity coefficient for soil particles
C_c	-	Coefficient of gradation (curvature) for soil particles
D_{10}	-	Diameter of soil particles at percent finer 10%
D_{30}	-	Diameter of soil particles at percent finer 30%
D_{60}	-	Diameter of soil particles at percent finer 60%
d_f	-	Estimated horizontal displacement at failure, mm (in this study it assumed as = 5 mm)
d_r	-	Displacement rate, mm/min
F	-	Shear force
I.D.	-	Iron deposits leaching into the relict structure and file it
Is	-	Point load index (index of strength)
M_1	-	Mass of container + wet soil
M_2	-	Mass of container + dry soil
M_c	-	Mass of container
M_s	-	Mass of dry soil
M_w	-	Mass of water
N	-	Normal vertical force acting on the specimen
n	-	Normal stress
O.A.	-	Older alluvium
PLT	-	Point-load test

SYMBOL		DEFINITION
R	-	Rebound number
R.S.	-	Relict structure
RH	-	Schmidt (rebound) hammer test
S	-	Degree of saturation
SDI	-	Slake durability index
SDT	-	Slake durability test
SHI	-	Shore hardness index
t_{50}	-	Time required for the specimen to achieve 50 percent consolidation under the specified normal stress (or increments thereof), min
t_{90}	-	Time required for the specimen to achieve 90 percent Consolidation under the specified normal stress (or increment thereof), min
t_f	-	Total estimated elapsed time to failure, min
UCS	-	Uniaxial compression strength
w	-	Moisture content
W.G.	-	Weathering granite
μ	-	A susceptibility coefficient
σ	-	Total normal stress
σ'	-	Effective normal stress
σ_{c0}	-	Dry uniaxial compression strength
σ_{csat}	-	Fully saturated uniaxial compressive strength
τ	-	Shear strength
θ	-	Volumetric water content of soil
ϕ	-	Friction angle
γ	-	Unit weight of rock

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CHAPTER 1

INTRODUCTION

1.1 General Concept

A geotechnical engineer must take precautions when the materials at hand cannot be classified as rock or as soils in terms of their behaviour in slopes or in civil engineering works in general. In their in situ form, the geologic formations may have appearances that imply rocklike behaviour but behave very much different when it is subjected to saturated condition. Older alluvium or semi cemented sediment which was eroded, deposited and reshaped by water in a non-marine setting has this characteristics. Once disturbed, this formation may degrade to soil-size particles in a time frame and their engineering properties will deteriorate drastically, that is relevant to the long term performance of slopes built in or in other civil engineering work. The wide distribution for older alluvium in Malaysia creates problems in many field of construction such as excavation, slope stability and foundation in understanding their engineering characteristics especially the changes in dry and wet condition. The water content is known as one of the most important factors lowering the strength of rocks. A small increase in the water content may lead to a marked reduction in strength and deformability (Erguler and Ulusay, 2009).

Study in basic engineering properties such as the grain size distributions, hardness, strength, durability and shear strength parameters (cohesion c, friction angle ϕ) is important to understand the behaviour for the older alluvium and avoid the inherence problems (David, 2007). Many previous researchers, Abdul Shakoor and Barefild, 2009; Engin et al., 1998; Vásárhelyi and Ván, 2006; Romana and Vásárhelyi, 2007; Edward and Abdulshakoor, 2006; Namdar, 2010; Joseph et. al., 2009 studied the changes of engineering properties for igneous and sedimentary rocks but very minimal works has been carried out for older alluvium. Thus, this research is carried out to study the effect of water content to the shear strength, durability and strength parameters c, ϕ of the older alluvium. Determining the characteristics of this material is essential for effective evaluation of the behaviour of subsurface as a whole for many civil engineering applications (Torok and Vásárhelyi, 2010; Mohd For, 2008).

In general, the point load index I_s and uniaxial compressive strength UCS will decrease by increase of moisture content (Vásárhelyi and Ván, 2006; Adnan, 2008; Margaret Kasim and Abdul Shakoor, 1996). In addition, Edy Tonnizam et al. (2008) noted the increase of water absorption with weathering grade. Neyde Fabiola et al. (2003) found that micro-morphological features in kaolinitic soils were related to compaction, increased tensile strength, penetrometer resistance, bulk density and hard setting behaviour. Fine particles of silt and clay form structural connections between sand particles and as the material dried out the strength of these connections increased (Mathieu Lamotte et al., 1997). Namdar (2010) compared between several types of mixed soil from the mineralogy, optimum moisture content OMC, cohesion of soil, friction angle of soil and soil bearing capacity, and he found that the soil cohesion decreases continuously with reduction of clay minerals in the soil.

In rock engineering projects, the effect of moisture content is important for the safety and stability of slopes and underground openings. In addition, for conservation and reclamation of ancient buildings and monuments, determination of the effect of the moisture content on rock strength has a prime importance. This behaviour is more pronounced in fine-grained sedimentary rocks, particularly in clay-bearing rocks. Engineering properties of the rocks (i.e. the grain size distributions, hardness, strength, durability and shear strength parameters) are very important parameters for rock classification and design of structures either upon or inside rock. In addition, they are essential for judgment about their suitability for various construction purposes. Some rock is weakened by the addition of water, the effect being a chemical deterioration of the cement or clay binder.

1.2 Importance Of The Study

This material have become notorious as a result of the numerous foundation, slope stability, excavation and embankment failure problems with which they are often associated. Most of these problems resulted from the change of moisture content. By increasing the water content, the older alluvium exhibit significant reductions in strength and deformability. Thus, by understanding the behaviour of this material will certainly help in the designing stage with the actual performance of this material.

1.3 Problem Statement

This case study is represent of one of this statement, an older alluvium at Desa Tebrau, south of Johor state in Malaysia showing different engineering properties for the same material within different conditions (dry, wet and saturated). The older alluvium behaviour at dry condition as rock, otherwise, at saturated condition it become week. In rock and soil engineering projects, the effect of moisture content is important for the safety and stability of slopes and underground openings. In addition, for conservation and reclamation of ancient buildings and monuments, determination of the effect of the moisture content on rock and soil strength has a prime importance.

1.4 Objectives

The objectives of this research are:

1 - To investigate the occurrences and basic engineering properties of the older alluvium (i.e. the grain size distributions, hardness, durability and moisture content)

2 - To determine the shear strength and shear strength parameters (friction angle $\emptyset$ and cohesion c) of the older alluvium under dry, wet and saturated condition.

1.5 Scope And Limitations

This case study should focused on study some engineering properties of older alluvium:

-Collect the sample from the site location and description the older alluvium at field.

-Field test applies by the Schmidt hammer test.

-Laboratory tests should be include point-load test PLT, slakes durability test SDT, moisture content and direct shear strength test for the samples at different conditions.

-Laboratory tests also should be include wet sieve analysis.

-Determine the rebound number R, point load index (index of strength) I_s and slake durability index SDI for different condition than compare between its.

-Determine the shear strength τ and shear strength parameters (friction angle $\emptyset$, cohesion c) for the samples at both conditions from the laboratory tests.

CHAPTER 2

LITERATURE REVIEW

2.1 Geological Background

At general alluvium is loose unconsolidated soil or sediments, eroded deposited and reshaped by water to make non-marine setting. But this older alluvium are semi consolidated, and classified to two kinds of beds a1 overlay a2, otherwise to recognise this type of alluvium from the young one, it called older alluvium. Table 2.1 shows comparison between the older alluvium and alluvium at Johor state.

Table 2.1 : Comparison between the older alluvium and alluvium at Johor State.

Name	Older Alluvium	Alluvium
Age	Pleistocene	Recent to Sub-Recent
Description	- Semi –Consolidated sand and clay - Boulder beds	Unconsolidated
Components	Type a1 : Boulder beds Type a2 : Gravel , Sand and Clay	Gravel, Sand and Clay
Origin	Fluviatile and Shallow-marine	Fluviatile and Shallow-marine

The previous geological surveyed and geological map (Burton, 1973) are mention to:

a . Old alluvium are located at south Johor.

b . In general it consist of coarse feldspathic (which it come from granite source) sand with occasional rounded phenoclasts also there are represented for the gravelly clay, sandy gravel, sandy clay, silty clay, clayey sand and clay. It contains phenoclasts (fragment from rocks) of vein quartz, quartzite, sandstone, siltstone, shale hornfels, granite, granite porphyry, alaskite, aplite, rhyodacite, andeiste and tuff.

c . The condition of fresh older alluvium can be described as partly consolidated argillaceous member are intermediate between clay and mudstone and most the arenaceous are intermediate between sand and sandstone.

d . In general, for the structure it can organise semi-flat lying with some traces for gentle folder which have less than 15^0 slope. Therefore there are some bedding steeply inclined to vertical for a few feet's in small tight folds.

e . For Palaeogeography and age, the old alluvium is occur during Pleistocene period. However there some evidence the direct to the shallow marine environment such as occurrence of plant remains and echinoid spines .

2.2 Relation Of Moisture Content With Rebound Hammer (R), Point Load Index (I_s) and Slake Durability Index (SDI)

One of the indicated index of engineering properties is rebound number. Adnan (2008) mentioned from the experimental methods that moisture content of the rock within the zone of influence of impact may considerably affect the rebound values depending on its microstructural character. Moisture facilitates inter-grain sliding and leads to softening of grains and loose skeletal bonding (plasma) holding the grains together. These mechanisms are most effective in weathered, porous, loosely cemented and/or mudrocks but may also be significant in fresh crystalline rocks with abundant intra-grain microcracks. When the purpose of the SH tests is to derive correlations between UCS and/or E and rebound values, all tests should be carried out at the same moisture content (Adnan, 2008).

The point load Index I_s can be useful to conduct uniaxial compressive strength UCS easily and faster than use uniaxial compression test. Otherwise at rocks there are clear influence of moisture content for the strength of rock so the rock mechanics and rock engineering projects will be change by changing at moisture content. In general, the point load index and uniaxial compressive strength will decrease by increase of moisture content (Ibrahim and Sefer, 2008; Vásárhelyi and Ván, 2006).

Moreover the slake durability test SDT can be used to estimate uniaxial compressive strength UCS. SDT is easy conduct because is not require any sample preparation and the results are repeatable providing that operator. Basic to empirical reviews that the slake durability index will decrease if the moisture content increase, and there are many prospect reasons explain how that be such as: The water can soften the bonds or interact with mineral surfaces and alter their surface properties. The pore water pressure may cause instability along weakness planes. The water decrease frictional shearing resistance. Influence of the water for the behaviour of the clay minerals at rock.

Table 2.2 shows effect of various structural, mineralogical and water transmission properties on strength, hardness and durability (Engin et al., 1998).

Table 2.2 Factors affecting UCS, SHI and SDT (Engin et al., 1998).

	Compressive strength	Shore hardness	Slake durability
Microstructure			
Grain shape (angularity)	+	no effect	–
Grain size			
Coarse	–	no effect	+
Fine	+		–
Degree of alignment	–	no effect	–
Packing density (dense)	+	+	+
Fabric type			
Skeletal	–	–	–
Turbostratic	+	+	+
Sutured/straight grain to grain contact	+	+	+
Porosity			
High	–	no effect	depends on permeability
Low	+	no effect	depends on permeability
Degree of bonding			
Well bonded	+	+	+
Weakly bonded	–	–	–
Mineralogy			
Grains	x	x	x
Cement and bonding material			
Quartz	+	+	+
Clay minerals	–	–	–
Miscellaneous			
Permeability			
High	–	no effect	–
Low	+		+
Post depositional factors (diagenesis and metamorphosis)	+	+	+
Water content			
High	–	–	–
Low	+	+	+
Sample moisture			
Natural	–	–	–
Oven-dried	+	+	x
Soft/soluble minerals	–	–	–
Microfractures	–	no effect	–
Inclusions	–	no effect	no effect

(+) An expected increase in test result.(–) An expected decrease in test result.(x) Depends on type of minerals.

2.3 Relation Of Moisture Content With Weathering And Strength

Table 2.3 shows that the effect of moisture content is more signification when the rock yield to high weathering activity, on other words, the effective of moisture content will be more accuracy when the weathering grade increases. That will be related to the clayey mineral in the rock material because when the weathering grade increases, it becomes more dominant due to the decomposition of the original minerals, so the porosity of the material increases. Followed that exist more void and pores within the grains at rock, which would assist the absorption of moisture within the rock material. However, as weathering increases, dry unit weight decreases and water adsorption increases (Edy Tonnizam et. al., 2008).

Moisture content is an important factor that affects the strength of the weak rock materials. The effect is more obvious on grade IV materials where the dry and

Table 2.3 : Summary of the physical and mechanical properties with weathering grade (Edy Tonnizam et. al., 2008).

Rock Type	Location	Weathering Grade	Dry unit weight (kN/m^3)	Water adsorption after 15min soaking (%)	Total porosity (%)	Strength Reduction (%)
Fine Sandstone	Bukit Indah and Mersing	II	23.6-	<5%	6.2-10.3	6 - 13
		III	23.0-	<5%	6.4-12.2	16 - 20
		IV a	23.3-	<5%	12.3-	22 - 41
			23.8	5-10%	12.5-	44 - 49
		IV b	22.3-	5-10%	13.5-	51 - 61
			23.5	10-15%	13.7-	62 - 70
		V a	22.0-	10-15%	>15.0	70 - 82
			22.4	15-20%		83 - 84
		V b	21.3-	15-20%	>15.0	85 - 92
			22.3	20-25%		92 - 94
				25-30%		94 - 97
				30-35%		98 - 100
	Desa Tebrau	IV a	23.5-	5-10%	12.3-	38 - 48
			23.7	10-15%	12.5-	46 - 69
	Kempas	Va	22.1-	<5%	>15.0	77 - 90
			22.5	5-10%		92 - 94

wet materials can significantly affect the strength of rock materials. The fresher samples grade II show the least absorption of moisture while grade V shows the most absorption. The percentage of water absorption increases with increasing weathering grade.

2.4 Relation Of Moisture Content With Mineralogy

Some researchers tried to connected the influencing for increase moisture content on the engineering properties such as UCS to the amount of clay minerals at the rock, i.e. Steiger and Leung (1990) gave some data extracted from an EXXON comprehensive research program on shale typical properties, see Table 2.4. Shale G is composed by a 50% of smectite, which can explain the big drop in strength and the simultaneous increase in surface area (Romana and Vásárhelyi, 2007).

Hawkins and McConnell (1992) published a paper analyzing the sensitivity to water saturation of several mechanical properties of 35 British sandstones. Their results have been revised by Vásárhelyi and Ván (2006) that they found a clear correlation between saturated and air dry unconfined strengths.

Table 2.4 : Data on UCS of typical shales (Steiger and Leung, 1990).

Shale	Dry UCS (MPa)	Sat UCS (MPa)	Decrease	Surface area (m^2/g)
E	96,5	44,8	32 %	2,2
F	82,7	27,6	67 %	3
G	34,5	3,5	90 %	10

Lau et al. (1993) carried out a study on the effects of temperature and water saturation on the mechanical properties of the Lac du Bonnet granite. The water saturated specimens were observed to display lower stress values associated with crack initiation when compared with dry specimens. The reduction was in the order of 13% and was explained as due to the very low permeability, the undrained test conditions and the increase of pore water pressure during loading (Romana and Vásárhelyi, 2007).

Ajalloian and Karimzadeh (2003) described the engineering properties of Givi dam foundation on andesitic rocks. Unconfined compressive test were performed both in saturated and dry condition in samples of the right bank. The reduction in strength was in the order of 18% (Romana and Vásárhelyi, 2007).

Sachpazis (2004) collected representative samples of Bernician Great limestone (England) from four different metamorphic degrees, toward marble: A, none; B, low; C, high; D, completely metamorphised. Several geomechanics tests were performed, both in dry and saturated conditions. The mean results for unconfined compression tests are shown in the Table 2.1, all the samples were very strong (Romana and Vásárhelyi, 2007).

Vásárhely and Ván (2006) have studied systematically the reduction in unconfined compressive strength (also in deformation modulus) when saturating different rocks. Their results are shown in Figure 2.1 and Table 2.5.

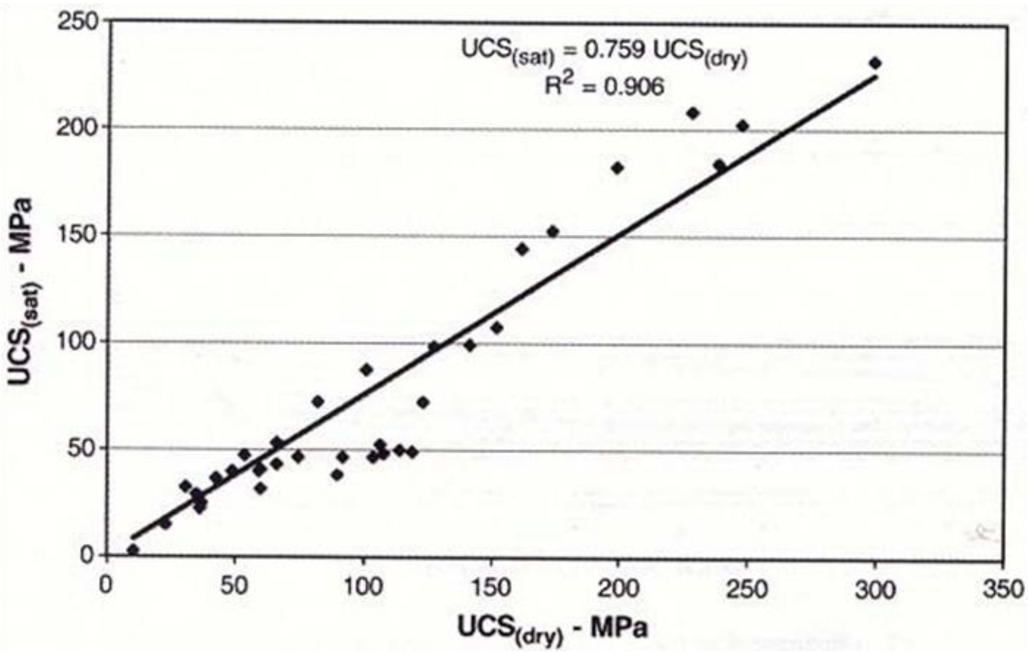


Figure 2.1 Saturated UCS vs. dry UCS in British sandstone samples (Vásárhely and Ván, 2006).

Table 2.5 : Decrease in UCS of saturated (Romana and Vásárhelyi, 2007).

Rock type	Decrease (%)	$\sigma_{sat}/\sigma_{dry}$	$\sigma_{sat}/\sigma_{dry}$ Reference	Year
Sivac marble	7	0,93	Vásárhely and Ledniczky	1999
Volcanic tuffs	27	0,729	Vásárhely	2002
Miocene limestone	40	0,659	Vásárhely	2005
British sandstones	30	0,759	Vásárhely and Vanon Hawkins and Mc Connell	2006

Romana and Vásárhelyi (2007) found from the data that they collected, the following geomechanics classifications, see Table 2.6.

Table 2.6 : Geomechamics classification for the some rocks type with ratio of UCS_{sat}/UCS_{dry} (Romana and Vásárhelyi, 2007).

Rock type	UCS_{sat}/UCS_{dry}
well indurated strong rocks	7
cemented medium strength rocks	27
soft argillaceous rocks	40

Abdul Shakoor and Barefild (2009) connected between the mineralogy and the ability to absorption of water, and they found that Stronger, lower absorption sandstones show a trend of consistent, linear reduction in unconfined compressive strength with increasing degrees of saturation, see Tables 2.7 and 2.8. For weaker sandstones, the majority of unconfined compressive strength is lost between 0% and 20% degrees of saturation, and only minimal or irregular strength losses occur at higher degrees of saturation.

Table 2.7 : Results of petrographic analysis (Abdul Shakoor and Barefild, 2009).

Sandstone	Quartz	Feldspar	Rock Fragments	Matrix	Muscovite Mica	Cement Hematite	Cement Calcite	Pore	QFR Percentages Quartz	QFR Percentages Feldspar	QFR Percentages Rock Fragments	Matrix %
Berea A	196	20	61	9	2	1	3	8	70.8	7.2	22.0	3.0
Berea B	200	23	45	4	0	10	0	18	74.6	8.6	16.8	1.3
Black Hand	199	34	21	22	0	0	0	24	78.3	13.4	8.3	7.3
Cow Run	162	35	63	1	0	7	0	32	62.3	13.5	24.2	0.3
Homewood	172	1	99	9	18	0	0	1	63.2	0.4	36.4	3.0
Juniata	103	6	118	5	0	48	0	20	45.4	2.6	52.0	1.7
Lower Freeport	180	24	47	2	1	18	0	28	71.7	9.6	18.7	0.7
Lower Mahoning	176	58	25	1	0	0	0	40	68.0	22.4	9.6	0.3
Sharon	219	34	23	3	0	1	0	20	79.3	12.3	8.4	1.0

Modal composition (N = 300). QFR = quartz, feldspar, rock fragments.

Table 2.8 : Mean values of engineering properties for each sandstone (Abdul Shakoor and Barefild, 2009).

Sandstone	Dry Density (pcf)*	Absorption (%)	Bulk Specific Gravity	Bulk Specific Gravity (Saturated Surface-Dried)	Effective Porosity (%)	Mean Dry Unconfined Compressive Strength (psi)**
Lower Freeport	134.3	4.33	2.17	2.27	9.31	5,870
Berea A	135.0	5.42	2.17	2.29	11.72	10,846
Homewood	152.7	2.95	2.44	2.52	7.23	14,385
Juniata	157.5	1.63	2.53	2.57	4.12	21,464
Berea B	131.0	6.06	2.11	2.24	12.72	6,718
Sharon	132.9	5.53	2.14	2.26	11.77	7,116
Lower Mahoning	137.9	3.62	2.22	2.30	8.00	11,459
Cow Run	129.1	6.05	2.09	2.22	12.51	2,824
Black Hand	133.2	5.41	2.15	2.26	11.54	7,585

*pcf = pounds per cubic foot; 1 pcf = 0.016 g/cm^3.
**psi = pounds per square inch; 1 psi = 0.006895 MPa.

Moreover, high-strength and low-absorption sandstones (1–3 percent), namely the Homewood sandstone and Juniata sandstone, display the most drastic unconfined compressive strength reductions with increasing degrees of saturation. Strength reductions between the mean dry unconfined compressive strength and saturated states range as high as 62.6% and 71.6% for the Homewood sandstone and Juniata sandstone, respectively. Other sandstones display lesser, but still significant, strength reductions between 20% and 40%. However, it can be declare that there is a clear reduction in unconfined compressive strength between the dry and saturated states for all sandstones studied. Also, the unconfined compressive strength trends to reduction vary between sandstones (Abdul Shakoor and Barefild, 2009; Margaret Kasim and Abdul Shakoor, 1996).

2.5 Relation Of Moisture Content With Strength Parameters And Uniaxial Compressive Strength (UCS)

Vásárhelyi (2003) analyzed the published data for the measured uniaxial compressive strength, and for the tangent and secant deformation moduli, in the case of dry and fully saturated conditions and showed that there is a linear correlation between the dry and fully saturated uniaxial compressive strengths, σ_{c0} and σ_{csat}, respectively as shown in Figure 2.2. The overall best-fit equation for the 35 investigated sandstones is:

$$\sigma_{csat} = 0.759\,\sigma_{c0} \quad (R^2 = 0.906)$$

where

σ_{csat} is fully saturated uniaxial compressive strength

σ_{c0} is dry uniaxial compression strength

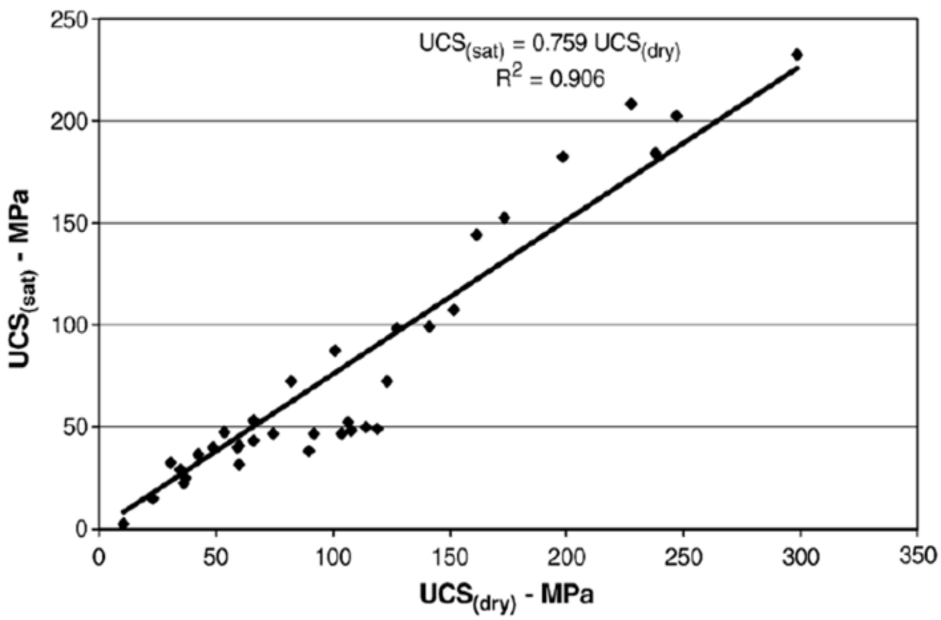


Figure 2.2 Relationship between the dry and the saturated uniaxial compressive strength (UCS) for 35 British sandstones (Vásárhelyi, 2003).

Figure 2.3 shows the best-fit lines plotted for the 15 different rock types for water content values up to 5%. It is apparent that the strength of the rock is very sensitive to the water content, an increase in water content of as little as 1% from the dry state can have a marked effect on strength.

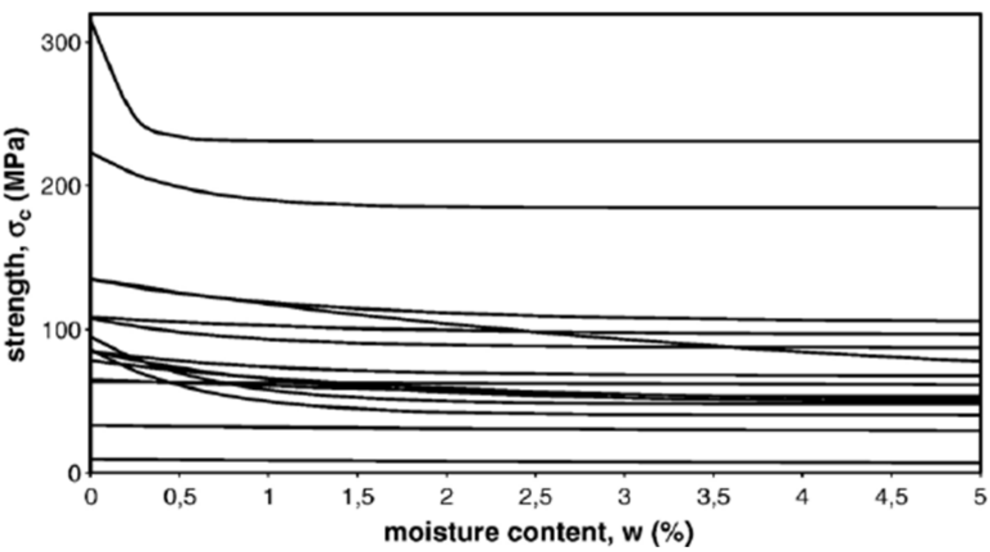


Figure 2.3 Strength–moisture content curves, fitted to experimental data (Vásárhelyi, 2003).

2.6 Relation Of Moisture Content And Mineralogy With Strength Parameters

Namdar (2010) compared between several types of mixed soil from the Mineralogy, optimum moisture content (OMC), cohesion of soil, friction angle of soil and soil bearing capacity and he found that the soil cohesion decreases continuously with reduction of clay minerals in the soil as shown in Tables 2.9, 2.10 and 2.11 also Figure 2.4. Form the evaluation of mixed soil types 2 (consist of 55% red soil and 45% of light brown soil), mixed soil types 3 (made up from 55% of red soil and 45% of black soil) and mixed soil types 4 (developed from 55% red soil and 45% of green soil) which are with lowest cohesion and from mineralogy evaluation of these three mixed soil types could be conclude the mixed soil types 2 and 3 due to availability of carbonate mineral could be observed of significantly reducing soil cohesion and in the mixed soil type 4 also one or more minerals presented in the green soil which have negative affected on the soil cohesion. However, Namdar (2010) found that the illite, muscovite, saponite, sauconite presented in the red plastic soil play main factors in soil cohesion as soil cohesion also the carbonate has negative in soil cohesion which other mineral may be similar in the green soil.

Adeniran and Babatunde (2010) carried out investigation on wetland (Fadama) soil properties affecting optimum soil cultivation. A cone penetrometer and a shear vane apparatus 19 mm were used to determine the cone index and the torque that cause the soil to shear at different moisture contents. The study shows that the cone index and shear vane of Fadama soils increased with depth and decreased with increase in moisture content. High moisture content reduced the soil cohesion. The internal friction angle of the soil was 37.9^{o}. The following values were obtained for soil cohesion 112kN/m^2, 62kN/m^2, 38kN/m^2, 30kN/m^2, and 12kN/m^2 at moisture contents of 0%, 5%, 10%, 15% and 20% respectively. Moisture content between 10% and 15% (dry basis) appeared ideal for cultivation of the soil. For this soil the critical moisture content was found to be 23.72%. Moisture content beyond the critical level needs to be drained before cultivation is carried out.

Table 2.9 : Minerals of soil sample (Namdar, 2010).

Sl. No	Soil Name	Minerals in the soil sample
1	Red soil	quartz, illite, muscovite, saponite, sauconite and carbonate- fluorapatite
2	Black soil	quartz, pyrophyllite, carbonate- fluorapatite and orthochamosite
3	Yellow soil	quartz, brucite, clinochlore and sandoite
4	Light brown soil	quartz and carbonate
5	Dark brown soil	nacrite, odinite, amesite, chamosite and biotite
6	Green soil	quartz, cancrisilite, chamosite, orthochamosite and brucite

Table 2.10 : Mixed soil types (Namdar, 2010).

Model No	% Of Red Soil	% Of Black Soil	% Of Green Soil	% Of Dark Brown Soil	% Of Yellow Soil	% Of Light Brown Soil
1	100	0	0	0	0	0
2	55	0	0	0	0	45
3	55	45	0	0	0	0
4	55	0	45	0	0	0
5	55	0	0	45	0	0
6	55	0	0	0	45	0
7	90	2	2	2	2	2
8	80	4	4	4	4	4
9	70	6	6	6	6	6
10	60	8	8	8	8	8
11	50	10	10	10	10	10
12	70	10	10	10	0	0
13	70	10	10	0	10	0
14	70	10	10	0	0	10
15	70	10	0	10	10	0
16	70	10	0	10	0	10

Table 2.11 : Mixed soil type under loose and optimum moisture content OMC condition (Namdar, 2010).

Model No	Optimum Moisture Content (%)	γ_d (KN/m^3)	Φ Degree	C (KN/m^2)	S. B. C (KN/m^2)
1	11.2	10.8	27	10	279.61
2	14.56	11.2	26	2	336.07
3	22.39	11.35	24	6	171.96
4	18.86	11.62	31	4	324.93
5	14.56	14.41	20	10	157.56
6	14.23	11.08	28.5	10	326.59
7	16.83	10.11	32	10	445.97
8	18.27	10.6	25	8	199.20
9	16.76	11.8	20	24	243.72
10	20.21	12.23	17	14.5	142.12
11	18.68	11.2	21	14	178.69
12	19.34	11.5	21	10	166.03
13	16.55	9.99	23.5	20	291.38
14	21.14	11.27	18	19	191.16
15	20.79	12.89	13	20	145.73
16	16.31	10.05	26.5	8	230.78

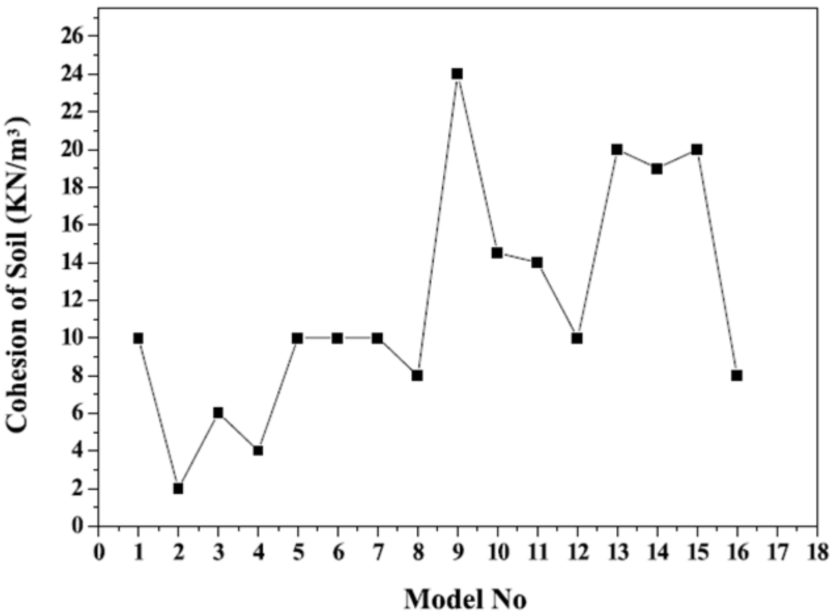


Figure 2.4 Cohesion of soil Vs model no (Namdar, 2010).

Kemper and Rosenau (1984) found that the cohesional forces associated with water are in the range to be able to account for measured moduli of rupture in moist soils. However, high moduli of rupture of soils such as the Billings, when oven dry, indicate formation of solid phase bonds at particle-to-particle contacts. Increases of aggregate stabilities and moduli of rupture with time of storage or "curing" under air-dry conditions, indicate that migration of bonding components to strengthen these bonds continues even when there is as little as one molecular layer of water on the mineral surfaces as shown in Figures 2.5 and 2.6.

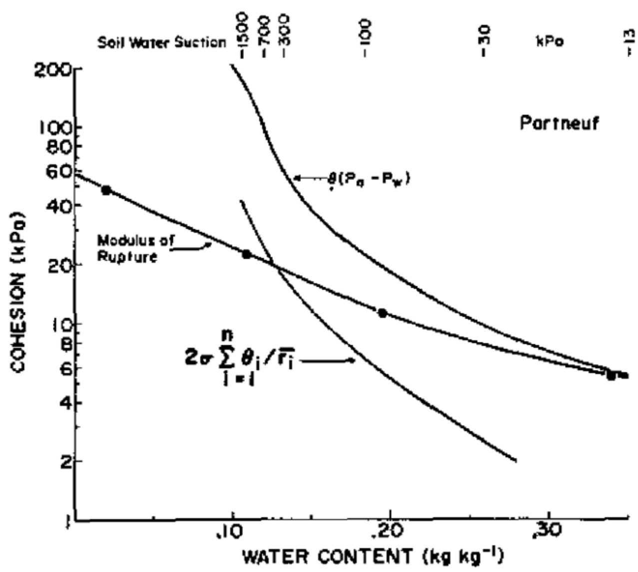


Figure 2.5 Comparison of measured moduli of rupture of Portneuf to estimated cohesion due to surface and hydraulic tensions (Kemper and Rosenau, 1984).

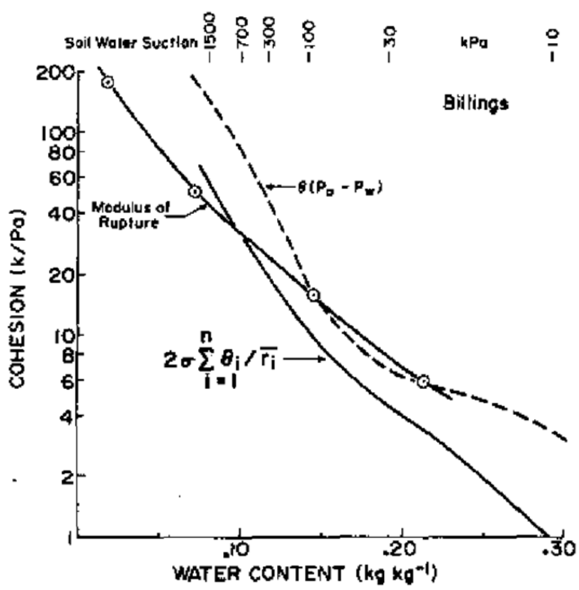


Figure 2.6 Moduli of rupture of Billings soil and estimated cohesion due to surface and hydraulic tensions (Kemper and Rosenau, 1984).

Matsushi and Matsukura (2006) found that the shear strength of the soils clearly decreased with an increase in moisture content. Table 2.12 summarizes the test results for the specimens with varied moisture conditions (the six moisture conditions referred to as A-F in the sand-soil samples and G-L in the silt-soils). Average volumetric water contents ranged from 0.04 to 0.43 for the sand-soil (7.6-84.4% saturation) and from 0.14 to 0.52 for the silt-soil (22.0-93.1% saturation).

Figure 2.7 shows the results with a simple linear regression for each of the groups. Table 2.13 lists the values of y-intercept and the inclination of the regression lines, i.e. cohesive strength and angle of shearing resistance in terms of simple linear-regression (Matsushi and Matsukura, 2006).

The inclinations of the regression lines are largest in the driest conditions and drastically decrease for the wetter samples, converging at 25 and 33 in the sand soil and 27 and 31 in the silt-soil as shown in Figure 2.7 and Table 2.13. In other words, the angle of shearing resistance of the moist soils seems to be constant, independent of volumetric water content, except in the driest condition (Matsushi and Matsukura, 2006).

The y-intercepts of the regression lines decreased with increasing moisture content and approached a minimum value at the saturated condition from 25.4 to 4.4 kPa in the sand soil, 37.8 to 5.2 kPa in the silt soil as shown in Table 2.13 (Matsushi and Matsukura, 2006).

Table 2.12 : Moisture contents and shear test results of each specimen group (Matsushi and Matsukura, 2006).

Group	Moisture condition	Volumetric water content (m^3/m^3)		Degree of saturation (%)		Normal stress (kPa)	Shear strength (kPa)
Sand-soil							
A	Oven-dried (40°C)	0.03	Av. (0.04)	6.7	Av. (7.6)	10	33.2
		0.03		4.8		20	40.5
		0.03		6.4		30	53.8
		0.07		12.4		40	65.6
B	Air-dried (25°C)	0.11	(0.11)	19.6	(20.7)	10	32.5
		0.13		24.3		20	36.8
		0.10		20.0		30	40.6
		0.09		18.8		40	40.1
C	Natural water content	0.18	(0.16)	35.7	(30.3)	10	20.4
		0.16		31.6		20	26.2
		0.15		28.8		30	29.9
		0.14		25.2		40	40.1
D	Add 6 ml water	0.24	(0.24)	47.9	(47.2)	10	13.3
		0.25		49.4		20	20.0
		0.24		46.2		30	25.9
		0.24		45.4		40	32.3
E	Add 13 ml water	0.37	(0.36)	73.3	(70.2)	10	12.6
		0.36		69.5		20	15.1
		0.36		67.2		30	22.8
		0.36		70.9		40	25.5
F	Capillary saturation	0.40	(0.43)	79.2	(84.4)	10	11.3
		0.42		85.1		20	16.6
		0.43		85.0		30	21.2
		0.40		77.6		40	26.8
		0.44		83.6		10	8.9
		0.46		89.4		20	15.9
		0.41		80.6		30	20.2
		0.46		94.9		40	27.6
Silt-soil							
G	Oven-dried (40°C)	0.15	(0.14)	25.0	(22.0)	10	45.1
		0.06		9.9		20	75.0
		0.15		24.9		30	77.0
		0.18		28.3		40	88.3
H	Air-dried (25°C)	0.28	(0.28)	45.8	(47.0)	10	32.1
		0.29		46.6		20	35.2
		0.29		48.8		30	41.2
		0.28		46.8		40	66.0
I	Natural water content	0.40	(0.38)	68.8	(66.6)	10	15.5
		0.37		66.3		20	22.6
		0.36		64.8		30	29.9
		0.37		66.3		40	32.5
J	Add 5 ml water	0.44	(0.45)	75.1	(75.9)	10	13.7
		0.45		77.4		20	19.3
		0.45		77.6		30	24.6
		0.43		73.6		40	29.5
K	Add 10 ml water	0.47	(0.48)	81.4	(83.3)	10	13.4
		0.48		84.0		20	17.8
		0.49		83.4		30	24.2
		0.47		84.4		40	28.3
L	Capillary saturation	0.53	(0.52)	94.9	(93.1)	10	10.9
		0.52		95.8		20	15.9
		0.53		92.7		30	20.2
		0.49		89.2		40	26.8

Table 2.13 : Shear strength parameters obtained by a simple linear regression for the data set of each specimen group (Matsushi and Matsukura, 2006).

Sand-soil						
Group	A	B	C	D	E	F
y-intercept (kPa)	20.1	25.4	13.2	6.9	7.2	4.4
Inclination (°)	48.4	30.5	32.6	32.7	25.3	29.6
Silt-soil						
Group	G	H	I	J	K	L
y-intercept (kPa)	37.8	16.2	10.3	8.4	7.9	5.2
Inclination (°)	53.3	47.7	30.7	28.2	27.5	27.9

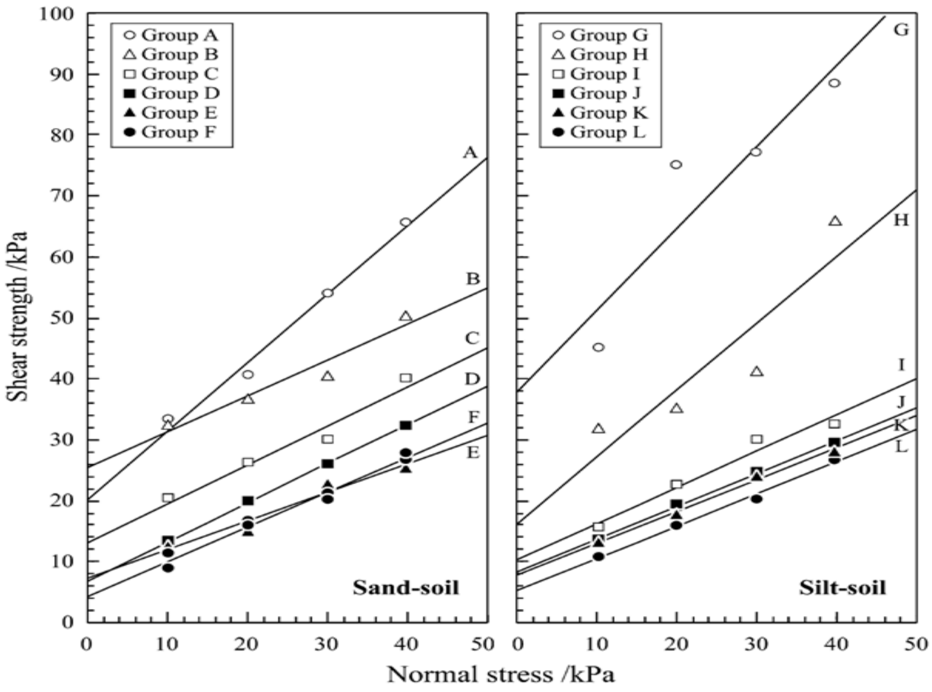


Figure 2.7 Results of the shear tests, the solid lines indicate the simple linear-regression lines for each specimen group (Matsushi and Matsukura, 2006).

The cohesive strength of an unsaturated soil was formulated as an exponential function of volumetric water content. In the formulation, shear strength s was expressed as:

$$\tau = \sigma' \tan \emptyset' + c\, e^{-\mu\theta}$$

Where :

σ'	is	net normal stress,
$\emptyset'$	is	effective angle of shearing resistance,
c	is	maximum cohesion,
μ	is	a susceptibility coefficient a
θ	is	volumetric water content of soil.

An advantage of this formulation is that all the parameters required are available without any elaborate soil testing. The variables can be obtained by a basic shear test and a subsequent regression analysis. In the case of the two undisturbed residual soils reported here, the predictive errors of the equation are less than a few kilopascal. It is considered that this empirical method provides a convenient alternative for engineering practice (Matsushi and Matsukura, 2006).

In another side, the grain size analysis have important influence on the behaviour of soil. For example the fine material have effective influence on the strength parameters such as the cohesion that the small area for the fine particle make power of attract between the particles is high especially when there are water between the particles. However, the coarse material have effective influence on the strength parameter such as friction angle that the resistance between the relative big particles tend to be against the movement of soil.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Introduction

In order to make this project more effective, useful, easier and typical it managed and organised at several steps and stages, see Figure 3.1. On other side, the literature review was extended in order to got the new information that related to the project. The standards and tools and equipments such as Schmidt hammer, geological hammer, geological compass and digital camera were used in-situ during the field visit (Burt, 2007; Paul et al., 2001).

During the field and laboratory tests, the steps were reordered and documented by a digital camera to improve the effectiveness of explanation. Some of the steps on the research were change to became correspondent with the new situation in site and with ability of samples for tests. For example: The tests at wet condition for the point-load test and slake durability test were ignored because the results from the Schmidt hammer which indicated the weakness and unreality of samples on wet condition for the those tests. The standards which used were, ASTM5873, 2005 for Schmidt hammer test, ASTM D5731, 2008 for point-load test, ASTM D 4644, 2008 for slake durability test, BS1377: Part 2: 1990 for sieve analysis (wet sieving), ASTM D4959, 07, 2000 for natural moisture content, ASTM D3080, 2004 for direct shear test, (Daniel and Charles, 2007; Kalinski, 2006 ; Verwaal, 2004; Eugene and Renk, 1999; Khan, 2008; Roots of Peace, 2008; Biscontin, 2007; Alkhafaji, 1992).

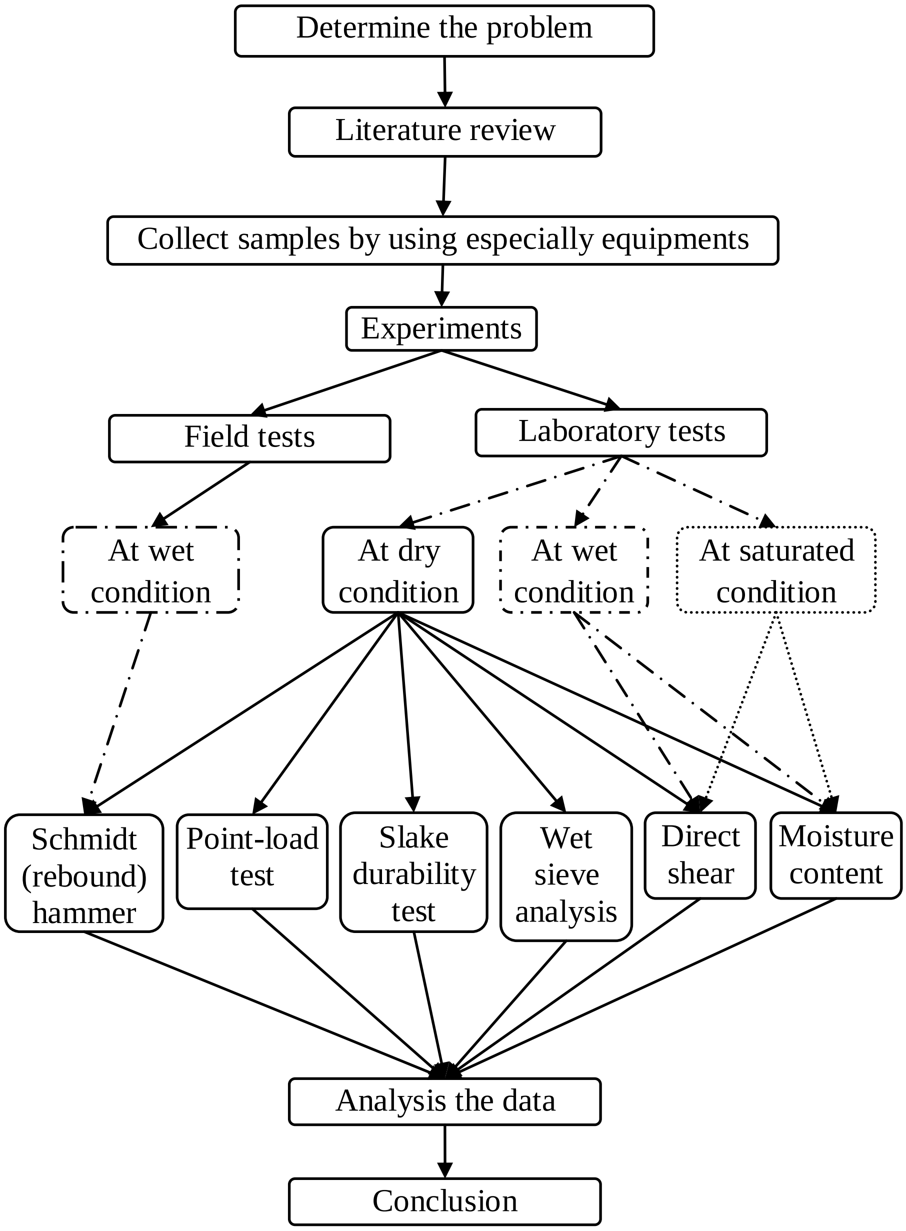


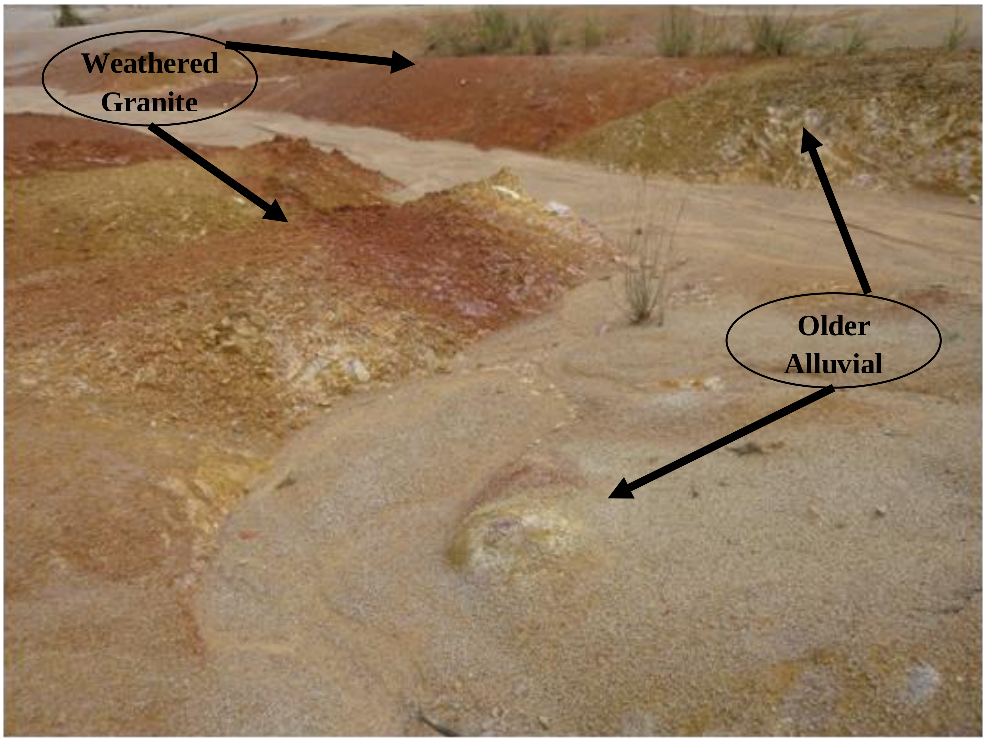
Figure 3.1 : Flow chart of research methodology.

3.2 Site Visit And Material Sampled

During the site visit, the general condition for site location was recorded and observed. The colour of older alluvium and the weathered granite were recorded. The geological structures were also recorded by using a digital camera and a geological compass. Moreover the samples from the material which filled the joints were studied, see Figures 3.2, 3.3 and 3.4.

Figure 3.2 Photograph showing older alluvium found at Desa Tebrau, Johor.

The boundaries between the older alluvium and weathered granite were recorded as shown in Figure 3.3.



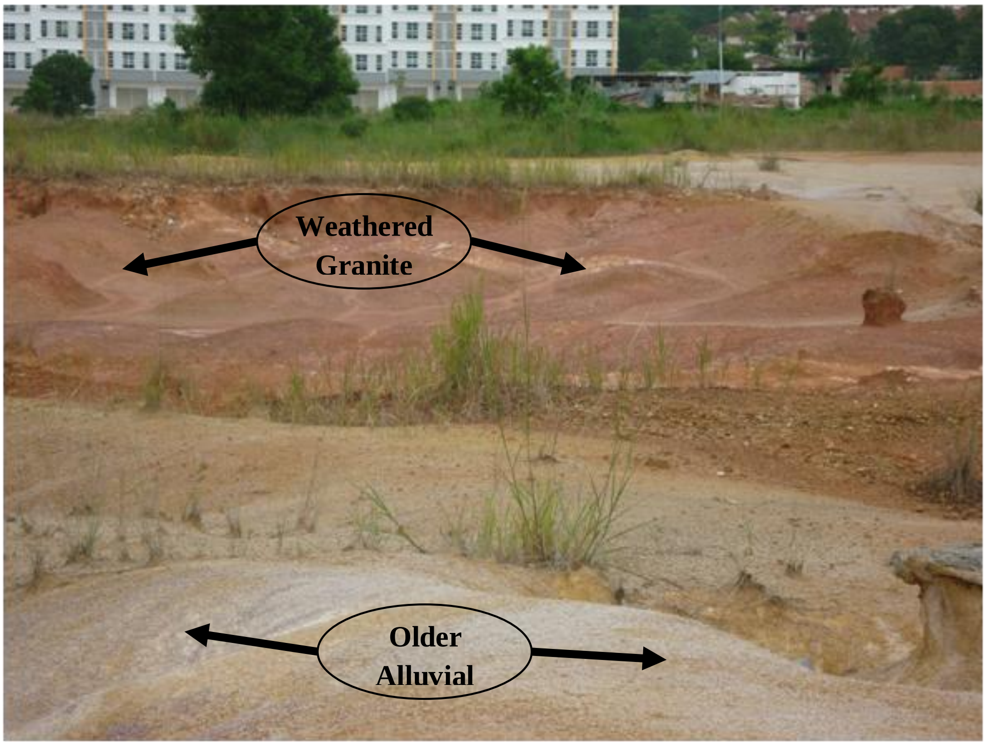
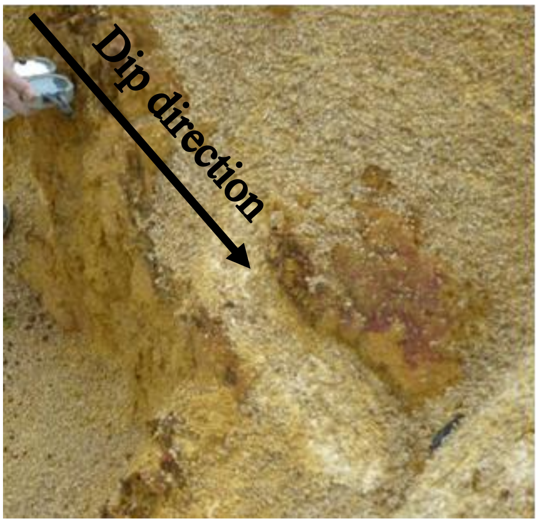


Figure 3.3 The boundaries between the older alluvium and weathered granite.



(a)

(b)

Figure 3.4 Photography showing measuring of
(a) Dip direction, (b) Dip angle (slope).

The material was sampled from an exposed outcrop after levelling of earthwork. A total of 35 samples were collected on the surface of the outcrop. The profiles were described using ISRM (1981) suggested method. Intact samples of measuring approximately 30 cm in length, 30 cm in height and 20 cm in width, were collected from site, then sealed in plastic and hessian bags for transport to the laboratory.

During collection of samples for direct shear test several difficult were encountered such as: First, after collected the irregular samples the processes of reshaped was very difficult and impractical even by using several methods and equipments, so the only way to reshaped by extracted directly with regular shape in-situ. Second, extract the regular samples in situ required to fabricated tools get the perfect shape for sample. Third, the weathering had effect at the potential of extracted samples as well as the quality of samples so the good samples was collected after rainfall at location.

3.3 Field tests

3.3.1 At Wet Condition

The samples at location were wet and no possibility to test or take dry samples, so the field test was applied on the wet condition only. The test which carried out at field was Schmidt (rebound) hammer test.

3.3.1.1 Schmidt (Rebound) Hammer (RH)

The proposer from RH is testing the surface hardness of rock sample. By using Schmidt hammer L-type, as shown in Figure 3.5, which is a portable and simple equipment to handle. RH is index test (indirect) strength test. The sample were blocks. At this test the index data which obtained is rebound number R.

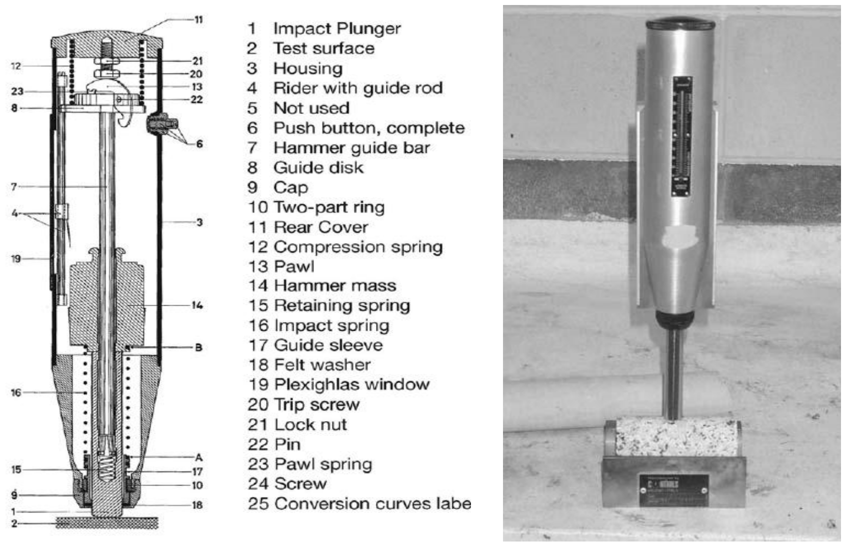


Figure 3.5 Schmidt (rebound) hammer test (Vellone and Merguerian, 2007) .

Rebound number R, is related to the surface strength of rock sample tested (Mohd For, 2008; Daniel and Charles, 2007):

$\log_{10}JCS = 0.00088\ \gamma\ (R) + 1.01$

where

JCS	is	joint compressive strength
γ	is	unit weight of rock
R	is	rebound number

3.3.1.1.1. The Procedure For Schmidt (Rebound) Hammer Test

The details about the procedure and classification for this test found at ASTM5873 (2005). While the all the sample on side was wet and no dry samples, the test was applied on the wet samples and the rebound number was recorded, see Figure 3.6 (Adnan, 2008).

Figure 3.6 Measuring surface of hardness of older alluvium by Schmidt (rebound) hammer.

3.4 Laboratory Tests

3.4.1 At Dry Condition

The first step to prepared the dry samples was by dry. Several samples were placed onto oven at a temperature of 110±5^{o} C. The samples were dried for one week, the long period for drying the samples was for guarantee full dried of samples. The samples extract and started applied the laboratory test at dry condition, see Figure 3.7.

Figure 3.7 First step to prepare the dry samples by placed the samples into the oven.

3.4.1.1 Schmidt (Rebound) Hammer (RH)

For theory refer to section 3.3.1.1

3.4.1.1.1 The Procedure For Schmidt (Rebound) Hammer Test

Refer to 3.3.1.1.1 for theory and details about the procedure and classification for this test.

The dry samples were tested by using Schmidt (rebound) hammer, then the rebound number R was recorded, see Figure 3.8.

Figure 3.8 Measuring surface of hardness by Schmidt (rebound) hammer test at the laboratory.

3.4.1.2 Point-Load Test (PLT)

This method for determining point load index I_s, then determine strength classification of rock materials through an index test. Sample can be core or irregular block. Equipment is easy to handle and portable, so the test can be undertaken in-situ, see Figure 3.9. The data which obtained from this test consider as index properties for strength of sample tested. A simple test and therefore, no constraint on number of test that can be carried out. The index value I_s can be converted to uniaxial compressive strength UCS (Mohd For, 2008; Ibrahim and Sefer, 2008):

$UCS \approx 24\ I_s$

or $UCS \approx 26\ I_s$ for granite

or $UCS \approx 18\ I_s$ for sandstone

where

UCS is unaxial compressive strength

I_s is point load index (index of strength)

Figure 3.9 Point-load tester.

3.4.1.2.1 The Procedure For Point-Load Test

The details about the procedure and classification for this test found at ASTM D 5731 (2008).

For point-load test the samples were prepared into two types of samples regular (cylindrical i.e. core sample) and irregular (rectangular i.e. cubic sample) shape. Moreover, the test applied only on the dry samples, see Figures 3.10 and 3.11.

Figure 3.10 Preparation the core samples for point-load test.

Figure 3.11 Preparation the irregular shape samples for point-load test.

The apparatus was prepared then the irregular shape sample was tested on the point-load test apparatus then the result was recorded, see Figures 3.12 and 3.13.

Figure 3.12 Preparation the point-load apparatus.

Figure 3.13. Applied the pressure over the irregular sample by the point-load apparatus.

3.4.1.3 Slake Durability Test (SDT)

The propose for SDT is to determine the durability of weak or soft rocks subjected to cycles of wetting and drying. In another words, to determine ability the rock sample to resistance of weakening and disintegration when subjected to drying and wetting (weathering process). The stronger is the rock the higher in slake durability index SDI, see Figure 3.14 (Edy Tonnizam et al., 2008; Mohd For, 2008).

Figure 3.14 Slake durability apparatus.

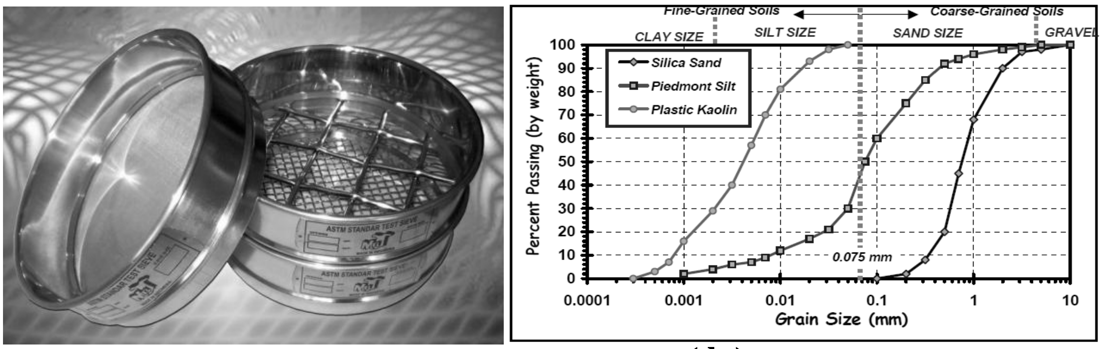
The slake durability was cancelled because the wet and dry samples were fully destroyed when submerged in water for ten minutes even without applied any movement for the samples, see Figures 3.15 and 3.16 (Edy Tonnizam et al., 2008).

Figure 3.15 Submerging the sample into water.

Figure 3.16 The samples fully destroyed after submerged in water for ten minutes.

3.4.1.4 Sieve Analysis (Wet Sieving)

Wet sieving was carried out, because the older alluvium samples contains fine material less than 75µm. Thus, when samples of older alluvium dry, fine particles of silt and clay can stick to sand and gravel size particles and cannot be separated by dry sieving, even if prolonged. Washing is the only practicable means of ensuring complete separation of fines for a reliable assessment of their percentage. However, the test carried out to determine the percentage of various grain sizes. The grain size distribution is used to determine the textural classification of soils (i.e., gravel, sand, silty clay, etc.). The parameter which come from this test consider as Basic properties. The distribution of different grain sizes affects the engineering properties of soil, see Figure 3.17.



(a) **(b)**

Figure 3.17 Sieve analysis (a) Sieves , (b) Representative grain size curves for several soil types (Paul W., 2001).

3.4.1.4.1 The Procedure For Wet Sieving

The details about the procedure and classification for this test found at BS1377 Part 2 (1990), see Figures 3.18, 3.19, 3.20 and 3.21 (Kalinski, 2006).

Figure 3.18. Mixed the sodium hexametaphosphate with the sample during carried out the wet sieving.

Figure 3.19 Starting the wet sieving by using sieve size 425**µ**m set over sieve size 63**µ**m.

Figure 3.20. Brushed and washing the wet particles during carried out the wet sieving.

Figure 3.21 Brushed the dry particles during carried out the last stage of wet sieving.

3.4.1.5 Moisture Content

The test carried out to determine the moisture content of a soil samples, as was sampled in the field or at the moment of testing for the accurate determination of in-situ water content. The moisture content is the ratio, expressed as a percentage of the mass of pore or free water in a given mass of soil to the mass of the dry soil solids.

For many soils, the moisture content may be an extremely important index used for establishing the relationship between the way a soil behaves and its properties. The consistency of a fine grained soil largely depends on its moisture content. The moisture content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil.

3.4.1.5.1 The Procedure For Moisture Content

The details about the procedure and classification for this test found at ASTMD4959, 07 (2000). This procedure used also for wet and saturated condition (Kalinski, 2006).

3.4.1.6 Direct Shear Test

The shear strength is one of the most important engineering properties of a soil, because it is required whenever a structure is dependent on the soil's shearing resistance. The shear strength is needed for engineering situations such as determining the stability of slopes or cuts, finding the bearing capacity for foundations, and calculating the pressure exerted by a soil on a retaining wall.

The direct shear test is one of the oldest strength tests for soils. In the laboratory, a direct shear device will be used to determine the shear strength of a soil. From the plot of the shear stress versus the horizontal displacement, the maximum shear stress is obtained for a specific vertical confining stress. After the experiment is run several times for various vertical confining stresses, a plot of the maximum shear stresses versus the vertical (normal) confining stresses for each of the tests is produced. From the plot, a straight line approximation of the Mohr-Coulomb failure envelope curve can be drawn, $\emptyset$ may be determined, and for cohesionless soils (cohesion c = 0), the shear strength can be computed from the following equation:

$$\tau = \sigma \tan\emptyset$$

where

τ	is	shear strength
σ	is	total normal stress
$\emptyset$	is	friction angle

The direct shear test measures peak and residual direct shear strength as a function of the stress normal to the plane of shearing. The results are utilised in, for example, equilibrium analysis of slope stability problems or for the stability analysis of dam foundations, tunnels and underground openings, see Figures 3.22, 3.23 and 3.24 (Evert Hoek, 2000).

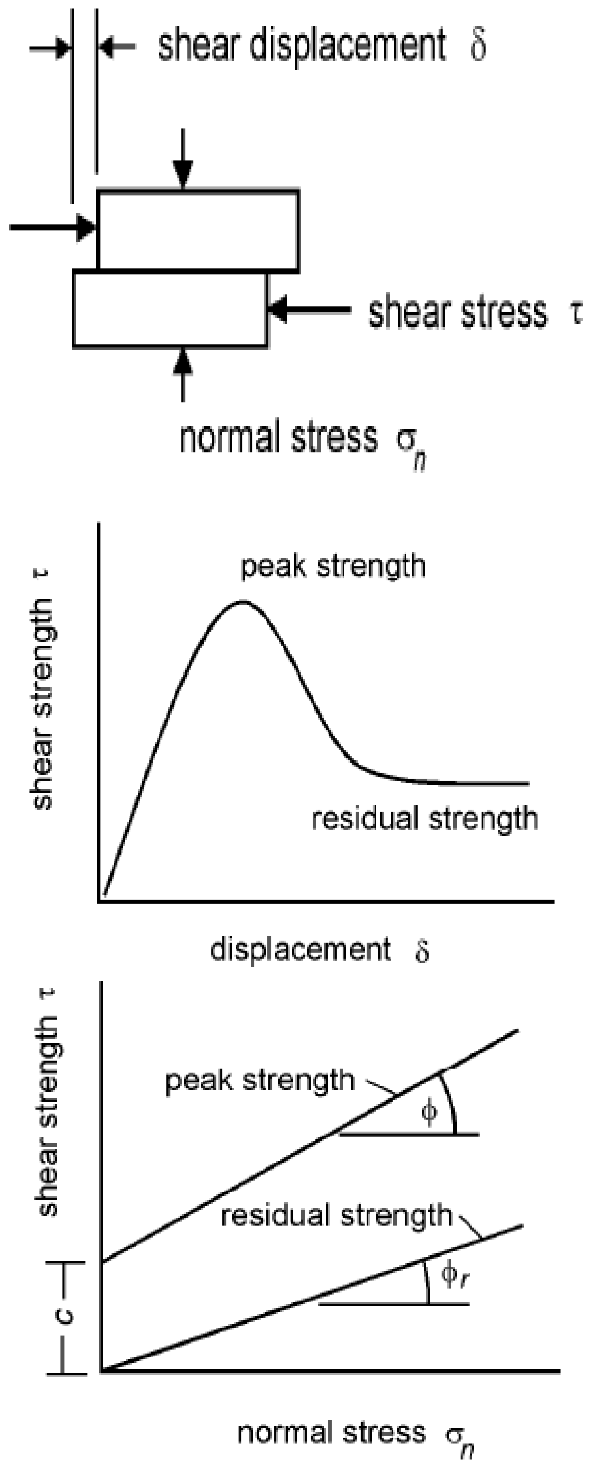


Figure 3.22 Shear testing of discontinuities (Evert Hoek, 2000).

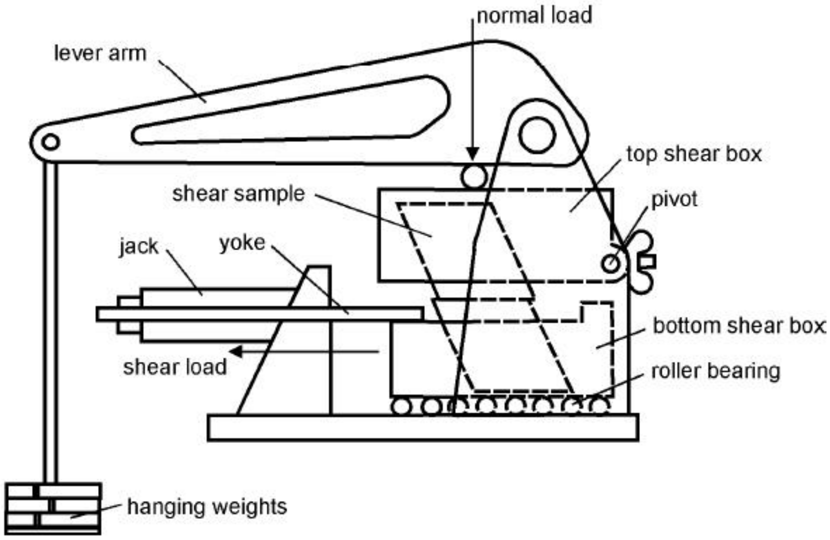


Figure 3.23 Diagrammatic section through shear machine (Evert Hoek, 2000).

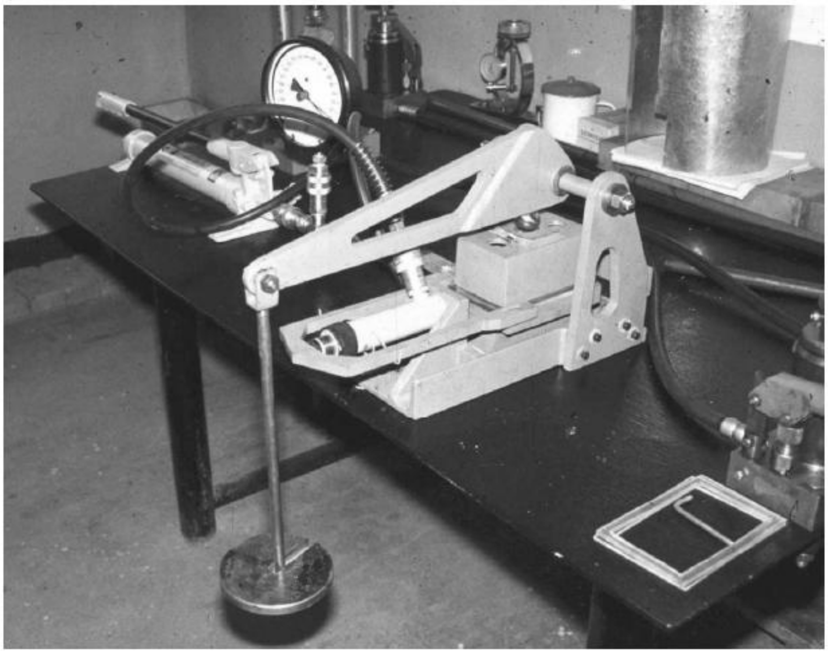


Figure 3.24 Shear machine of the type used for measurement of the shear strength of sheet joints in Hong Kong granite (Evert Hoek, 2000).

3.4.1.6.1 The Procedure For Direct Shear Test

The details about the procedure and classification for this test found at ASTMD3080 (2004). In this project different methods used to prepare the samples for direct shear test such as:

- Coring sample apparatus, see Figure 3.25.
- Pressure machine, see Figure 3.26.
- Manually, see Figure 3.27.

Even all that methods, the sample was disturb partially every time, and have not suitable dimension which correspondent with test require (the cubic shape sample correspondent with rectangular or circular ring of the direct Shear box apparatus). Thus, because the cohesion for the sample had not enough strong to keep the sample stand until shaped into suitable shape.

Figure 3.25 The dry sample was disturbed partially during use coring apparatus to prepare it for the direct shear test.

Figure 3.26 Photo shows on effort to prepare cubic sample for direct shear test by pressure with hydraulic machine (the sample was broken).

Figure 3.27 Preparation of sample by cutting for direct shear test.

Consequently, in order to overcome the previews problem the samples was prepared in-site by using fabricated tools and the direct shear cutting ring with dimension 100x100mm. Figure 3.28 shows the fabricated tools were used for extract samples such as:

- Two type of geological hammer (wide and sharpened) which used to puncture a piece of metal rod.
- A piece of wood or metal rod (prefer wood for more safety to save cutting ring) to transfer the energy of hammer to the cutting ring.
- Rectangular wood (in this case it also used two T shape of wood) of wood to transfer the energy of hammer to go enough deep when the deep was more than 20 mm.
- Sheet metal to extract the sample from site position under the cutting ring.
- Two long sharp spoon

The steps of preparation the samples in-situ were shown in Figures 3.29, 3.30, 3.31, 3.32, 3.33, 3.34, 3.35, 3.36 and 3.37.

Step 1: Clear the chosen area from the upper 5mm (to reduce the weathering effective) and make it as flat surface .

Step 2: Put the cutting ring then the metal rod over it and starting puncture by hammer.

Step 3: When reach to enough deep (equal the cutting ring high) clean the surrounded area to prepare to extent the deep.

Step 4: Use the fabricated wood with the rod and hammer to penetrate deep.

Step 5: Measure the sample thickness, if enough, then extract the sample by using the metal sheet.

Step 6: Collect the sample carefully and put it on sealed plastic to keep the natural moisture content.

Step 7: If the sample need to reduce the thickness you reshape it directly when it still wet.

(a 1)

(a 2)

(b)

(c)

(d)

(e)

(f)

Figure 3.28 The tools which used to preparation the sample in-situ

(a1, a2) Cutting ring with fabricate wood (b) Metal rod

(c) Two pieces of T shaped wood. (d) Sheet metal

(e) Geological hammer (f) Two long sharp spoon.

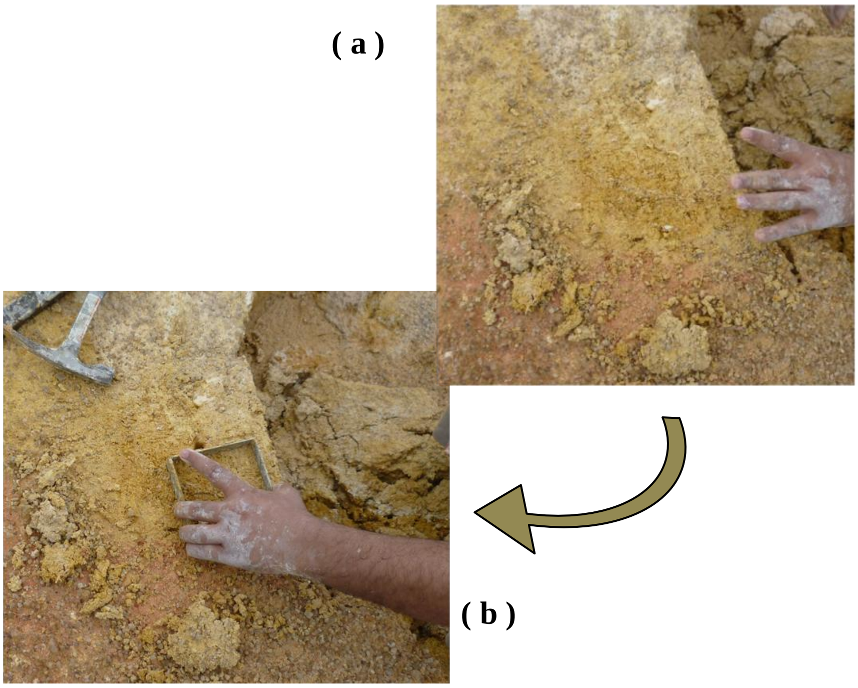


Figure 3.29 Step 1: Clearing the chosen area from the upper 5mm and make it as flat surface (procedure to preparation samples for the direct shear test)

(a) making flat surface (b) sit the cutting ring.

Figure 3.30 Step 2: Put the cutting ring then the metal rod over it and starting puncture by hammer (procedure to preparation samples for the direct shear test).

Figure 3.31 Step 3.a: Sample inside the ring reach to enough deep equal the cutting ring high (procedure to preparation samples for the direct shear test).

Figure 3.32 Step 3.b: Material surrounding cutting were cleared to retrieve samples (procedure to preparation samples for the direct shear test).

Figure 3.33 Step 4: Use the fabricated wood to penetrate more (procedure to preparation samples for the direct shear test).

Figure 3.34 Step 5.a: Measuring the sample thickness (procedure to preparation samples for the direct shear test).

Figure 3.35 Step 5.b: Extract the sample by using the metal sheet, when the thickness of sample enough (procedure to preparation samples for the direct shear test).

Figure 3.36 Step 6: Collect the sample carefully and put it on sealed plastic to keep the natural moisture content (procedure to preparation samples for the direct shear test).

Figure 3.37 Step 7: Reshape the sample when it still wet in order reduce the thickness (procedure to preparation samples for the direct shear test).

3.4.2 At Wet Condition

This condition was have the moisture content for the in-situ condition. However, this procedure for test moisture content a was the same as at dry condition. Otherwise for direct shear test the same stage which carried out in section 3.4.1.6.1 with performance the tests when the samples have natural moisture content. The procedure at which used at ASTM D3080 (2004).

3.4.3 At Saturated Condition

The procedure for test moisture content and direct shear test a was the same as at dry condition.

CHAPTER 4

RESULTS AND ANALYSIS

4.1. Introduction

In this case study, a lot of works done and many problems challenged the implementation of the project whatever at field or at laboratory. Otherwise, most of the problems are solved, by using the available facilities. Some suggestion was presented at next chapter.

Some of tests did not take a long time and already canalled because from the preparing steps for the test it shown clear and strong evidence about useless continuous of the tests, one of those tests is the slake durability tests, the same thing as for point load test and Schmidt (rebound) hammer test. The weakness that shown by samples of older alluvium give adequate answer for natural of this material.

However the grain size analysis shows the present of fine material with rate of 38% compare to medium and coarse material. Moreover the natural moisture content was around 18% at the time when the samples was collected from the site

The longest test which double time to finishing it was the direct shear test whatever, during prepared the samples or during carried out the tests. However the results show the effect of increase the moisture on cohesion, friction resistance force and shear strength. In another hand, increase of water content to high value give opposite effect on the cohesion.

4.2. Site Description For Mass Properties

The study has been conducted at Desa Tebrau, south of Johor . The older alluvium covers about $300m^2$ of the site. The older alluvium was surrounded by weathered granite deposits and colour of older alluvium is tend to be yellowish with some dark brown-red lines. Moreover some of relict structures with about 1 to 10 m long with main dip direction about 35 south west (145^o), and approximately vertical dip and between 13^o to 80^o dip angle. Otherwise, on some of relict structure there are iron deposits which leaching through this structure to make iron deposits occurrence between the joint of older alluvium to make dark brown-red lines.

However no occurrence for fossils or trace for remain old organic. At weathered granite the red colure was occur clearly which it can give evidence for iron deposits. In another side the apparent grain size of particles of older alluvium deposits does not exceed more than 7.5mm. The angular shape of granular soil particles give evidence that the location of deposits of older alluvium are closed from the source of this deposits, which was represented by Quartz veins (ASTM 2488, 2009; David F., 2007). At the location, the mean maximum daily temperature is 38°C and the mean minimum is 30°C. Annual rainfall is approximately 1260 mm, see Figures 4.1; 4.2; 4.3; 4.4.

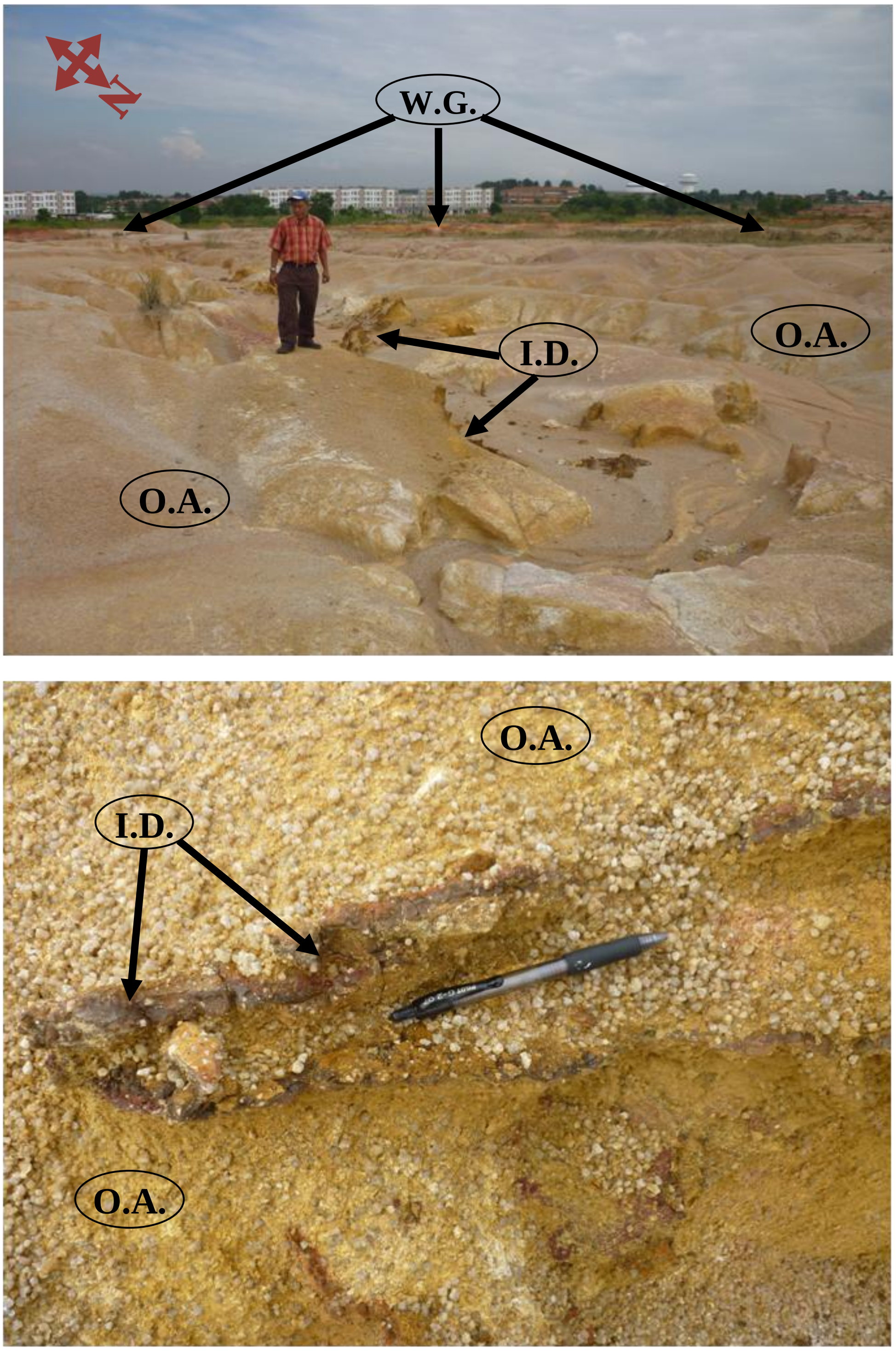


Figure 4.1 Site description:

O.A. = Older alluvium W.G. = Weathered granite.

I. D. = Iron deposits leaching into the relict structure and file it.

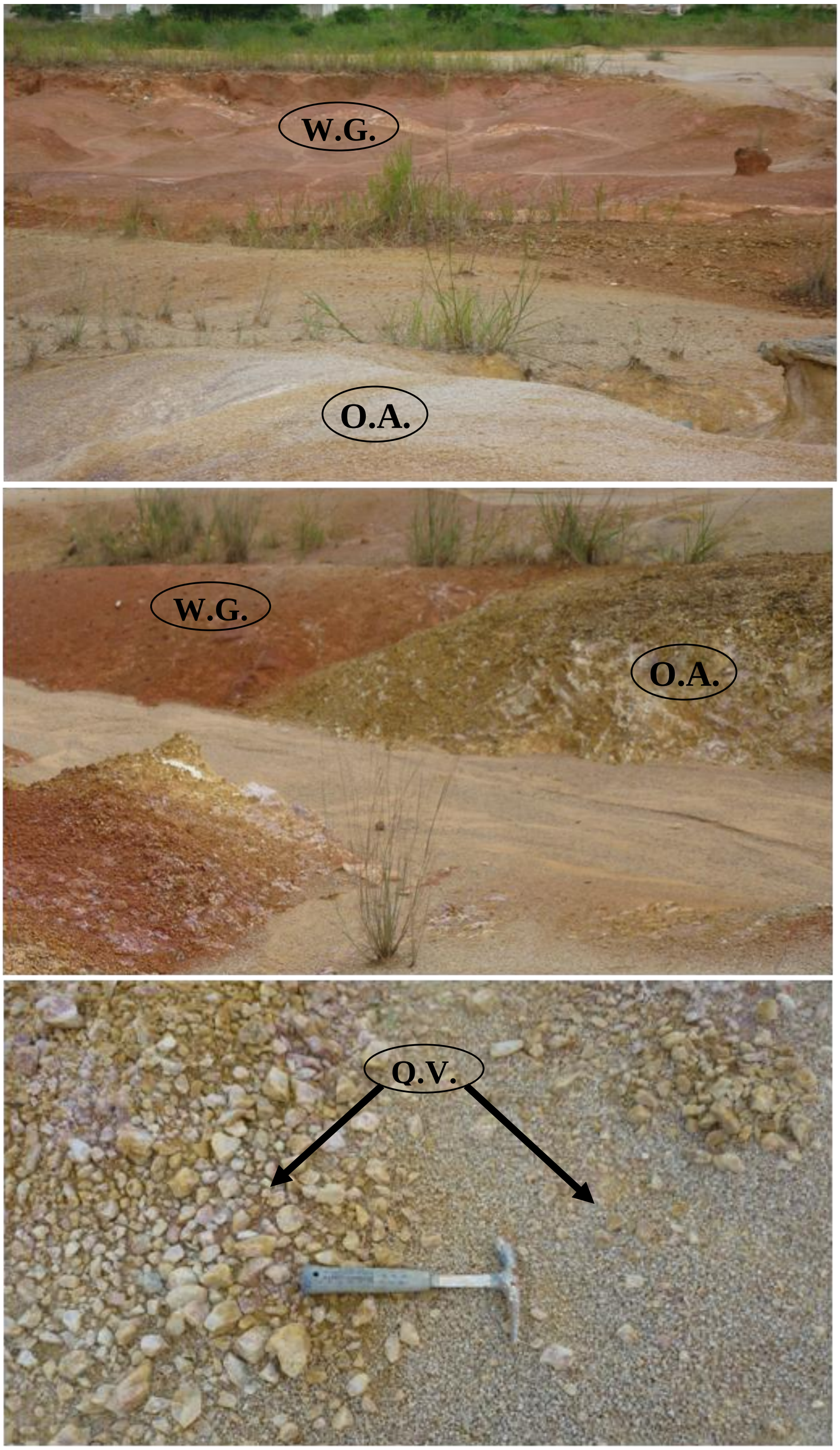


Figure 4.2 Site Description: O.A.= Older alluvium with yellowish colour. W.G.= Weathered granite with red colour.
Q.V= Quartz veins deposits.

Figure 4.3 Site description: O.A. = Older alluvium.
R.S. = Relict structure without iron leaching.

Figure 4.4 Site description: Apparent grain size of particles of older alluvium deposits < 8mm.

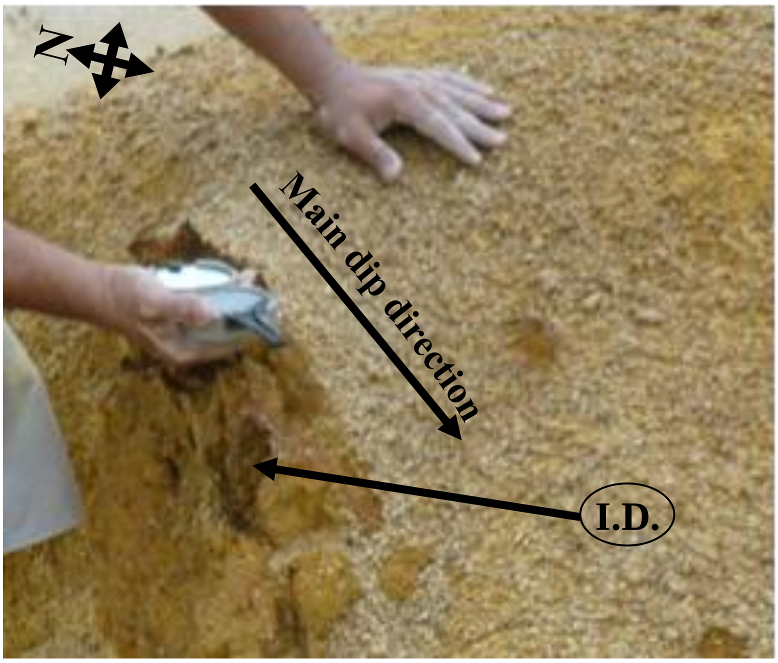


Figure 4.5 Site Description: Main dip direction

I. D. = Iron deposits leaching into the relict structure and file it.

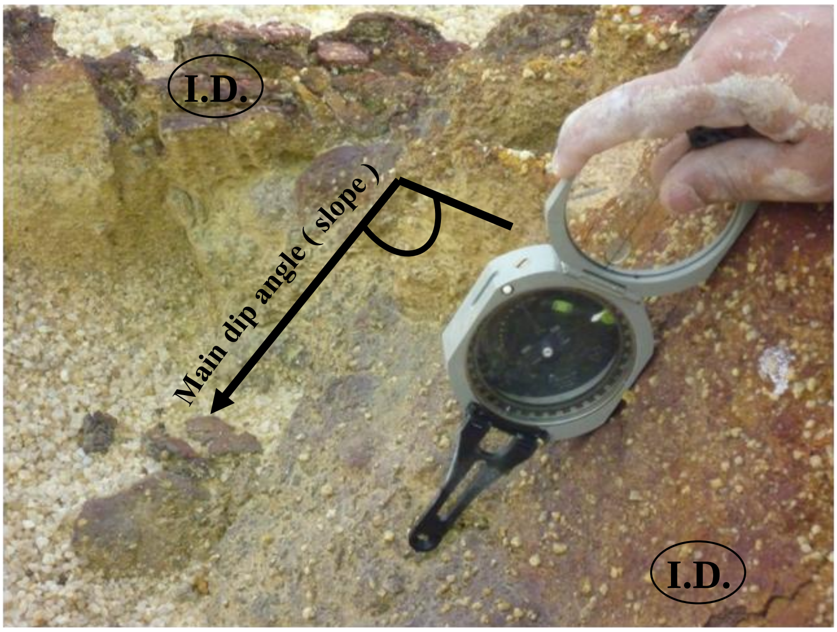


Figure 4.6 Site Description, main dip angle (slope)

I. D. = Iron deposits leaching into the relict structure and file it.

4.3. Schmidt (Rebound) Hammer (RH)

The results from the in-situ Schmidt hammer test for was be equal zero, whatever in-situ or on the laboratory, even the samples on the laboratory were dried for week on the oven at a temperature of 110±5^{o} C (Kalinski, 2006).

No reading was recorded when test the wet sample. Otherwise in the laboratory the dried sample was destroy when tried to carried out the Test, see Figure 4.7.

Figure 4.7 The sample was destroyed during the Schmidt (rebound) hammer test.

4.4. Point Load Test (PLT)

From the results at Schmidt rebound hammer test which give zero record for the sample at wet condition, PLT was carried out only on the dry condition. However when the regular sample (cylindrical i.e. Core sample) was prepared, the sample destroyed, because it cannot stand during made the coring, see Figure 4.8. Consequence the PLT was applied on preliminary irregular shape (rectangular i.e. cubic sample, the dimension for the samples did not taken) to avoid waste the time if the sample did not stand the same as at Schmidt hammer test (Ibrahim and Sefer, 2008; Mohd For, 2008).

Figure 4.8 The sample was destroyed during coring to prepared regular sample (cylindrical i.e. core sample) for point-load test PLT**.**

However, the result which come from carried out the point-load test PLT on the preliminary irregular (rectangular i.e. cubic sample) shape was similar from the RH, that the sample did not sustain (the sample was cracked than destroyed before the apparatus recorded anything) and no record was given form the apparatus, see Figure 4.9.

Because the point-load apparatus was manually and not so sensitive, so the no result recorded at apparatus. Consequence the result for carried out point-load index considered as zero (Edy Tonnizam et al., 2008).

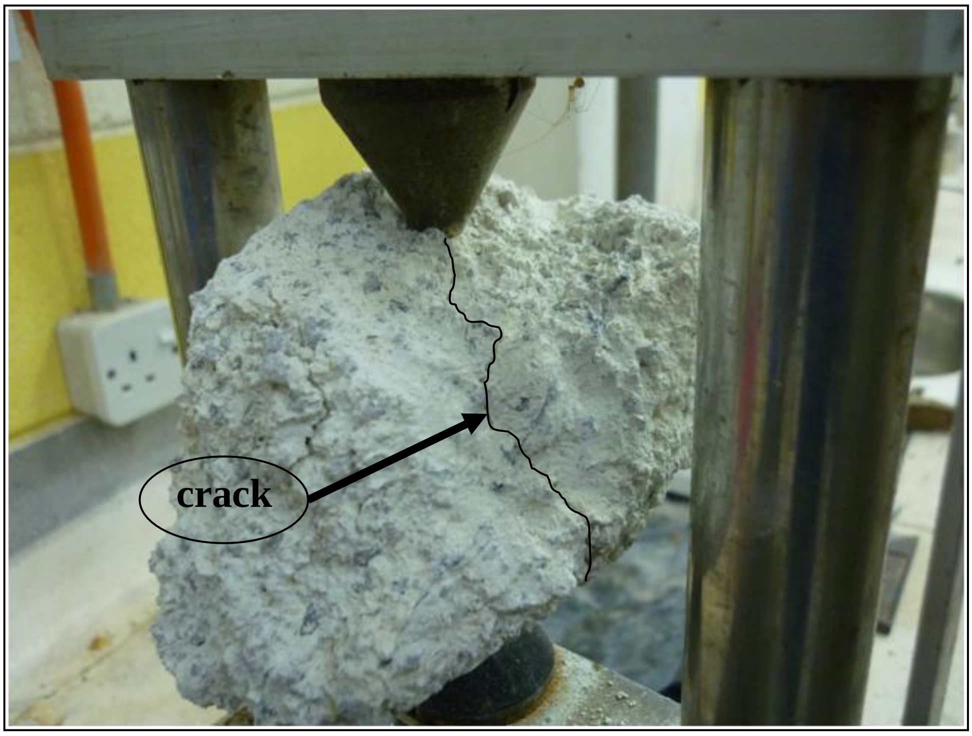


Figure 4.9 The sample was cracked than destroyed before the point-load tester recorded anything.

4.5. Slake Durability Test (SDT)

By referring to section 3.4.1.3, the slake durability was cancelled, because the wet and dry samples were fully destroyed, after submerged in water for ten minutes even without applied any movement for the samples, see Figure 4.10. Consequence the result from SDT was considered as zero (Edy Tonnizam et al., 2008).

Figure 4.10 The samples fully destroyed after submerged in water for ten minutes.

4.6. Analysis Of Results Of: Schmidt (Rebound) Hammer (RH), Point-Load Test (PLT) And Slake Durability Test (SDT)

The results from Schmidt (rebound) hammer RH, point-load test PLT and slake durability test SDT were shown in Table 4.1. It is appear no data can be recorded during the tests. The samples were destroyed easily, because it had not enough strength to stand or resist the applied force.

Table 4.1. Conclusion of the results of: Schmidt (rebound) hammer test, point-load test and slake durability test

Test Name	Sample Shape	Sample Condition	Considered results	Sample reaction
RH	Irregular	Wet and Dry	Zero	Destroyed before the apparatus recorded anything
PLT	Irregular	Dry	Zero	Cracked and destroyed before the apparatus recorded anything
SDT	Irregular	Wet and Dry	Zero	Destroyed after submerged 10 minutes

RH = Schmidt (rebound) hammer test.
PLT = Point-load test.
SDT = Slake durability test

Schmidt hammer test carried out at irregular samples in dry condition. The same as for point-load test, that preparation of regular samples was very difficult and non reliable, because the samples were destroyed during preparation it at laboratory, so the irregular samples were used.

There is no data recorded on the Portable point load tester mainly due to the insensitivity of the apparatus. The similar result also noted in the Schmidt hammer test. For that results, the samples of older alluvium can be described as weaker than intact weak rock (not exceed grade III) (Edy Tonnizam et al., 2008; Daniel et al.,2007; Adnan, 2008; Zhang, 2006; Margaret and Abdul Shakoor, 1996; Ibrahim and Sefer, 2008).

During preparation for the slake durability test, it was found that the samples were fully destroyed after submerged in water for less than 10 minutes. This results indicates the harmful effects with the increase of moisture content (Edy Tonnizam et al., 2008; Vásárhelyi, 2003; Vásárhelyi and Ván, 2006; Abdul Shakoor and Barefild, 2009).

4.7. Wet Sieve Analysis

In this project, it focus only on the general distribution for particles grain size, while the analysis for fine material was ignored because it over project scope.

4.7.1 Wet Sieve Analysis: Results

The results for wet sieve were showing in Table 4.2 and Figure 4.11.

Table 4.2 : Results of wet sieve analysis.

Opening sieve size (mm)	Mass retained on each sieve (g)	Cumulative mass (g)	Percent finer*** (%)
10	0	0	100
6.3	18.36	18.36	98.23
5	66.61	84.97	91.79
3.35	203.67	288.64	72.11
2	109.5	398.14	61.53
1.18	61.42	459.56	55.60
0.600	69.09	528.65	48.92
0.425	28.07	556.72	46.21
0.300	25.67	582.39	43.73
0.212	20.02	602.41	41.80
0.150	16.79	619.2	40.17
0.063	20.92	640.12	38.15
Pan	394.88	1035	0.00
	ΣM =1035		

$$\text{*** Percent finer} = \frac{\Sigma M - \text{column } 3}{\Sigma M} \times 100 = \frac{1035 - \text{column } 3}{1035} \times 100$$

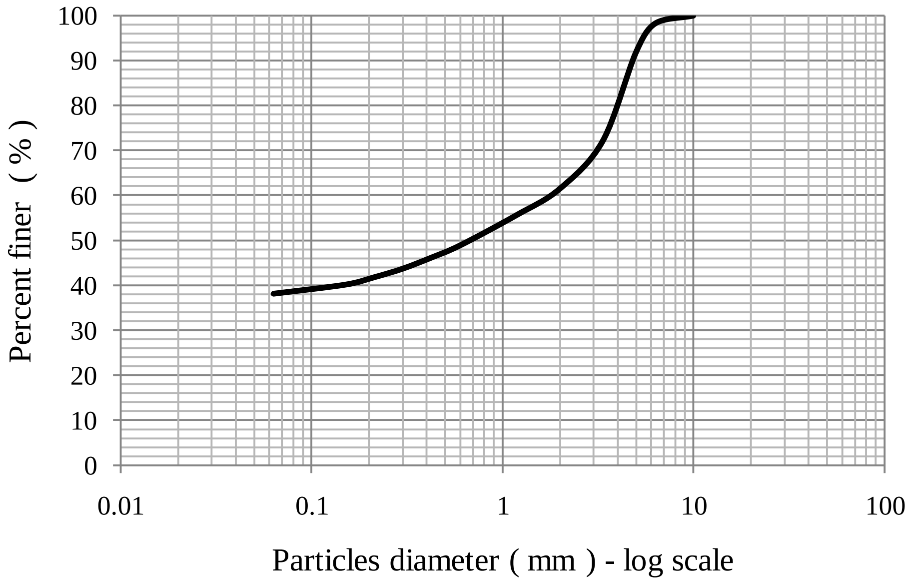


Figure 4.11 Results of wet sieve analysis.

4.7.2 Wet Sieve Analysis: Calculation

The formula that used at Table 4.2 was:

$$\text{Percent finer} = \frac{\Sigma M \ - \ \text{column 3}}{\Sigma M} \times 100 = \frac{1035 - \text{column 3}}{1035} \times 100$$

From (Table 4.2. and Figure 4.9.) it can get the following data :

$$\left.\begin{array}{l} D_{10} < 0.063 \\ D_{30} < 0.063 \end{array}\right\} \text{Assumption value as 0.01mm because there are no analysis for fine material.}$$

D_{60} = 1.84 mm

$$\text{Uniformity coefficient} = C_u = \frac{D_{60}}{D_{10}} = \frac{1.84}{0.01} = 184 \quad (C_u > 5 \quad \text{well -graded soil})$$

$$\text{Coefficient of gradation (curvature) } C_c = \frac{(D_{30})^2}{D_{60} \ x \ D_{10}} = \frac{(0.01)^2}{1.84 \text{ x } 0.01} = 0.0054$$

(C_c < 0.1 indicate a possible gap-graded soil)

4.7.3 Wet Sieve Analysis: Analysis

The older alluvium O.A. shows equilibrium between distribution of clay/silt and gravel with lowers presence of sand and it can classified as Clayey-Gravel (CL or CH) or Silty-Gravel (ML or MH) according to ASTM D2487, 2010 (David, 2007).

From Table 4.2, it can conclude that the percentage of finer of fine material (clay and silt <63 µm) are ≈ 395 g from the total mass of 1035g, in another words, it represented ≈ 38% of the component of older alluvium. The coarse material recorded about 640.12g from total mass 1035g with percent finer ≈ 61.9%. Moreover, the sand (from > 63 µm to < 2 mm) recorded about 241.98g from total mass 1035g with percent finer ≈ 23.4%. However, the gravel was recorded about 398.14g from total mass 1035g with percent finer ≈ 38.5%.

The high percentage of gravel (about 38.5% of whole O.A.) gives a good explanation for the high portion of friction angle. However the presence the fine material (about 38% of whole O.A.) give ability for soil to stick together and provide strong bond between the particles (cohesion c). The fine material (i. e. clay) considering as good source of cohesion, but its weakest binding material in rock (Mathieu Lamotte et al., 1997; Engin et. al., 1998).

Moreover, according to uniformity coefficient (C_u = 184 > 5), the soil can be described as well-grad soil. Otherwise, according to coefficient of gradation, curvature (C_c = 0.0054 < 0.1) it was indicated a gap-graded descriptive for soil.

4.8. Moisture Content Test

The number of sample which used for this test were six samples.

4.8.1 Moisture Content Test: Results

The results from moisture content test are shown in Table 4.3.

Table 4.3 : Results of moisture content tests.

	Units	Test 1	Test 2	Test 3	Test 4	Test 5	Test 6
Depth of sample	**m**	1	1	1	1	1	1
Sample number	**N/A**	1	2	3	4	5	6
Container Number	**N/A**	MG 22	MG 36	MG 44	MG 58	MG 59	MG 62
Mass of container M_c	**g**	29.732	29.527	29.375	29.249	29.382	29.514
Mass of container + wet sample M_1	**g**	219.107	181.571	192.196	199.962	187.158	196.210
Mass of container + dry soil M_2	**g**	189.706	157.374	165.462	172.777	163.116	170.085
Mass of water M_w	**g**	29.401	24.197	26.734	27.185	24.042	26.125
Mass of dry soil M_s	**g**	159.974	127.847	136.087	143.528	133.734	140.571
Moisture content w%	**%**	18.38	18.93	19.65	18.94	17.98	18.59
The Average of Natural Moisture Content = 18.75 %							

4.8.2 Moisture Content Test: Calculation

The formulas which to determine moisture content was :

$$w\ \% = = \frac{M_1 - M_2}{M_2 - M_c} \text{ x } 100 = \frac{M_w}{M_s} \text{ x } 100$$

Where:

w % = Water content %

M_1 = Mass of container and wet specimen in g

M_2 = Mass of container and dried specimen in g

M_c = Mass of container in g

M_w = Mass of water in g

M_s = Mass of solid particles (dry soil) in g

4.8.3 Moisture Content Test: Analysis

In general, the natural moisture content was within range of 17.98% to 19.65% with average of 18.75%. It should notice that the samples was token after one day rain (low to medium rain density), and the same moisture content was approved as moisture content for wet condition. However even the natural moisture content is can give some explanation for the change of behaviour of strength parameter of older alluvium at different depths but unfortunately all the samples were collected from the same depth, so it cannot compare between the values of moisture content at different depths. However, carried out of the direct shear test at different value of moisture content can give enough explanation for that behaviour.

4.9. Direct Shear Test

4.9.1 Direct Shear Test: Results At Dry Condition

The results of direct shear test at dry condition are shown in Table 4.4 and Figure 4.12.

Table 4.4 : Conclusion of results of direct shear test at dry condition.

Sample number	A	B	C	D
Applied normal stress (kPa)	21.1	30.9	50.5	11.3
Peak stress (kPa)	74.4	92.9	146.6	49.6

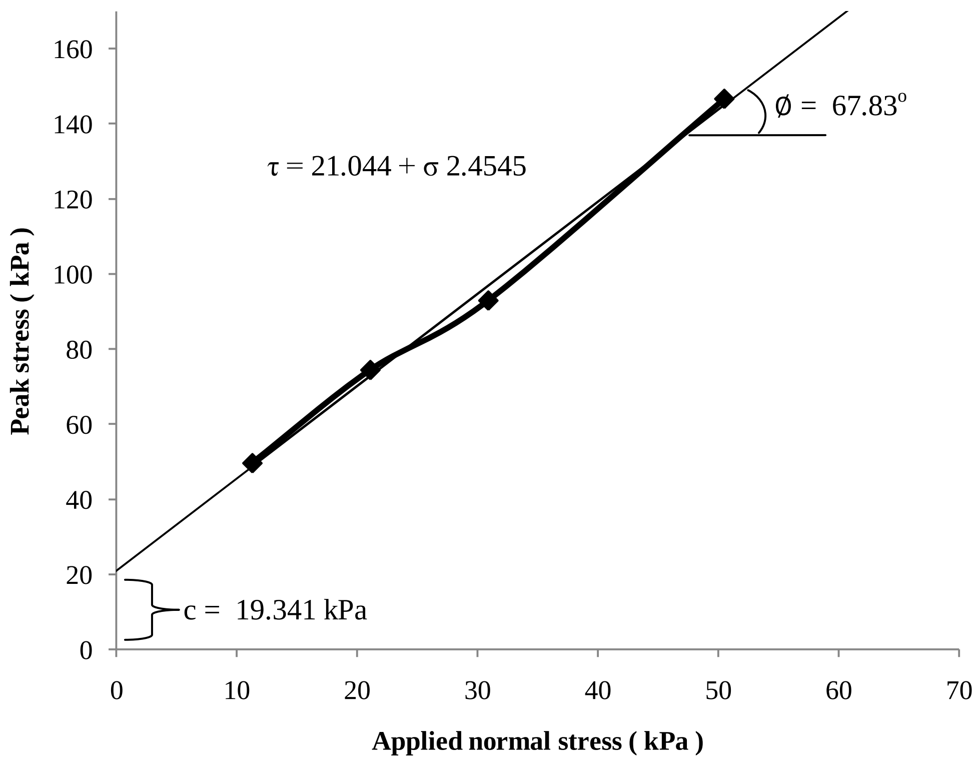


Figure 4.12 Conclusion of results of direct shear test at dry condition.

4.9.2 Direct Shear Test: Results At Wet Condition

The results of direct shear test at wet condition are shown in Table 4.5 and Figure 4.13.

Table 4.5 : Conclusion of results of direct shear test at wet condition .

Sample number	A	B	C
Applied normal stress (kPa)	11.3	21.1	30.9
Peak stress (kPa)	38.2	57.6	66.4

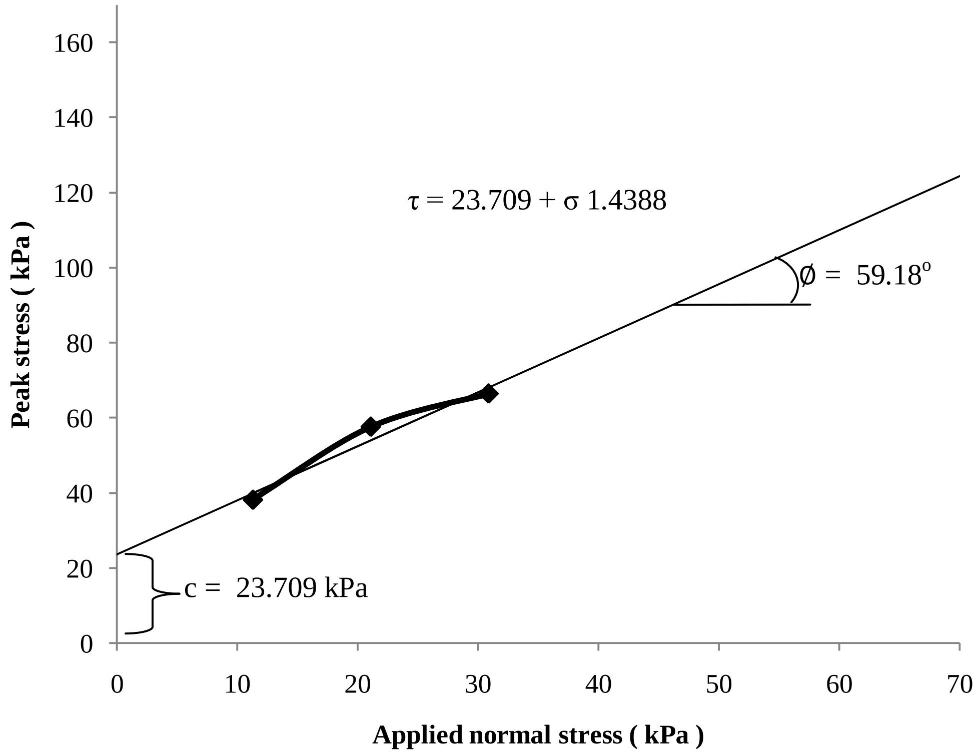


Figure 4.13 Conclusion of results of direct shear test at wet condition.

4.9.3 Direct Shear Test: Results At Saturated Condition

The results of direct shear test at saturated condition are shown in Table 4.6 and Figure 4.14.

Table 4.6 : Conclusion of results of direct shear test at saturated condition.

Sample number	A	B	C
Applied normal stress (kPa)	11.3	21.1	30.9
Peak stress (kPa)	14.2	18.3	22.3

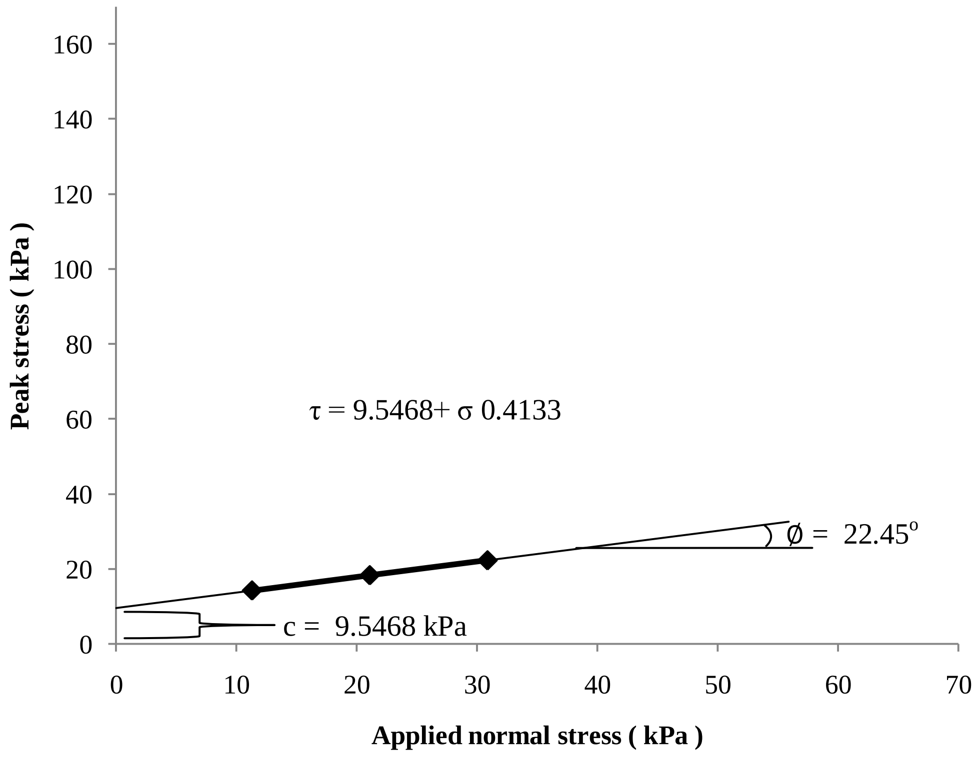


Figure 4.14 Conclusion of results of direct shear test at saturated condition.

4.9.4 Direct Shear Test: Calculation

Many formulas were used, for calculate the displacement rate:

$t_{50} = \frac{t_{90}}{4.28}$

$t_f = 50\ t_{50}$

$d_r = \frac{d_f}{tf}$

where :

t_{50} = Time required for the specimen to achieve 50 percent consolidation under the specified normal stress (or increments thereof), min.
t_{90} = Time required for the specimen to achieve 90 percent consolidation under the specified normal stress (or increment thereof), min.
4.28 = Constant, relates displacement and time factors at 50 and 90 percent consolidation.
t_f = Total estimated elapsed time to failure, min.
d_f = Estimated horizontal displacement at failure mm, in this study it assumed as = 5 mm.
d_r = Displacement rate mm/min.

For calculate the normal and shear stress :

$\tau = F / A$

$n = N / A$

where :

τ = Nominal shear stress, kPa.
F = Shear force, N.
A = Initial area of the specimen, mm^2.
n = Normal stress, kPa.
N = Normal vertical force acting on the specimen, N.

However, the basic equation for shear stress was:

$\tau = c + \sigma \tan \emptyset$

where :

c = Cohesion, kPa.
σ = Total stress, kPa.
$\emptyset$ = Friction angle, degree.

4.9.5 Conclusion And Analysis Of Results Of Direct Shear Test

A total of 21 tests were performed on the samples. However only 10 tests were succeeded and the other 11 tests were give unsatisfactory results. Table 4.7 shows the results of 10 samples (4 dry, 3 wet and 3 saturated samples). The total number of samples that collected and prepared were 30 samples, and it collected after rainfall in order to reduce the potential of extracted the samples because reduce the strength of older alluvium after rainfall (Joseph et. al., 2009; Romana and Vásárhelyi, 2007). Table 4.8 and Figure 4.15 show the applied load which used, i.e. 11.3kPa, 21.1kPa and 30.9kPa for dry, wet and saturated condition, while the load of 50.5 kPa was used only for dry condition. The moisture content measured was 0% for dry condition, between 18.1% to 21.7% for wet condition and between 25.3% to 26.8% for saturated conditions.

Table 4.7 : Comparison of peak stress, applied normal stress, W.C., condition and type of soil of older alluvium samples.

Sample No.	condition	W.C.* (%)	Degree of saturation (%)	Type of soil**	Normal stress (kPa)	Peak stress (kPa)
1	Dry	0	0	Dense	11.3	49.6
2	Dry	0	0	Dense	21.1	74.1
3	Dry	0	0	Dense	30.9	92.9
4	Dry	0	0	Dense	50.5	146.6
5	Wet	18.1	80.26	Dense	11.3	38.2
6	Wet	20	78.13	Dense	21.1	57.6
7	Wet	20.7	90.83	Dense	30.9	66.4
8	Saturated	25.3	86.14	Dense	11.3	14.2
9	Saturated	26.8	92.61	Dense	21.1	18.3
10	Saturated	26.5	91.87	Dense	30.9	22.3

* W.C. = Moisture content (Water content)
** Type of soil refer to the type of curve between horizontal and vertical displacement.

Table 4.8 : Conclusion of the results of direct shear test for different condition (dry, wet and saturated).

Sample condition	**Shear stress equation**	**Cohesion c (kPa)**	**Friction Angle Ø°**
Dry	$\tau = 21.044 + \sigma\ 2.4545$	21.044	67.83
Wet	$\tau = 23.709 + \sigma\ 1.4388$	23.709	55.19
Saturated	$\tau = 9.5468 + \sigma\ 0.4133$	9.5468	22.45

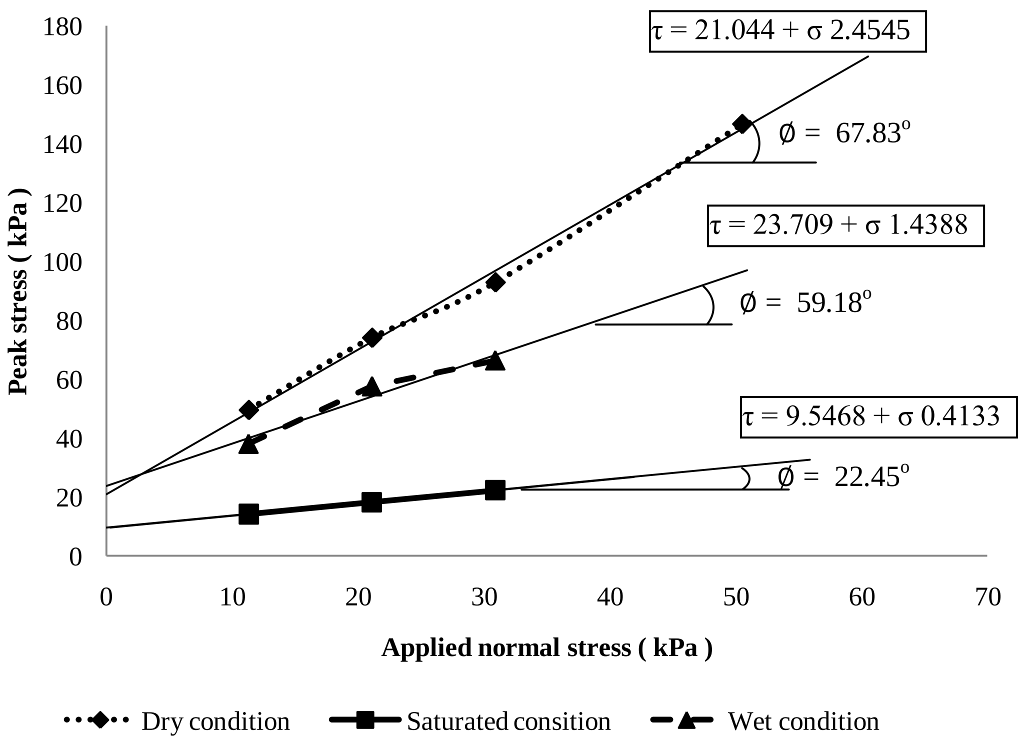


Figure 4.15 Conclusion of results of direct shear test at dry, wet and saturated condition.

The results show that the shear strength τ decrease with the increase of moisture content and degree of saturation. The shear strength as represented by the peak stress was within range 49.6kPa to 92.9kPa, which indicate that the older alluvium can be classified as stiff soil (considering the applied normal stress σ = 21.1kPa which represented 1 m beneath of older alluvium deposits surface) (Budhu, 2007). At wet condition, the shear strength range was from 38.2kPa to 66.4kPa, which can also be classified as stiff soil. However, at saturated condition, the result range were from 14.2kPa to 22.3kPa which it can be classified as soft soil (Das, 2006; 2008). Moreover, the relationship of horizontal displacement and vertical displacement during the shearing the samples show that older alluvium deposits act as dense soil at dry and wet condition, while act as loose material at saturated conditions, see appendix A, B and C (Whitlow, 2001).

Table 4.8 and Figure 4.15 show the friction angle Ø at dry, wet and saturated condition were 67.83^o, 55.19^o and 22.45^o respectively. In addition, the results show the cohesion c value at dry, wet and saturated condition were 21.044kPa, 23.709kPa and 9.5468kPa respectively. The explanation for relative high value of friction angle and cohesion at dry and wet conditions on older alluvium can be related to the mixture of particles especially the percent finer for gravel and clay (Namdar, 2010).

Table 4.9 shows the reduction of shear strength. At wet condition (moisture content w= 20%), the reduction on shear strength noted as 22.3% in comparison to value at dry condition. However, a slight increase of moisture content at saturated condition (moisture content w =26.8%) produced a reduction of shear strength up to 75.3%. On the other hand, the value of friction angle gave reduction up to 18.6% at wet condition, and 66.9% at saturated condition in comparison to value at dry condition. However the effect of change moisture content at cohesion was variable, the cohesion increased progressive with increase the moisture content to be greater by +12.7%, until reach to specific value of increasing the moisture content w= 25.3%, then the magnitude of cohesion start to decrease, so the reduction became 54.6% in comparison to the dry condition.

Table 4.9 : Comparison reduction of shear strength, friction angle and cohesion at dry, wet and saturated condition of older alluvium samples (considering the applied normal equal 1m beneath the surface).

Sample condition	Moisture content (%)	Degree of saturation (%)	Normal stress σ (kPa)	Shear strength τ (kPa)	Reduction at τ (%)	Friction angle ∅ °	Reduction at ∅ (%)	Cohesion c (kPa)	Reduction at c (%)
D.	0	0	21.1	74.1		67.83		21.044	
W.	20	78.13	21.1	57.6	22.3%	55.19	18.6%	23.709	+12.7%
S.	26.8	92.61	21.1	18.3	75.3%	22.45	66.9%	9.5468	54.6%

D. = Dry condition W. = Wet condition S. = Saturated condition
\+ Means no reduction but there was increase in the value

The explanation for that behaviour is related to percent the water between the soil particles. At dry condition the friction angle was 67.83^{o} which represented the friction resistance forces between gravel and coarse sand. Otherwise the fine material (clay and silt < 0.63 μ m) which have percent finer about 38% (this percent is high) created high portion of cohesion. Otherwise, at wet condition, appearance of water increased the force of cohesion and decrease friction force, in another word the exit water between the medium and big particles act as lubrication so the sliding movement between the particles will be easier because decrease the friction resistance between it. Furthermore, the existence of water increase the cohesion between the fine particles which represented around 38% of particles percent size and give the soil higher consistency, so the bond between the fine particles add extra cohesion force to the soil. However, the extra increase of water (such as at saturated condition) will make friction resistance force between the gravel and coarse sand tend to be the lowest value and reduced to 66.9%, also the bond between the fine particles come down, because the high portion of water tend to extend the distance between the fine material, cause decreased the attraction force between fine materials, so the cohesion reduced, consequently the shear strength will reduced rapidly to become 75.3% in comparison to dry condition.

Figure 4.16 shows the shear strength at normal stress (σ = 11.3kPa, 21.1kPa and 30.9kPa) reduced gently with increased moisture content, until the moisture content cross over the natural moisture content then the value of shear strength will fall down rapidly to reach to the lowest value. This can give signature to zone of moisture content which have high harmful effect on the older alluvium soil. This harmful zone of moisture content could be start when the value moisture content become higher than 22%.

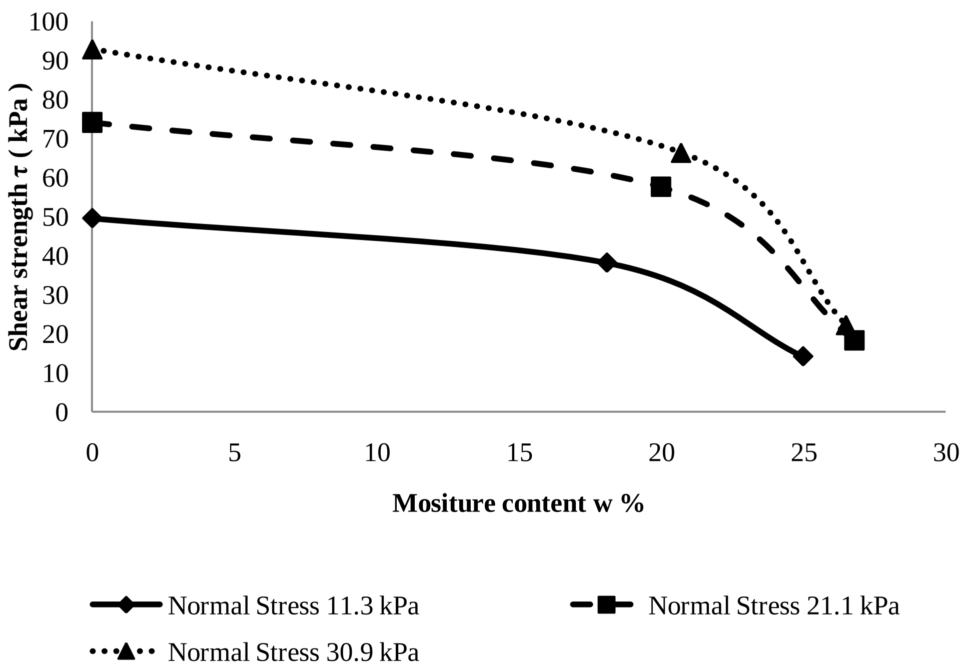


Figure 4.16 Comparison reduction of shear strength with moisture content at different applied normal stress 11.3, 21.1 and 30.9 kPa).

Figure 4.17 shows the effect of change of moisture content on shear strength parameters (friction angle Ø and cohesion c), for friction angle the same as shear strength it reduced gently with increase the moisture content until the moisture content cross over the zone of natural moisture content w > 21%, then the value of friction angle go down rapidly. In another side, the cohesion increase slowly with increase of moisture content then when the moisture content value cross over > 21% the cohesion start to go down and reduced with increase the moisture content.

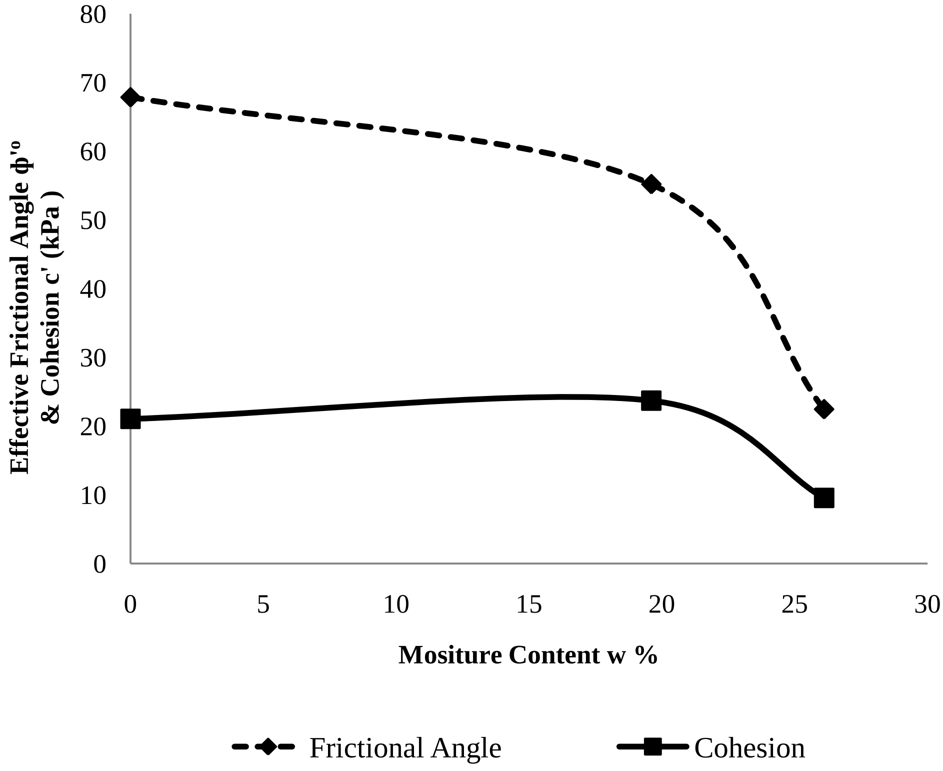


Figure 4.17 Comparison change of shear strength parameters (friction angle Ø and cohesion c) with moisture content.

4.10. General Review For All Tests

Table 4.10 shows the suitability, sample shape and sample condition for the geotechnical tests which carried out on the older alluvium. It is clear that the geotechnical tests which applied for rock such as Schmidt hammer test, point-load test and slake durability test is not suitable for the older alluvium sample because the samples always destroyed and failure before the apparatus show any reading. That come from the weakness of older alluvium (which considering at GradeV and GradeVI) compare with the weak rocks (Adnan, 2008). However it should not carried the rock test on the older alluvium by using portable apparatus but with more sensitive apparatus such as Universal Test Machine, UTM (Edy Tonnizam et al., 2008).

On other side, the direct shear can be carried out on regular sample of older alluvium, but require prepare the sample in-situ, because the sample cannot stay undisturbed sample during conventional methods of preparation the sample. It can make remould sample, but it cannot be well represented for the actual condition in-situ. However, the special preparation for the sample give undisturbed sample which represented the actual condition in-situ.

Moreover the moisture content test and wet sieving can be carried directly on the conventional methods.

Table 4.10 : Suitability, sample shape and sample condition for the geotechnical tests.

Test Name	Sample Shape	Sample Condition	Suitability
Schmidt (rebound) hammer	Irregular and Regular	Wet	0
Schmidt (rebound) hammer	Irregular and Regular	Dry	0
Point-load test	Irregular and Regular	Wet	0**
Point-load test	Irregular and Regular	Dry	0**
Slake Durability	Irregular	Wet	0
Slake Durability	Irregular	Dry	0
Moisture Content	Irregular and Regular	Wet and Dry	2
Wet Sieve Analysis	-	Wet and Dry	2
Direct Shear	Regular	Dry	1
Direct Shear	Regular	Wet	1
Direct Shear	Regular	Saturated	1

** The apparatus which used was manually and it had low sensitive at reading.
0 = Unsuitable. 1 = Suitable (require preparing for the sample in-situ) .
2 = Suitable (Directly following the conventional standard) .

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

In general the older alluvium have different behaviour at different level of moisture content. Even the older alluvium have high strength at the dry condition but it is not acceptable to implement the rock test on it by using the low sensitive apparatus, because when the older alluvium samples subject for the rock tests, it show weakness that cannot stand during prepare samples (i.e. during preparation the core sample for point-load test). For Schmidt hammer, no results recorded at all condition (dry, wet and saturated conditions) also similar at point-load test.

From the previous data (data from rock test), it can conclude that should deal with older alluvium deposits as soil not as rock, and ignore all the rock test during estimate the engineering properties of it. Otherwise, it can at least use another apparatus which have more sensitive such as Universal Test Machine UTM to provide uniaxial compressive strength UCS.

In another side, the results from the grain size analysis show high percentage of fine material i.e. clay and silt which was around 38%, but generally it less than coarse material i.e. gravel 38.5% and sand 23.4%.

Moreover the natural moisture content was within range of 17.98% to 19.65% and with average of 18.75% .

However, the experiments show, it difficult to applied the direct shear test on samples without used specific steps and fabricated tools to preparation samples in-situ, because it required exist confined pressure during preparation the undisturbed samples. Otherwise, using disturb samples usually are not represented the actual situation at field.

The results from direct shear test show various values of shear parameter at different condition and the highest shear strength τ value recorded at dry condition, while the lowest value was at saturated condition. The same as for friction angle $\emptyset$ which give highest portion at dry condition, while the lowest portion at saturated. Otherwise, the cohesion c recorded the highest value at wet condition and lowest value at saturated condition.

The results shown the older alluvium deposits act as dense soil at dry and wet condition, while it act as loose material at saturated conditions. Moreover the description of older alluvium deposits was a stiff soil at dry and wet conditions, while was soft soil at saturated condition.

5.2 Recommendations And Suggestions

From this research, the difficulties which faced during implementation the project, it should attention to the following recommendation:

- During implementation any construction over the older alluvium, it should deal with it as soil not as rock.
- In case of required uniaxial compressive strength for design of classification, it should implement the test of by using more sensitive apparatus such as Universal Test Machine UTM.
- It should use undisturbed samples for direct shear test to give good represented for the field condition.
- Fabricated especial tools to extract the regular samples at field.
- Applied more tests on older alluvium at deeper depth to study engineering properties at layers where the pile foundation was penetration deep into soil, so the results will give actual strength parameter. Otherwise, should not depend on results from only standard penetration test SPT, which could not give clearly represented for actual behaviour of older alluvium when the moisture content change.
- Take care during tests, especially when the samples contain small amount of moisture content, because the result may will not within the safety range for portion of shear strength at saturated condition.

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APPENDIX A

Direct Shear Results For Dry Condition

A1 Sample Dry A

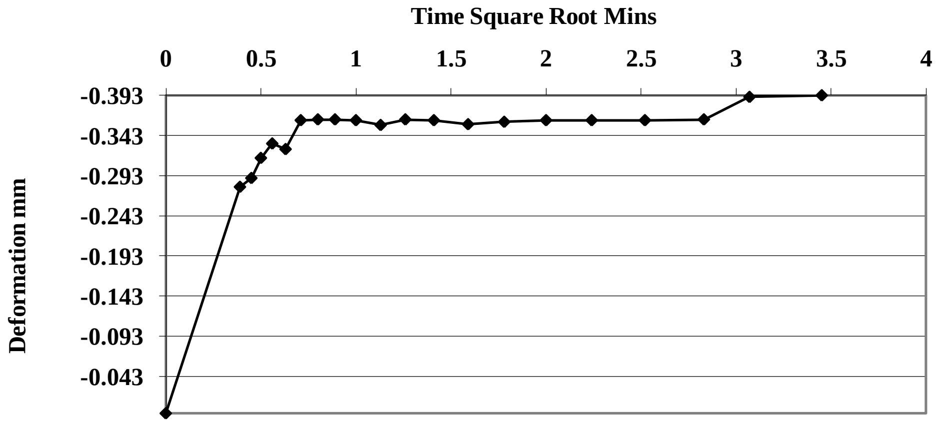
Client	BADEE	**Lab Ref**	
Project		**Job**	BADEE
Borehole		**Sample**	Dry

Test Details			
Standard	ASTM D3080 – 04	**Particle Density**	2.65 Mg/m3
Sample Type	Block sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	25.0 deg.C	**Location**	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	A	**Description**	
Depth within Sample	0.00mm	**Orientation within Sample**	
Initial Height	39.020 mm	**Area**	10000.00 mm2
Preparation		**Initial Moisture Content***	0.0 %
Bulk Density	1.58 Mg/m3	**Dry Density**	1.58 Mg/m3
Initial Voids Ratio	0.6759	**Degree of Saturation**	0.00 %
Dry or Submerged	Dry		
Comments			

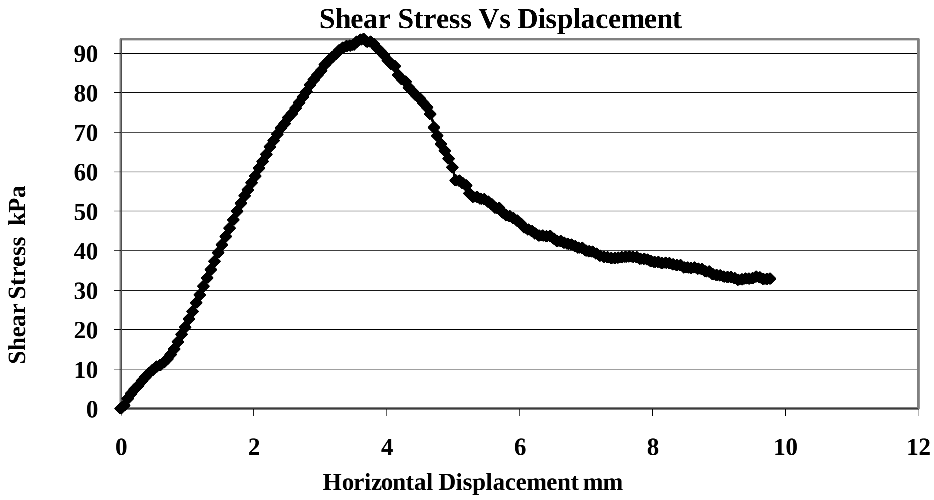
* Calculated from initial and dry weights of whole specimen.

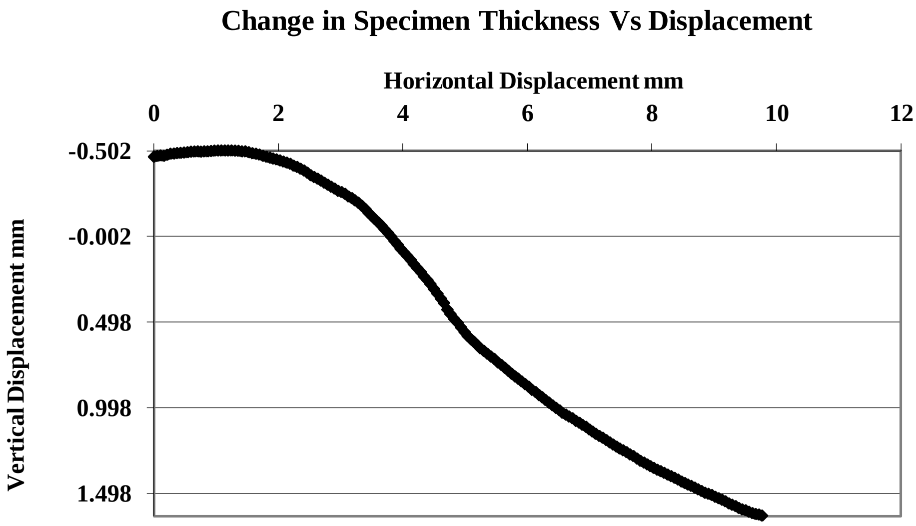
Settlement Vs Square Root Time



A1 Sample Dry A

Rates of Horizontal Displacement	Stage 1: 0.6000mm/min





Conditions at Failure	
Applied Normal Stress	30.9 kPa
Maximum Shear Stress	92.9 kPa
Horizontal Deformation	3.704 mm
Residual Shear Stress	0.0 kPa
Vertical Deformation	-0.043 mm
Cumulative Horizontal Displacement	9.770 mm

A2 Sample Dry B

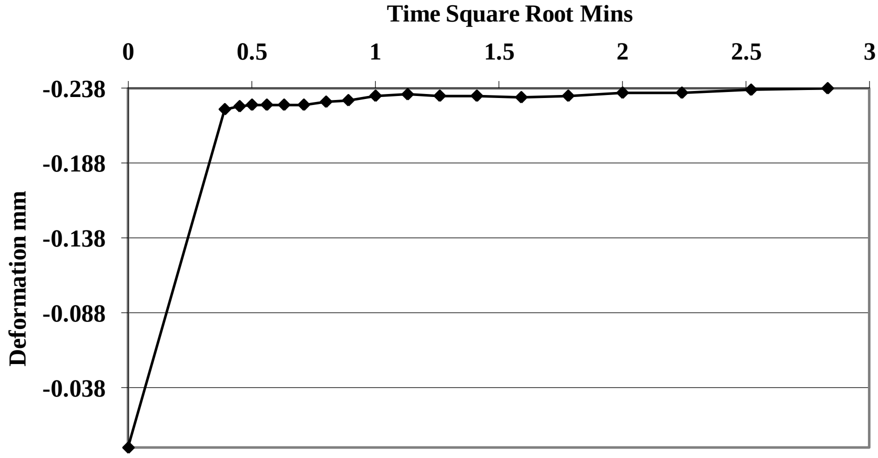
Client	BADEE	**Lab Ref**	
Project		**Job**	BADEE
Borehole		**Sample**	Dry

Test Details			
Standard	ASTM D3080 – 04	**Particle Density**	2.65 Mg/m3
Sample Type	Block sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	25.0 deg.C	**Location**	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	B	**Description**	
Depth within Sample	0.00mm	**Orientation within Sample**	
Initial Height	39.020 mm	**Area**	10000.00 mm2
Preparation		**Initial Moisture Content***	0.0 %
Bulk Density	1.58 Mg/m3	**Dry Density**	1.58 Mg/m3
Initial Voids Ratio	0.6759	**Degree of Saturation**	0.00 %
Dry or Submerged	Dry		
Comments			

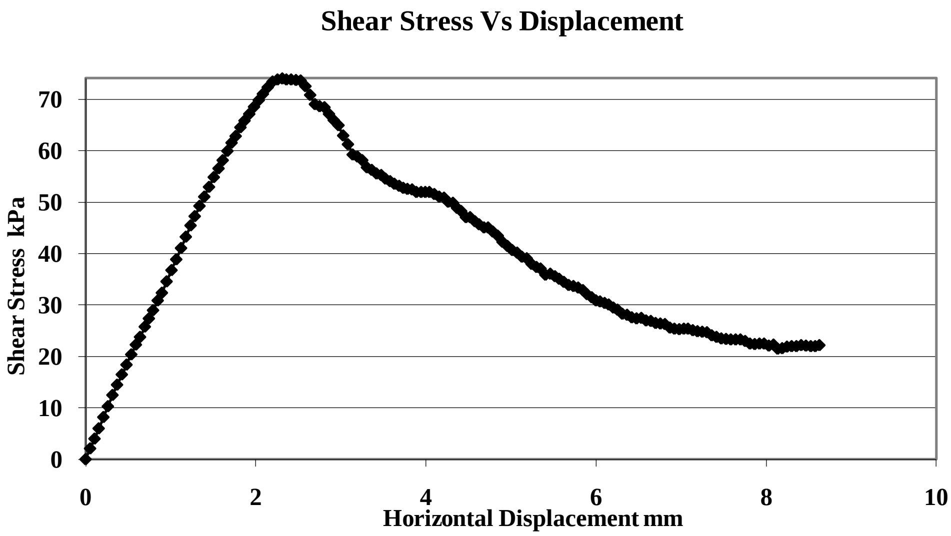
* Calculated from initial and dry weights of whole specimen.

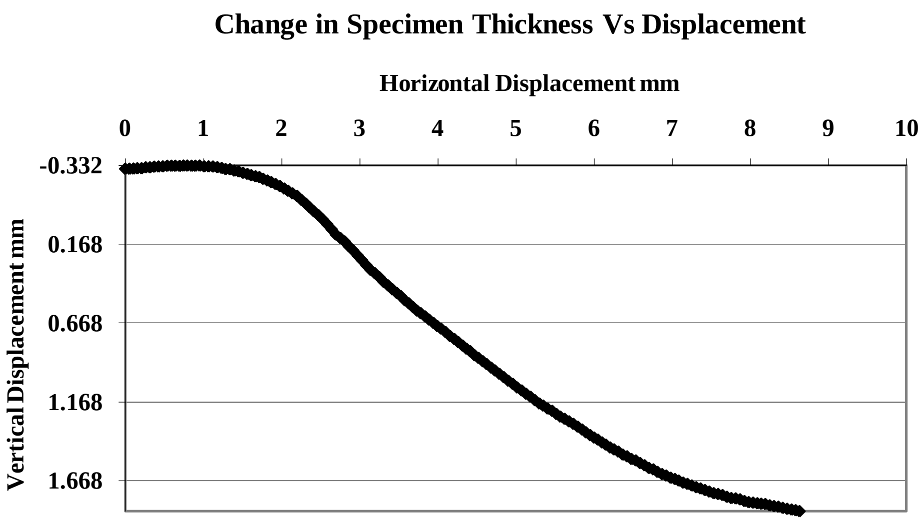
Settlement Vs Square Root Time



A2 Sample Dry B

Rates of Horizontal Displacement	Stage 1: 0.6000mm/min





Conditions at Failure	
Applied Normal Stress	21.1 kPa
Maximum Shear Stress	74.1 kPa
Horizontal Deformation	2.312 mm
Residual Shear Stress	22.1 kPa
Vertical Deformation	-0.090 mm
Cumulative Horizontal Displacement	8.625 mm

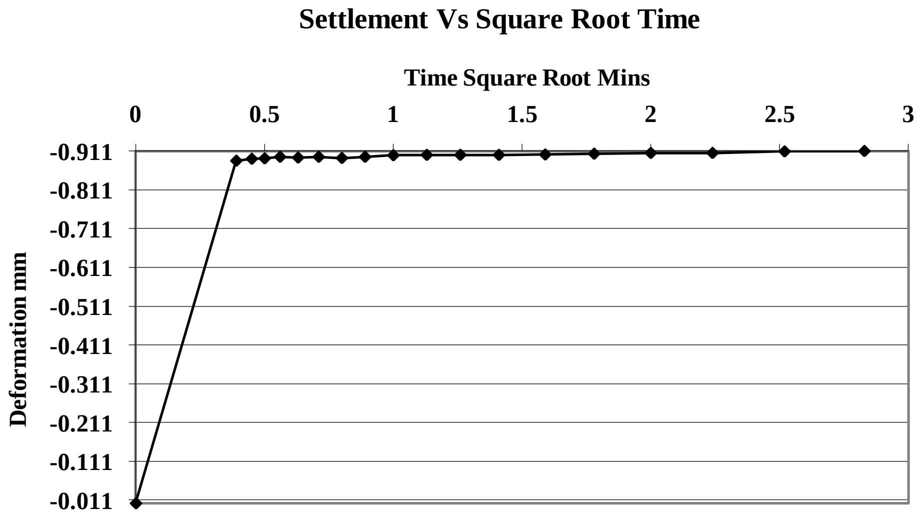
A.3 Sample Dry C

Client	BADEE	**Lab Ref**	
Project		**Job**	BADEE
Borehole		**Sample**	Dry

Test Details			
Standard	ASTM D3080 – 04	**Particle Density**	2.65 Mg/m3
Sample Type	Block sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	25.0 deg.C	**Location**	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	C	**Description**	
Depth within Sample	0.00mm	**Orientation within Sample**	
Initial Height	35.200 mm	**Area**	10000.00 mm2
Preparation		**Initial Moisture Content***	0.0 %
Bulk Density	1.65 Mg/m3	**Dry Density**	1.65 Mg/m3
Initial Voids Ratio	0.6055	**Degree of Saturation**	0.00 %
Dry or Submerged	Dry		
Comments			

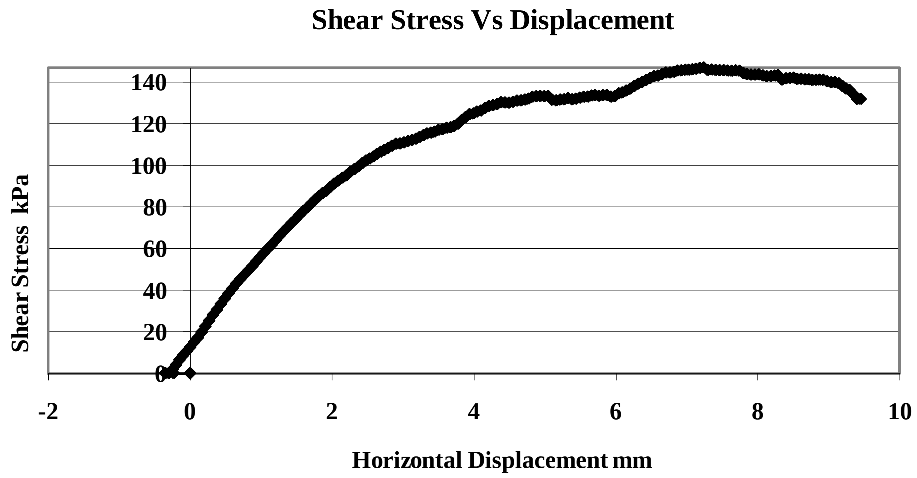
* Calculated from initial and dry weights of whole specimen.

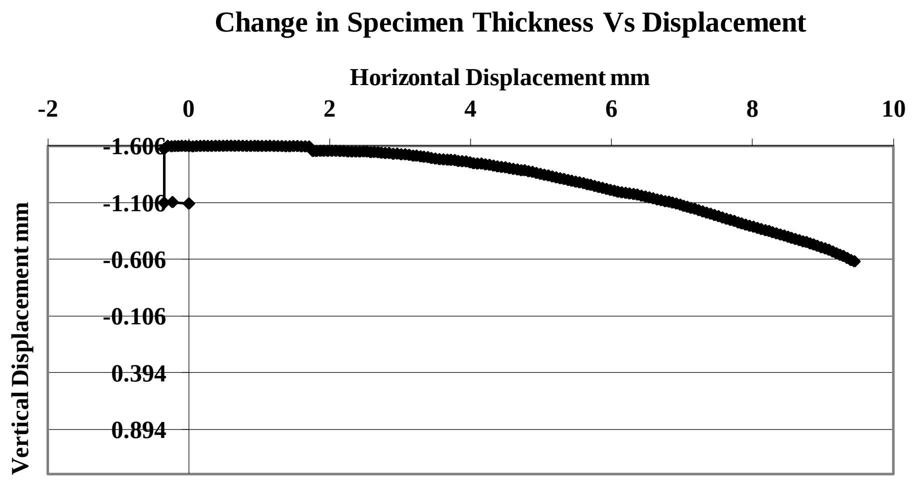


A.3

Sample Dry C

Rates of Horizontal Displacement	Stage 1: 0.6000mm/min





Conditions at Failure	
Applied Normal Stress	50.5 kPa
Maximum Shear Stress	146.6 kPa
Horizontal Deformation	7.181 mm
Residual Shear Stress	0.0 kPa
Vertical Deformation	0.820 mm
Cumulative Horizontal Displacement	9.447 mm

A.4 Sample Dry D

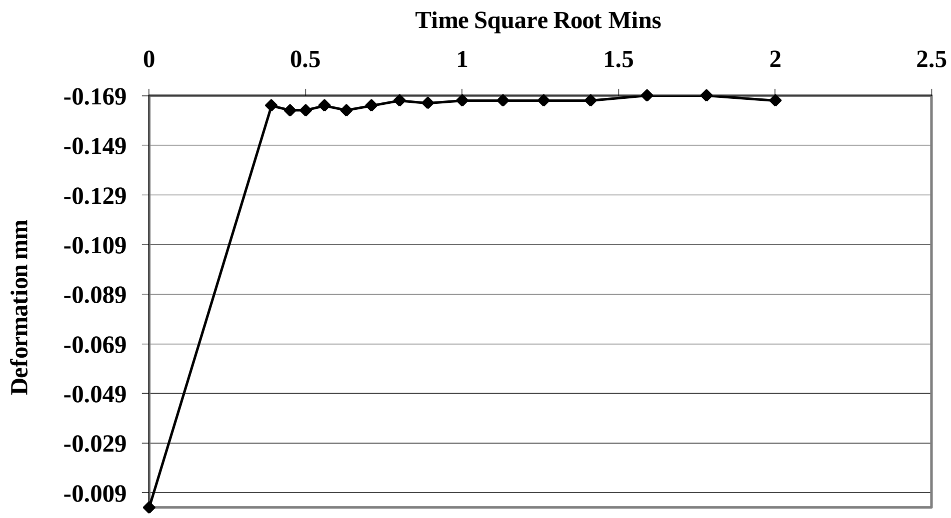
Client	BADEE	**Lab Ref**	
Project		**Job**	BADEE
Borehole		**Sample**	Dry

Test Details			
Standard	ASTM D3080 – 04	**Particle Density**	2.65 Mg/m3
Sample Type	Block sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	25.0 deg.C	**Location**	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	D	**Description**	
Depth within Sample	0.00mm	**Orientation within Sample**	
Initial Height	33.700 mm	**Area**	10000.00 mm2
Preparation		**Initial Moisture Content***	0.0 %
Bulk Density	1.54 Mg/m3	**Dry Density**	1.54 Mg/m3
Initial Voids Ratio	0.7240	**Degree of Saturation**	0.00 %
Dry or Submerged	Dry		
Comments			

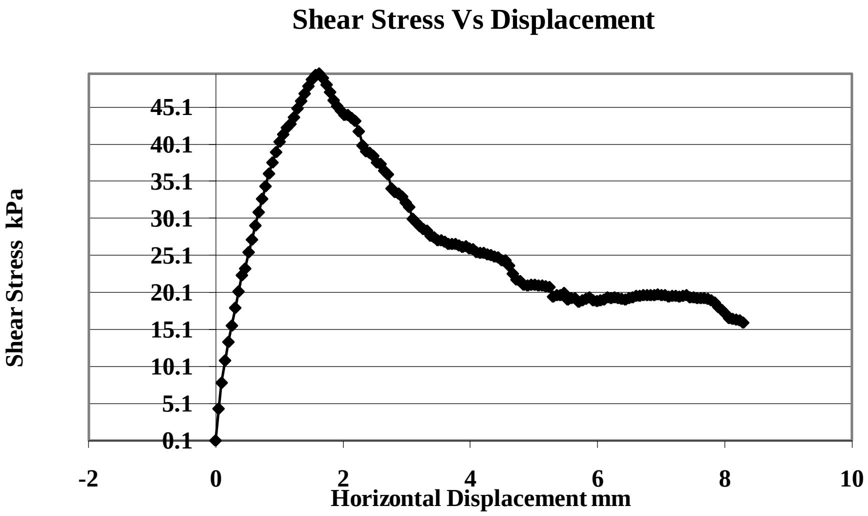
* Calculated from initial and dry weights of whole specimen.

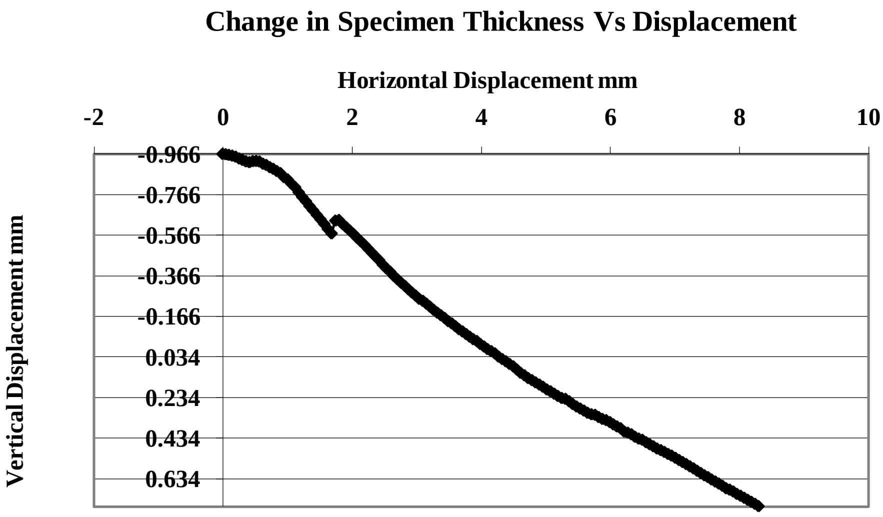
Settlement Vs Square Root Time



A.4 Sample Dry D

Rates of Horizontal Displacement	Stage 1: 0.9000mm/min





Conditions at Failure	
Applied Normal Stress	11.3 kPa
Maximum Shear Stress	49.6 kPa
Horizontal Deformation	1.623 mm
Residual Shear Stress	0.0 kPa
Vertical Deformation	-0.596 mm
Cumulative Horizontal Displacement	8.292 mm

A5. Tests Summary At Dry Condition

Test Summary				
Reference	**A**	**B**	**C**	D
Applied Normal Stress	30.9 kPa	21.1 kPa	50.5 kPa	11.3 kPa
Peak Strength	92.9 kPa	74.1 kPa	146.6 kPa	49.6 kPa
Corresponding Horizontal Displacement	3.704 mm	2.312 mm	7.181 mm	1.623 mm
Residual Shear Stress	-	-	-	
Rate(s) of Shear Displacement	Stage 1: 0.60mm/min	Stage 1: 0.60mm/min	Stage 1: 0.60mm/min	Stage 1: 0.9000mm/min
Final Height	37.38 mm	32.97 mm	33.25 mm	32.13 mm
Cumulative Displacement	9.770 mm	8.625 mm	9.447 mm	8.292 mm
Number of Traverses	1	1	2	1

APPENDIX B

Direct Shear Results For Wet Condition

B.1 Sample Wet A

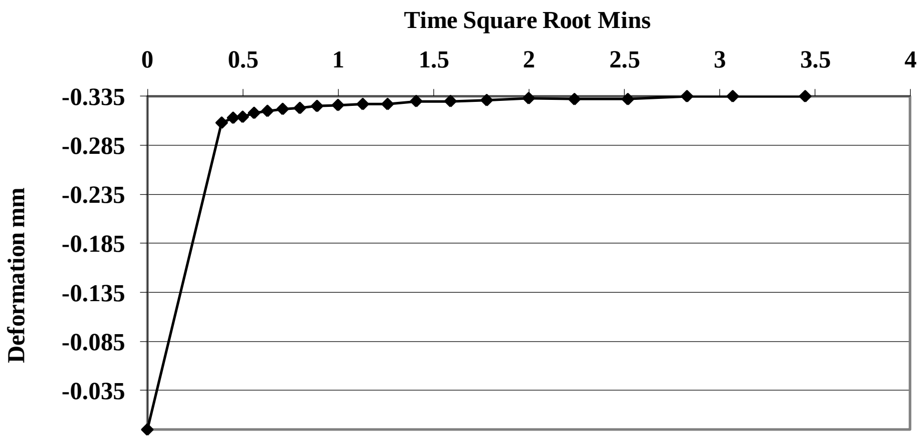
Client	BADEE	**Lab Ref**	
Project		**Job**	BADEE
Borehole		**Sample**	Wet

Test Details			
Standard	ASTM D3080 – 04	**Particle Density**	2.65 Mg/m3
Sample Type	Block sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	25.0 deg.C	**Location**	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	A	**Description**	
Depth within Sample	0.00mm	**Orientation within Sample**	
Initial Height	34.20 mm	**Area**	10000.00 mm2
Preparation		**Initial Moisture Content***	20.0 %
Bulk Density	1.89 Mg/m3	**Dry Density**	1.58 Mg/m3
Initial Voids Ratio	0.6783	**Degree of Saturation**	78.13 %
Dry or Submerged	Dry		
Comments			

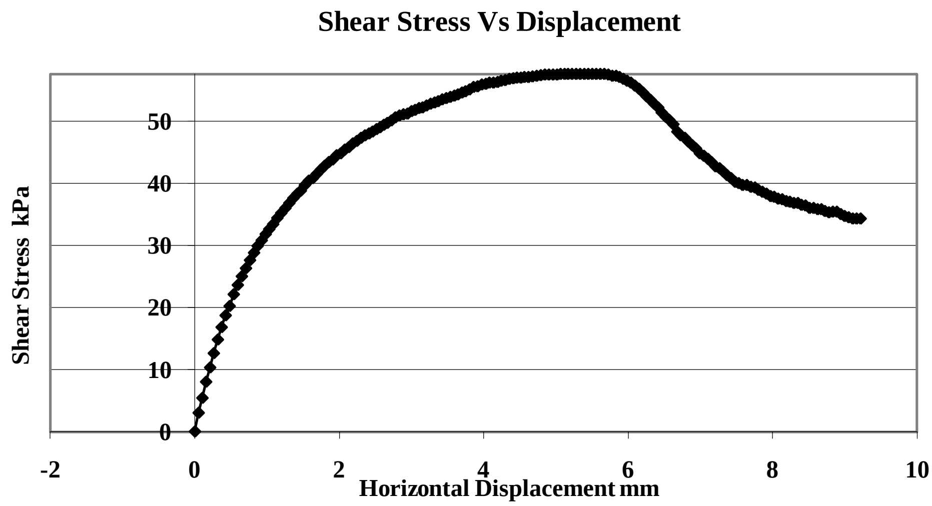
* Calculated from initial and dry weights of whole specimen.

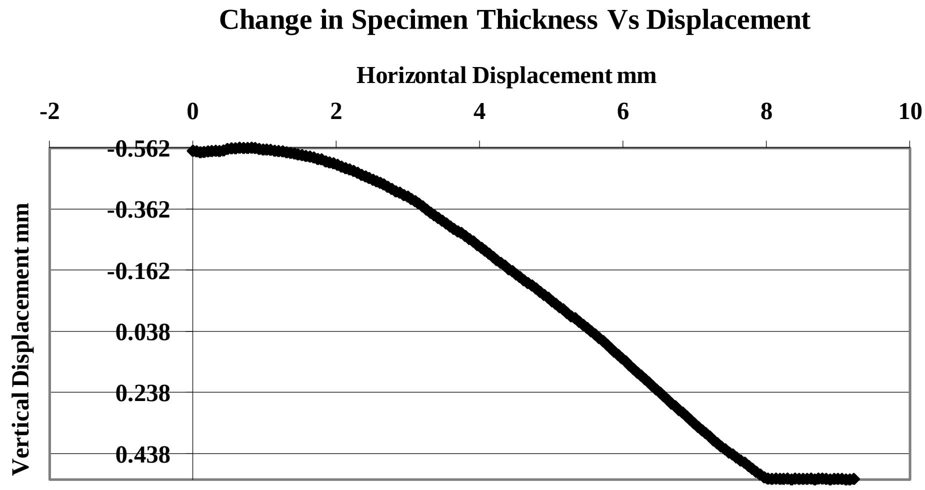
Settlement Vs Square Root Time



B.1 Sample Wet A

Rates of Horizontal Displacement	Stage 1: 0.6000mm/min





Conditions at Failure	
Applied Normal Stress	21.1 kPa
Maximum Shear Stress	57.6 kPa
Horizontal Deformation	5.499 mm
Residual Shear Stress	0.0 kPa
Vertical Deformation	0.028 mm
Cumulative Horizontal Displacement	9.215 mm

B.2 Sample Wet B

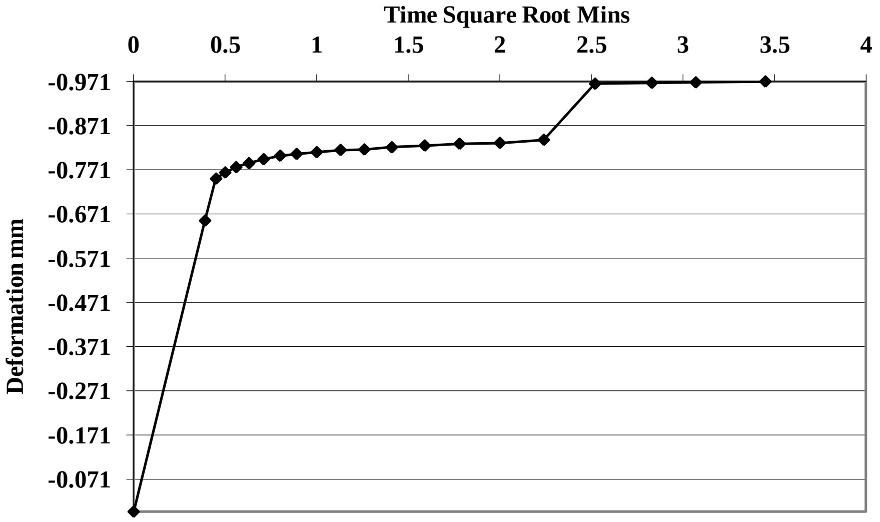
Client	BADEE	Lab Ref	
Project		Job	BADEE
Borehole		Sample	Wet

Test Details			
Standard	ASTM D3080 – 04	**Particle Density**	2.65 Mg/m3
Sample Type	Block sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	25.0 deg.C	**Location**	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	B	**Description**	
Depth within Sample	0.00mm	**Orientation within Sample**	
Initial Height	32.490mm	**Area**	10000.00 mm^2
Preparation		**Initial Moisture Content***	20.7 %
Bulk Density	1.99 Mg/m^3	**Dry Density**	1.65 Mg/m^3
Initial Voids Ratio	0.6056	**Degree of Saturation**	90.38 %
Dry or Submerged	Dry		
Comments			

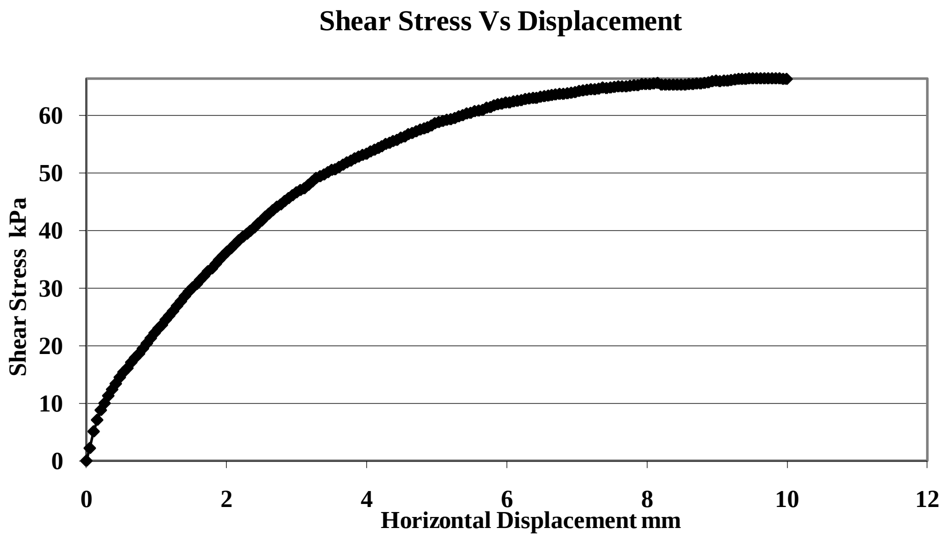
* Calculated from initial and dry weights of whole specimen.

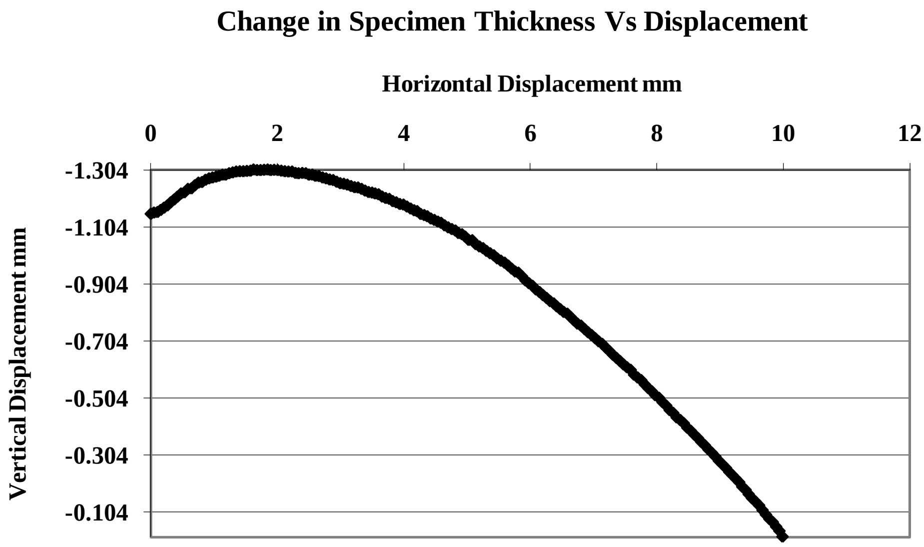
Settlement Vs Square Root Time



B.2 Sample Wet B

Rates of Horizontal Displacement	Stage 1: 0.6000mm/min





Conditions at Failure	
Applied Normal Stress	30.9 kPa
Maximum Shear Stress	66.4 kPa
Horizontal Deformation	9.783 mm
Residual Shear Stress	0.0 kPa
Vertical Deformation	-0.078 mm
Cumulative Horizontal Displacement	9.990 mm

B.3 Sample Wet C

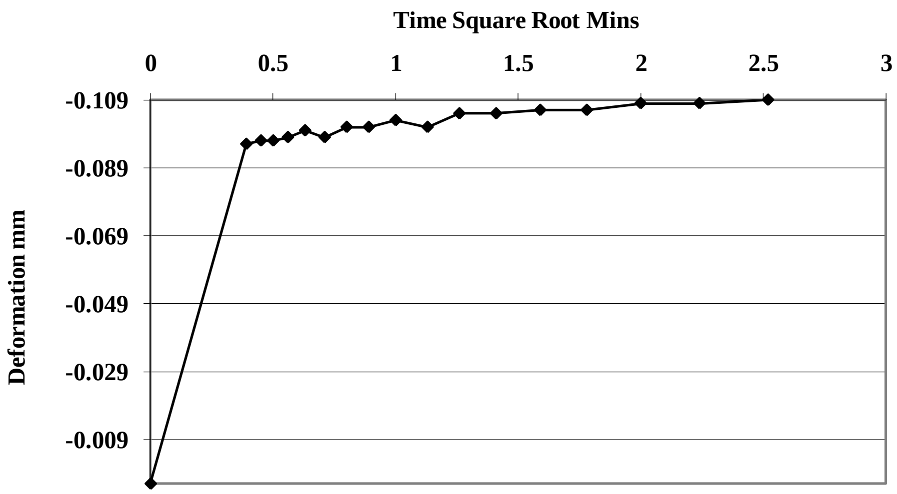
Client	BADEE	Lab Ref	
Project		Job	BADEE
Borehole		Sample	Wet

Test Details			
Standard	ASTM D3080 – 04	**Particle Density**	2.65 Mg/m3
Sample Type	Block sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	25.0 deg.C	**Location**	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	C	**Description**	
Depth within Sample	0.00mm	**Orientation within Sample**	
Initial Height	34.960 mm	**Area**	10000.00 mm2
Preparation		**Initial Moisture Content***	18.1 %
Bulk Density	1.96 Mg/m3	**Dry Density**	1.66 Mg/m3
Initial Voids Ratio	0.5970	**Degree of Saturation**	80.26 %
Dry or Submerged	Dry		
Comments			

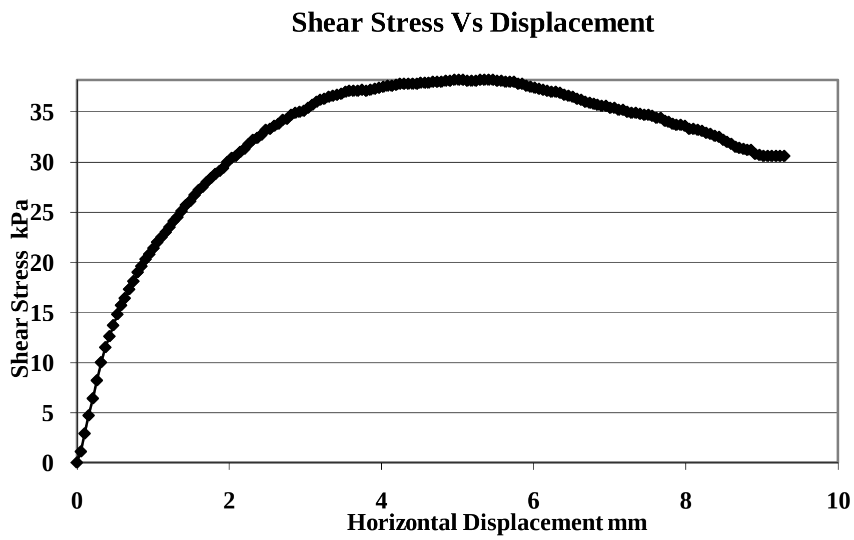
* Calculated from initial and dry weights of whole specimen.

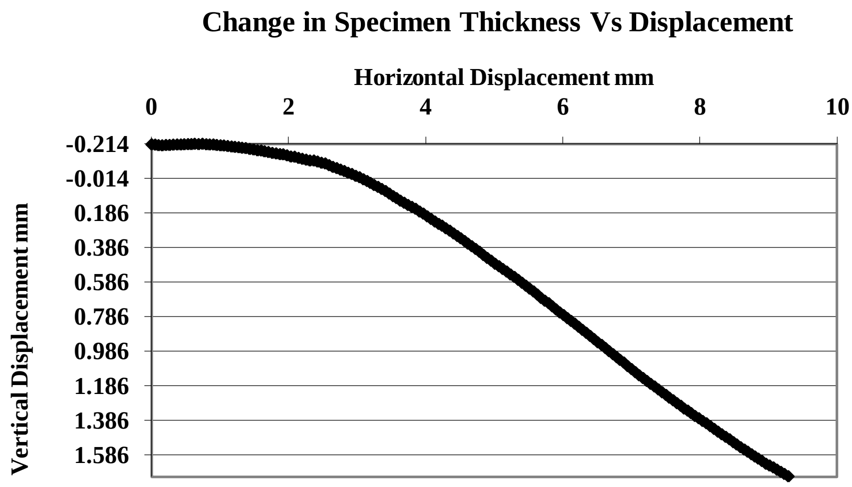
Settlement Vs Square Root Time



B.3 Sample Wet C

Rates of Horizontal Displacement	Stage 1: 0.6000mm/min





Conditions at Failure	
Applied Normal Stress	11.3 kPa
Maximum Shear Stress	38.2 kPa
Horizontal Deformation	5.354 mm
Residual Shear Stress	0.0 kPa
Vertical Deformation	0.575 mm
Cumulative Horizontal Displacement	9.295 mm

B4. Tests Summary At Wet Condition

Reference	**A**	**B**	**C**
Applied Normal Stress	21.1 kPa	30.9 kPa	11.3 kPa
Peak Strength	57.6 kPa	66.4 kPa	38.2 kPa
Corresponding Horizontal Displacement	5.499 mm	9.783 mm	5.354 mm
Residual Shear Stress			
Rate(s) of Shear Displacement	Stage 1: 0. 60mm/min	Stage 1: 0.60mm/min	Stage 1: 0.60mm/min
Final Height	33.47 mm	32.33 mm	33.15 mm
Cumulative Displacement	9.215 mm	9.990 mm	9.295 mm
Number of Traverses	1	1	1

APPENDIX C

Direct Shear Results For Saturated Condition

C.1 Sample Saturated A

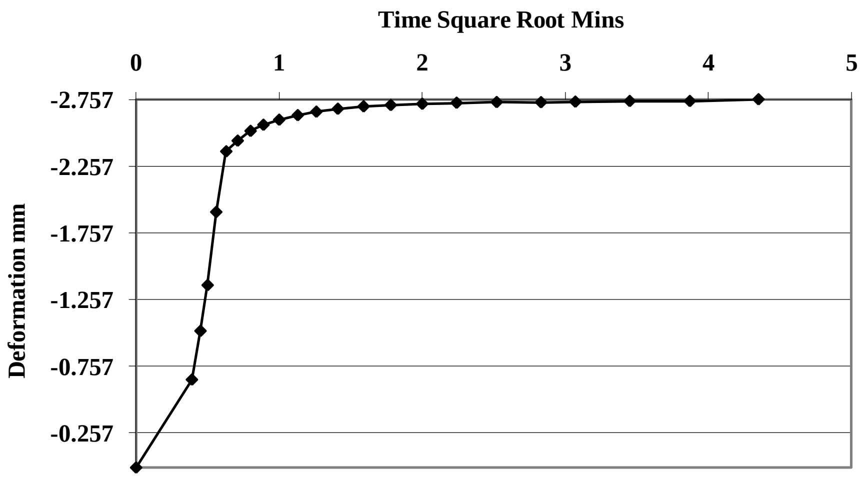
Client	BADEE	**Lab Ref**	
Project		**Job**	BADEE
Borehole		**Sample**	Saturated

Test Details			
Standard	ASTM D3080 – 04	**Particle Density**	2.65 Mg/m3
Sample Type	Block sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	25.0 deg.C	**Location**	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	A	**Description**	
Depth within Sample	0.00mm	**Orientation within Sample**	
Initial Height	27.170 mm	**Area**	10000.00 mm2
Preparation		**Initial Moisture Content***	26.5 %
Bulk Density	1.93 Mg/m3	**Dry Density**	1.52 Mg/m3
Initial Voids Ratio	0.7387	**Degree of Saturation**	95.20 %
Dry or Submerged	Submerged		
Comments			

* Calculated from initial and dry weights of whole specimen.

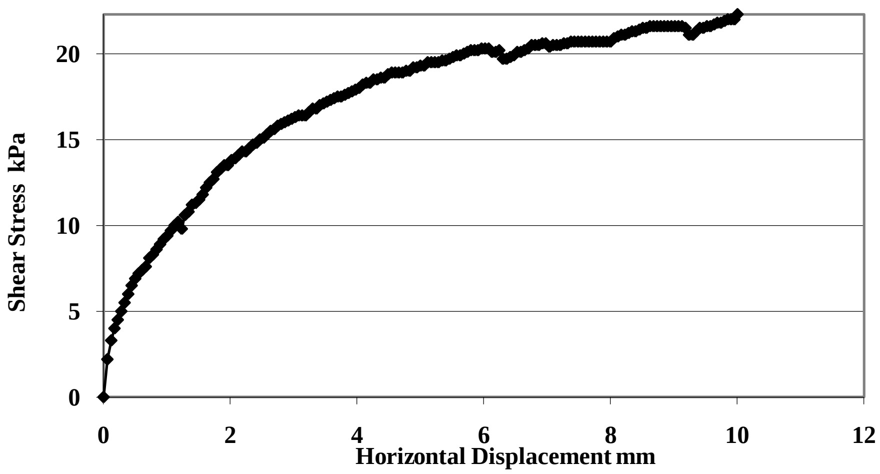
Settlement Vs Square Root Time



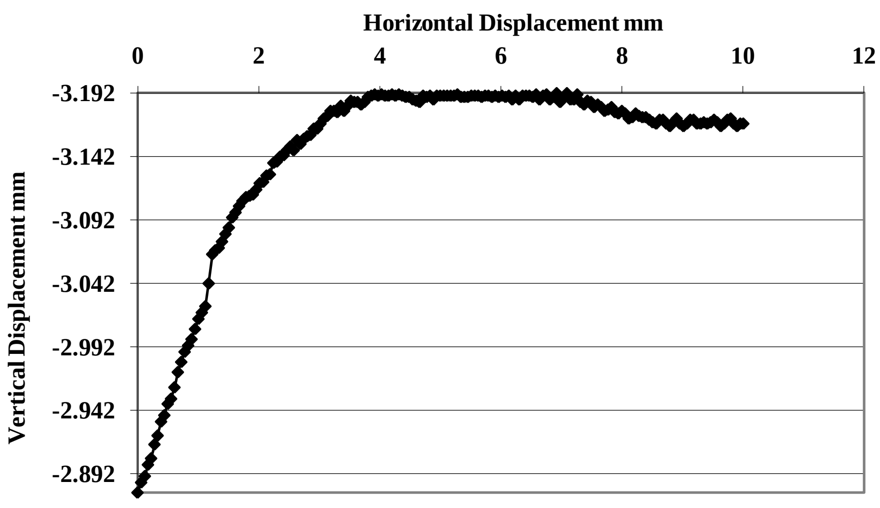
C.1 Sample Saturated A

Rates of Horizontal Displacement	Stage 1: 1.2000mm/min

Shear Stress Vs Displacement



Change in Specimen Thickness Vs Displacement



Conditions at Failure	
Applied Normal Stress	30.9 kPa
Maximum Shear Stress	22.3 kPa
Horizontal Deformation	10.003 mm
Residual Shear Stress	0.0 kPa
Vertical Deformation	-3.168 mm
Cumulative Horizontal Displacement	10.003 mm

C.2 Sample Saturated B

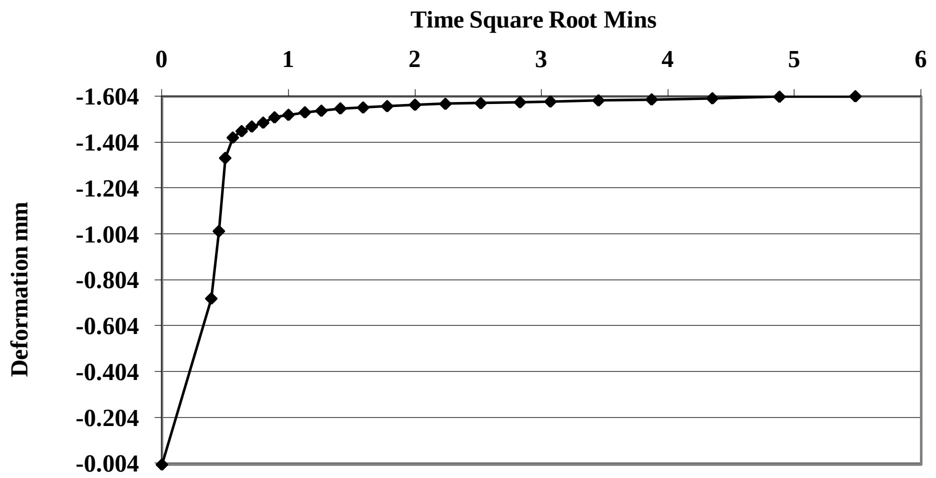
Client	BADEE	**Lab Ref**	
Project		**Job**	BADEE
Borehole		**Sample**	Saturated

Test Details			
Standard	ASTM D3080 – 04	**Particle Density**	2.65 Mg/m3
Sample Type	Block sample	**Single or Multi Stage**	Single Stage
Lab. Temperature	25.0 deg.C	**Location**	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	B	**Description**	
Depth within Sample	0.00mm	**Orientation within Sample**	
Initial Height	33.068 mm	**Area**	10000.00 mm2
Preparation		**Initial Moisture Content***	26.8 %
Bulk Density	1.90 Mg/m3	**Dry Density**	1.50 Mg/m3
Initial Voids Ratio	0.7662	**Degree of Saturation**	92.61 %
Dry or Submerged	Submerged		
Comments			

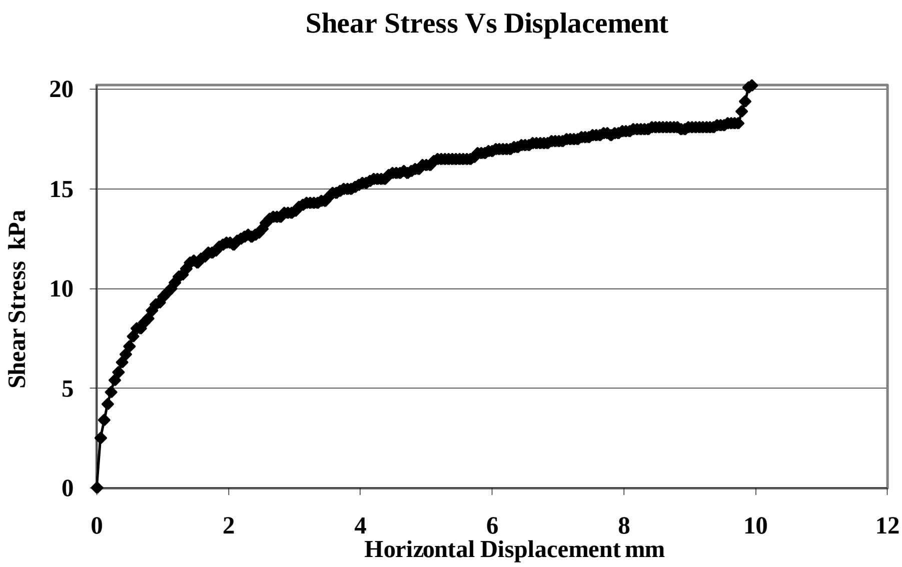
* Calculated from initial and dry weights of whole specimen.

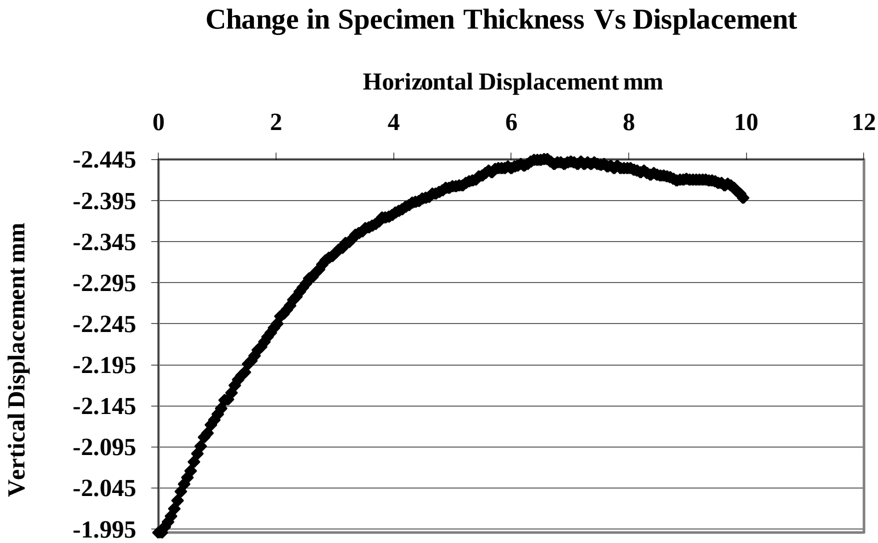
Settlement Vs Square Root Time



C.2 Sample Saturated B

Rates of Horizontal Displacement	Stage 1: 1.2000mm/min





Conditions at Failure	
Applied Normal Stress	21.1 kPa
Maximum Shear Stress	18.3 kPa
Horizontal Deformation	9.737 mm
Residual Shear Stress	0.0 kPa
Vertical Deformation	-2.413 mm
Cumulative Horizontal Displacement	9.945 mm

C.3 Sample Saturated C

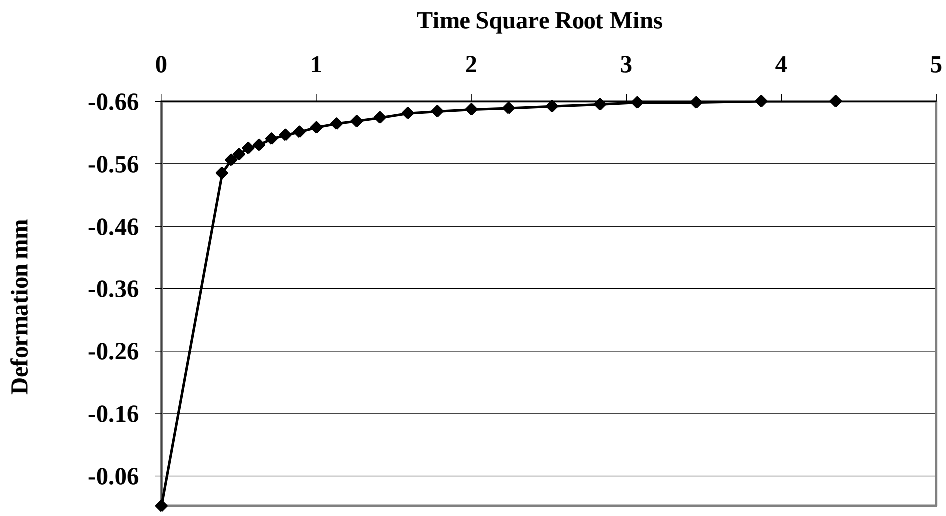
Client	BADEE	Lab Ref	
Project		Job	BADEE
Borehole		Sample	Saturated

Test Details			
Standard	ASTM D3080 – 04	Particle Density	2.65 Mg/m3
Sample Type	Block sample	Single or Multi Stage	Single Stage
Lab. Temperature	25.0 deg.C	Location	
Sample Description			
Variations from procedure	None		

Specimen Details			
Specimen Reference	C	Description	
Depth within Sample	0.00mm	Orientation within Sample	
Initial Height	34.450 mm	Area	10000.00 mm2
Preparation		Initial Moisture Content*	25.3 %
Bulk Density	1.87 Mg/m3	Dry Density	1.49 Mg/m3
Initial Voids Ratio	0.7796	Degree of Saturation	86.14 %
Dry or Submerged	Submerged		
Comments			

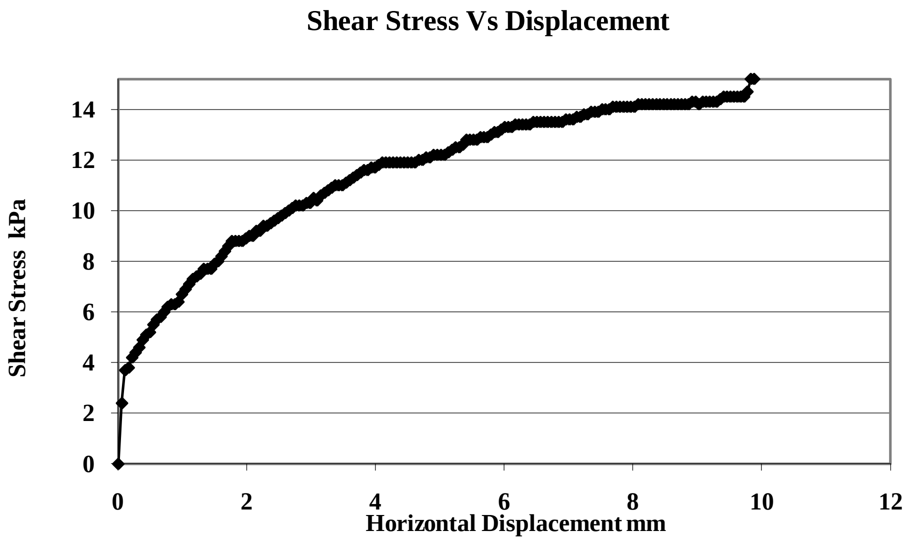
* Calculated from initial and dry weights of whole specimen.

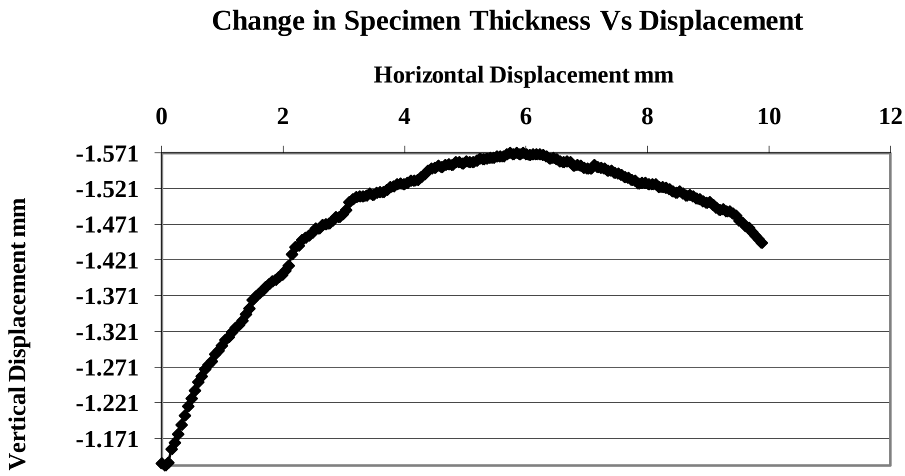
Settlement Vs Square Root Time



C.3 Sample Saturated C

Rates of Horizontal Displacement	Stage 1: 1.2000mm/min





Conditions at Failure	
Applied Normal Stress	11.3 kPa
Maximum Shear Stress	14.2 kPa
Horizontal Deformation	8.864 mm
Residual Shear Stress	0.0 kPa
Vertical Deformation	-1.506 mm
Cumulative Horizontal Displacement	9.884 mm

C.4 Tests Summary At Saturated Condition

Reference	**A**	**B**	**C**
Applied Normal Stress	30.9 kPa	21.1 kPa	11.3 kPa
Peak Strength	22.3 kPa	18.3 kPa	14.2 kPa
Corresponding Horizontal Displacement	10.003 mm	9.737 mm	8.864 mm
Residual Shear Stress			
Rate(s) of Shear Displacement	Stage 1: 1.2000mm/min	Stage 1: 1.2000mm/min	Stage 1: 1.2000mm/min
Final Height	30.22 mm	35.08 mm	35.41 mm
Cumulative Displacement	10.003 mm	9.945 mm	9.884 mm
Number of Traverses	1	1	1