EFFECT OF BOTTOM FUEL ASH ON THE SHEAR STRENGTH OF SOFT SOILS

By

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CERTIFICATION OF APPROVAL

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Civil Engineering Programme
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Approved by:

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Project Supervisor

UNIVERSITI TEKNOLOGI PETRONAS

TRONOH, PERAK

December 2007

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

AzliNa.

NOOR AZLINA BINTI AZHARI

ABSTRACT

The objective of this study is to evaluate the effect of Bottom Fuel Ash (BFA) on strength of soft soils. An effort to improve the strength of soil is mainly due to unsuitability of soil for particular use, such as construction of building, road, dam and any other structures. Soft soils have low shear strength, as well as high moisture content. Because of its weak engineering properties, soft soils usually need to be completely removed from construction site and replaced with selected suitable material. This operation, of course, will require ample time and money. As such, soil stabilization by using additives is introduced to increase the properties of soft soils. In general, untreated soil strength can be improved by adding proportions of additive, Bottom Fuel Ash for instance. Different ratios (percentages by weight) of BFA, 2%, 4%, 5%, 10%, 15%, 20% and 25% were added on soft soils to determine the optimum percentage of BFA for soil stabilization. Meanwhile, the water added for each proportion of BFA was in the increment of 3%. Upon addition of BFA, the compaction characteristics were improved. The maximum dry densities decreased and optimum moisture contents increased with an increase in percentage of BFA. The optimum moisture content for all portion of additive was utilized to investigate the strength of all samples. The test results provided that raw soils gave greatest shear strength, as compared to soils added with proportions of BFA. The samples for immediate test and one day curing showed almost similar trend. The shear strength was decreased with addition of more BFA on soils up to 20%, and then it rose slightly higher when 25% of BFA was added. The trend clearly tells that addition of BFA up to 20% does not improve the shear strength of treated soil. But the improvement can be seen at 25% BFA. Nevertheless, at an early stage, the addition of BFA is expected to increase engineering properties of soft soils, thus gives higher shear strength. Even though the result does not show a trend of improvement, it does not mean that BFA is not suitable for stabilization. Some changes can be recommended to rectify the properties of BFA to improve the impact on treated soils.

ABSTRACT

The objective of this study is to evaluate the effect of Bottom Fuel Ash (BFA) on strength of soft soils. An effort to improve the strength of soil is mainly due to unsuitability of soil for particular use, such as construction of building, road, dam and any other structures. Soft soils have low shear strength, as well as high moisture content. Because of its weak engineering properties, soft soils usually need to be completely removed from construction site and replaced with selected suitable material. This operation, of course, will require ample time and money. As such, soil stabilization by using additives is introduced to increase the properties of soft soils. In general, untreated soil strength can be improved by adding proportions of additive, Bottom Fuel Ash for instance. Different ratios (percentages by weight) of BFA, 2%, 4%, 5%, 10%, 15%, 20% and 25% were added on soft soils to determine the optimum percentage of BFA for soil stabilization. Meanwhile, the water added for each proportion of BFA was in the increment of 3%. Upon addition of BFA, the compaction characteristics were improved. The maximum dry densities decreased and optimum moisture contents increased with an increase in percentage of BFA. The optimum moisture content for all portion of additive was utilized to investigate the strength of all samples. The test results provided that raw soils gave greatest shear strength, as compared to soils added with proportions of BFA. The samples for immediate test and one day curing showed almost similar trend. The shear strength was decreased with addition of more BFA on soils up to 20%, and then it rose slightly higher when 25% of BFA was added. The trend clearly tells that addition of BFA up to 20% does not improve the shear strength of treated soil. But the improvement can be seen at 25% BFA. Nevertheless, at an early stage, the addition of BFA is expected to increase engineering properties of soft soils, thus gives higher shear strength. Even though the result does not show a trend of improvement, it does not mean that BFA is not suitable for stabilization. Some changes can be recommended to rectify the properties of BFA to improve the impact on treated soils.

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Noor Azlina Azhari

Universiti Teknologi PETRONAS

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND OF STUDY

Soils are naturally occurring materials that are subject to classification tests to provide a general concept of their engineering characteristics. Soils are needed as bearing layers for foundation. Nevertheless, some soils have unfavourable properties, including high permeability, low shear strength and low bearing pressure. When a load is placed on a soil, the soil deforms and the foundation settles. If the load is increased, the settlement increases. "As the load increases, a point will be reached beyond which the settlement will increase much more rapidly. In some soils, particularly sensitive clays, continuous movement will occur at loads greater than this limit, while in others, the rapid downward movement will eventually stop until the load is again increased", Sowers (1962). Soft soils, clay and peat for example are not suitable to be used as the foundation for structures and can be found in abundance in Malaysia. Balkema (1996) stated that "virtually, no clay is used for construction applications and even for non- structural applications, relatively little clay is employed". The need for constructing structures over these types of soils makes it necessary to investigate the behavior of such soils when subjected to loading. Nevertheless, there are two techniques available to encounter the problem, either to treat, or remove and replace the soils with suitable materials. However, this operation, of course will require ample time and money. As such, soil treatment was introduced to encounter the problem.

Soil treatment, also known as **soil stabilization**, "in the broadest sense is the alteration of any property of soil to improve its engineering performance", Lambe (1962). Soil stabilization is only one of several techniques available to increase the engineering properties of soil in order to serve as the founding for structures. It is most commonly applied for the strengthening of the soil components of highway and airfield pavements. Soil stabilization has become a solution to a number of unfavourable soil properties. Therefore, many attempts have been developed until soil stabilization is now used to alter almost every engineering properties of soil. One of them is stabilization by additives, lime and fly ash for examples. Many researches had found out that fly ash contains pozzolanic material properties and has the potential application to stabilize soft subgrade soil. The same goes to lime. "Calcium cations supplied by the hydrated lime replace the cations normally present on the surface of the soil mineral, promoted by the high pH environment of the lime-water system. Thus, the soil surface mineralogy is altered, producing some improvement to treated soil", Bagherpour and Choobbasti (2003).

Lambe (1962) disclosed that "the design of additive- stabilized soil consists of selecting the stabilizer, determining the amount and method of application of stabilizer, and determining the extent if soil to be stabilized". Previous researches intended to alter many unfavourable properties of soft soils, including California Bearing Ratio (CBR), permeability, shear strength and many more. Meanwhile, this paper is intended to study the effect of soil stabilization on shear strength of soft soils. Atkinson disclosed that "**shear strength** of soils refers to its ability to resist shear stresses". Shear stresses exist in a sloping hillside or result from filled land, weight of footings, and so on. If a given soil does not have sufficient shear strength to resist such shear stresses, failures in the form of landslides and footing failures may occur. Lin (2006) indicated that "untreated soil strength can be improved by adding a proportion of additives. However, the strength of soil improved may be reduced when the additives added reach a certain amount". Therefore, a series of laboratory works need to be carried out to determine the optimum content of additives suitable for soil stabilization.

1.2 PROBLEM STATEMENT

Soft, swampy and organic soils are generally plentiful and abundant in Malaysia. They are highly compressible and possessing high moisture content, high plasticity index, as well as low shear strength and are prone to large settlements. The construction of earth structures into highly compressible subsoils of low bearing capacity is almost impossible. Therefore, engineering behaviour of these types of soils need to be improved first, by using different ground improvement techniques such as deep compaction, stone columns, sand drains with preloading or stabilization with various additives.

One other technique is to remove and replace the soils with suitable material. However, this operation is costly. One crude example that can be appointed is railway double-track project in Batu Gajah, Perak. It took about RM150 million to remove and replace the soil. This is a whole lot of money to spend. As such, soil stabilization is more effective to be applied at optimum cost and more effective way to increase the properties of soft soils.

This research will only concentrate in improving properties of soils through soil stabilization by using additive. Previously, many researchers had found out that Lime (quick lime, calcium chloride), Portland cement, fly ash, bitumen and asphalt, bentonite and combinations are effective additives to be mixed with soft soils for soil stabilization. However, this study will only investigate the effectiveness of bottom fuel ash as the additive for soil stabilization. The success of improving shear strength in this research can be evaluated from the trend of increment of soils' strength after some laboratory experiments are conducted.

This research is only a part of bigger research that being done within Civil Engineering department in Universiti Teknologi Petronas. All researchers are each concentrating on improvement of shear strength or California Bearing Ratio (CBR) of soft soils by using different additives, namely bottom and fly fuel ash, and lime.

1.3 OBJECTIVES AND SCOPE OF STUDY

The objective of this research is to study the effects of using Bottom Fuel Ash (BFA) as the stabilizer for soft soils. Besides, it is intended to investigate the potential of BFA to improve shear strength of soft soils, as well as optimum percentage required to increase its engineering properties. Also, to identify which stabilizer (bottom ash, fly Ash or lime) is the most effective to improve the properties of soils. Therefore, a series of experimental program must be conducted to achieve the objectives. They include soil classification tests, additive properties tests and followed by main testing program, which is strength test in order to analyze the trend and evaluate optimum percentage of BFA suitable for soil stabilization.

CHAPTER 2

LITERATURE REVIEW AND THEORY

2.1 SOFT SOILS IN MALAYSIA

"Organic (peat) soils are often found in Malaysia and Southeast Asia. In general, they cover 7.2% of the total land area in Malaysia", Hobbs (1986). On the other hand, there are extensive deposits of very soft normally consolidated cohesive clay in Malaysia and other parts of the world. The distribution of quaternary sediments in Peninsular Malaysia is presented in Figure 2.1 below.

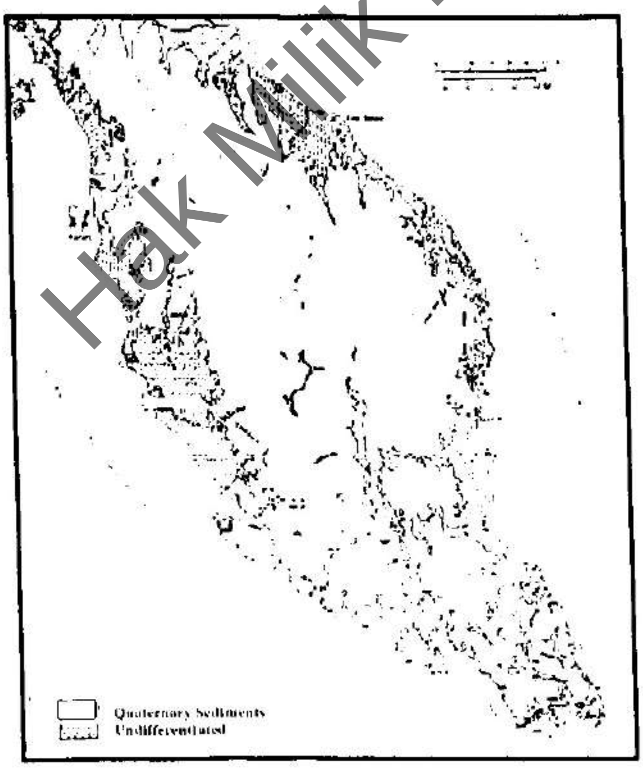

Figure 2.1 : Quaternary sediments in peninsular Malaysia (after Stauffer, 1973)

In both East and West Malaysia, "the depth of soft clay may exceed 40m. The water content of the soft clay is generally high, typically about 60 to 80%. The undrained shear strength is generally low, between 7 to 12 kPa", Pitts (1984). Organic clays are compressible soils possessing high moisture contents. In Malaysia, these types of soils can be found in abundance especially in the state of Perak. Mahmood et al. (2002) had reported that liquid and plastic limits of this organic clay were found to be 83.5% and 48.1% respectively. This gives the plasticity index of the soil as being equal to 35.4%. It has been found also that the soil had an organic matter content of 11.1% with an average specific gravity of 2.54. The result of their study will be compared with this research's and analyze whether these data are comparable and can be utilized for soil improvement.

Soft soils are well known for their low strength and high compressibility, which resulting to settlements. "It is however known that the consolidation behaviour of soil is closely related to ultimate failure behaviour which occurs as a result of shear stresses. High compressible soil is generally of low bearing capacity and is commonly refer as 'soft', which means that it has poor resistance to deformation and has low bearing capacity", Balkema (1996). Usually, due to sedimentary process on different environments, both physical and engineering properties (namely void ratio, water content, grain size distribution, compressibility, permeability and strength) show a significant variation. Further, they exhibit high compressibility, reduced strength, low permeability and compactness, and consequently low quality for construction.

"Peat and organic soil represent the extreme form of soft soil. They are subject to instability such as localized sinking and slip failure, and massive primary and long-term settlement when subjected to even moderate load increase", Jarrett (1995). Buildings on peat are usually suspended on piles, but the ground around it may still settle. In addition, there are discomfort and difficulty of access to the sites, a tremendous variability in material properties and difficulty in sampling.

For this research, soft soil was taken at Ipoh-Rawang Railway Double Track Project site in Batu Gajah, Perak.

2.2 BOTTOM FUEL ASH

Ash is by-product of coal combustion produced by thermal power plants. Two types of ash produced by burning coal are bottom and fly ash. Both ashes are also known as Coal Combustion Products (CCPs). When coal is burned, it leaves behind ash. Some ashes fall to the bottom of the boilers (bottom ash) and some are carried upward by the hot gases (fly ash). Bottom ash is an almost sand-like material that is sluiced from the bottom of the boilers. Bottom ash is agglomerated ash particles and formed in pulverized coal furnaces, which are too large to be carried in the flue gases. It impinges on the furnace walls or fall through open grates to an ash hopper at the bottom of the furnace. Physically, bottom ash is grey to black in color, quite angular, and has a porous surface structure.

Bottom ash is a pozzolanic material and has been classified into two classes, F and C, based on the chemical composition. For comparison purpose, the chemical compositions to classify any fly ash (ASTM C 618) are shown in Table 2.1.

Table 2.1. Chemical compositions for Fly Ash classification

Properties	Fly Ash Class	
	Class F	Class C
Silicon dioxide (SiO_2) plus aluminum oxide (Al_2O_3) plus iron oxide (Fe_2O_3), min, %	70.0	50.0
Sulfur trioxide (SO_3), max, %	5.0	5.0
Moisture Content, max, %	3.0	3.0
Loss on ignition, max, %	6.0*	6.0
* The use of class F fly ash containing up to 12% loss of ignition may be approved by the user if acceptable performance results are available		

The main difference between Class F and Class C bottom ash is in the amount of calcium and the silica, alumina, and iron content in the ash. In Class F ash, total calcium typically ranges from 1 to 12 percent, mostly in the form of calcium hydroxide, calcium sulfate, and glassy components in combination with silica and alumina. In contrast, "Class C ash may have reported calcium oxide contents as high as 30 to 40 percent", McKerall et. al. (1982).

Another difference between Class F and Class C is that the amount of alkalis (combined sodium and potassium) and sulfates (SO_4) are generally higher in the Class C ashes than in the Class F ashes.

Class F bottom ash is produced from burning anthracite and bituminous coals. This ash has siliceous or siliceous and aluminous material, which itself possesses little or no cementitious value but in the presence of moisture, will chemically react with calcium hydroxide at ordinary temperature to form cementitious compound. Class C bottom ash is produced normally from lignite and sub-bituminous coals and usually contains significant amount of Calcium Hydroxide (CaO) or lime. This class of ash, in addition to having pozzolanic properties, also has some cementitious properties (ASTM C 618-03).

Color is one of the important physical properties of bottom ash in terms of estimating the lime content qualitatively. It is suggested that lighter color indicate the presence of high calcium oxide and darker colors suggest high organic content.

Bottom fuel ash is largely produced during coal combustion and is abundant. As it contains toxic elements, it is hazardous to environment, as well as human beings. Hence, it is necessary to dispose the bottom ash in the most proper way. One of the techniques is to utilize it in geotechnical and concreting purposes. It has been used as filler in concrete casting, backfill materials, base course materials and embankment materials besides being used as a stabilizing agent. Its pozzolanic material properties provide a potential application to stabilize soft subgrade soils. However, bottom ash is not widely used in geotechnical purposes, as compared to fly ash and lime.

2.2.1 Hydration of Bottom Ash

Hydration is the formation of cementitious material by the reaction of free lime (CaO) with the pozzolans (AlO_3, SiO_2, Fe_2O_3) in the presence of water. The hydrated calcium silicate gel or calcium aluminate gel (cementitious material) can bind inert material together. For class C ash, the calcium oxide (lime) of the ash can react with the siliceous and aluminous materials (pozzolans) of the ash itself. Since the lime content of class F ash is relatively low, addition of lime is necessary for hydration reaction with the pozzolans of the ash. For lime stabilization of soils, pozzolanic reactions depend on the siliceous and aluminous materials provided by the soil. The pozzolanic reactions are as follows:

$$Ca(OH)_2 \rightarrow Ca^{++} + 2[OH]^-$$

$$Ca^{++} + 2[OH]^- + \underset{\text{(silica)}}{SiO_2} \rightarrow \underset{\text{(gel)}}{CSH}$$

$$Ca^{++} + 2[OH]^- + Al_2O_3 \rightarrow CAH$$

(alumina) (gel)

Hydration of tricalcium aluminate in the ash provides one of the primary cementitious products in many ashes. The rapid rate at which hydration of the tricalcium aluminate occurs results in the rapid set of these materials.

The hydration chemistry of bottom ash is very complex in nature. So the stabilization application must be based on the physical properties of the ash treated stabilized soil and cannot be predicted based on the chemical composition of the ash. The contractors mixed ash, created when coal is burned for fuel, into the moist soil to form a stiff substance. Because bottom ash is a waste product that might be sent to landfills if not used, it's relatively cheap but also ecologically sound.

2.3 OTHER ADDITIVE

2.3.1 Lime

Lime is dry powder obtained by treating quicklime with sufficient water to satisfy its chemical affinity for water, thereby converting the oxides to hydroxides. Depending upon the type of quicklime used and the hydrating conditions employed, the amount of water in chemical combination varies. Pozzolanic reaction of lime occurs when lime reacts with silica and aluminum contain in clay mineral. This reaction proceed with time, as such, strength also increase with time. Organic content affect the volume of lime being used. The amount of lime content is proportional to the organic content. Minimum 20% of silica and aluminum in clay mineral is required for pozzolanic reaction to take place. Besides, water content affects the effectiveness of lime stabilization. The lower the water content, the higher the shear strength of soils.

2.3.2 Fly Ash

Fly ash is finely divided residue that results from the combustion of ground and powdered coal used for power generation. ASTM C618 defines two classes of fly ash for use in concrete:

i. Class F, usually derived from the burning of anthracite or bituminous coal
ii. Class C, usually derived from the burning of lignite or subbituminous coal.

ASTM C618 also delineates requirements for the physical, chemical, and mechanical properties for these two classes of fly ash. Class F fly ash is pozzolanic, with little or no cementing value alone. Class C fly ash has self-cementing properties as well as pozzolanic properties. This pozzolanic material has been used as an additive to concrete for many years to improve workability and increase compressive strength. Its characteristic of becoming cementitious in the presence of moisture has led to its use in improving the strength of weak soils.

2.4 SOIL STABILIZATION AND IMPROVEMENT

An ideal building site is one whose foundation soils provides a safe as well as an economical design, be it for building, pavement or dam. Some sites do posses the qualities, but some do not. Soil stabilization is one of the methods available to achieve the qualities.

Soft soils exhibit high compressibility, reduced strength, low permeability and compactness, and consequently low quality for construction. For the aforementioned reasons, a comprehensive laboratory testing program were carried out by many researches in order to study the effect of additives on engineering behaviour of soft soils and to improve some desired properties of soft soils. "Soil improvement is frequently termed as soil stabilization which in its broadest sense is the alteration of any property of a soil to improve its engineering performance" (Germaine, 2003). It is a physical means whereby a soil can have its physical properties improved to increase bearing capacity, increase soil shear strength, decrease settlement, reduce compressibility and reduce soil permeability. "Soil reinforcement is considered an ideal method to strengthen these types of soils and has been used in Malaysia for the rehabilitation of soft soils", Toh et al. (1994). In addition to its other benefits, reinforcement provides the soil with the tension force that strengthens it and makes it capable of supporting more loads than it usually can.

There are variety techniques of stabilization, namely mechanical stabilization, chemical stabilization, electrical stabilization, thermal stabilization, stabilization by drainage, by heating and by cooling. Those techniques can be classified in various ways, according to the nature of the process involved, material added, desired result, etc. In general, the method of soil stabilization is determined by the amount of stabilizing required and the conditions encountered on the project. An accurate soil description and classification is essential to the selection of the correct materials and procedures.

To serve as founding for structures, soils must have good bearing layer. Therefore, weak bearing layer of soils must be treated first, so that they are capable to support high loads. The most common improvements achieved through stabilization includes better soil gradation, reduction of plasticity index or swelling potential, and increases in durability and strength. In wet weather areas, stabilization may also be used to provide a working platform for construction operations. These types of soil quality improvement are also referred as soil modification.

Soil stabilization is now used to alter almost every engineering properties of soft soil. Lambe (1962) indicated that the purposes of soil stabilization are as follows:

1. Increase or decrease strength, or reduce the sensitivity of strength to environment changes, especially moisture changes
2. Increase or decrease permeability
3. Reduce compressibility
4. Reduce frost susceptibility

The increase strength of the existing soil is to enhance its load-bearing capacity surface penetration, which is accomplished by placing a soil treatment material directly to the existing ground surface by spraying or distribution. Untreated soil strength can be improved by adding some proportions of additives. Some of the additives widely used nowadays in soil stabilization are fly fuel ash and lime. "Many research results have found that fly ash contains pozzolanic material properties and has the potential application to stabilize soft subgrade soil" (Lin, 2006). The same goes to lime. Bagherpour and Choobbasti (2003) validated that "calcium cations supplied by the hydrated lime replace the cations normally present on the surface of the soil mineral, promoted by the high pH environment of the lime-water system. Thus, the soil surface mineralogy is altered, producing some improvement to treated soil". However, Lin (2006) pointed out that "when the admixture added reaches a certain level, the strength of soil improved could be reduced".

Significant efforts concerning the effect of soil stabilization on various properties of soil have been conducted by various researchers beforehand. Cernica (1995) had concluded that "the engineering properties of cohesionless soils are significantly affected by the relative density of the soil, and not as much by the many variables cited in connection with the compaction of cohesive soils. Generally, an increase in density increases the shear strength of the soil and reduced its compressibility. On the other hand, for a given compacting effort, the density also increase in water content, up to a point, then it decreases with a further increase in water content. Hence, density is usually the only specified criterion for the compaction of cohesionless soil. The degree of moisture is not a specified criterion, as may be frequently the case for cohesive soils".

Bottom ash is rarely used for soil stabilization due to its inactive pozzolanic reaction. However, according to Jaturapitakkul and Cheerarot, 2003, they found out that "the quality of bottom ash can be improved by grinding until the particle size retained on sieve 325 was less than 5% by weight". Bottom ashes before and after being ground were investigated and compared for their physical and chemical properties. Moreover, they verified that, "the results indicated that the particle of bottom ash was large, porous and irregular shapes. The grinding process reduced the particle size as well as porosity of the bottom ash. They had came into conclusion that "the compressive strength of mortar containing 20 – 30% of bottom ash cement replacement were much less than that of cement mortar at all ages, but the use of ground bottom ash produced higher compressive strength than the cement mortar after 60 days. With the cement content in ground bottom ash concrete of 440 and 260 kg/m^3, the concrete needed 14 and 60 days, respectively to develop higher compressive strength than that of the concrete without bottom ash. As a result of compressive strength, it was concluded that ground bottom ash could be used as a good pozzolanic material".

Various studies had been carried out by Lin et al. (2006), who had investigated the effect of sludge ash/hydrated lime on the geotechnical properties of soft soil. In the study, "tests were conducted with the effective confining pressure at 25 and 50kPa to find the shear stresses. From their tests, they have found out that the cohesion, c and friction angel, φ for the case of 8% admixtures were 58.8kPa and 30.9° respectively, and 57.5kPa and 36.8°, respectively for the case of 16% admixture. On the whole, the cohesion, c increased with increase amounts of admixtures. On the contrary friction angle, φ reduced as more admixtures were added. This implies that more interactions among particles resulted in the increase of shear stress of soil mixtures".

A research on stabilizing soft fine-grained soils with fly ash was done by Tuncer B. E. et al. and published in 2006. "A laboratory study was conducted whereas soil–fly ash mixtures were prepared at different fly ash contents (10–30%) to evaluate how addition of fly ash can improve the CBR of wet and soft, fine-grained subgrade soils. Specimens were prepared at optimum water content, 7% wet of optimum water content (simulating the in situ condition in Wisconsin), and 9–18% wet of optimum water content (simulating a very wet condition). Based on the investigation, they have come into conclusion that CBR of soil–fly ash mixtures generally increases with fly ash content and decreases with increasing compaction water content. Adding 10 and 18% fly ash to fine-grained soils compacted 7% wet of optimum (the typical in situ condition) resulted in increases in CBR by a factor of 4 and 8, respectively. The CBR increased by a greater factor when fly ash was added to a wetter or more plastic (i.e., poorer) fine-grained soil".

Previous researches mainly concentrate in stabilizing soils by using fly ash, lime, microsilica or combination of additives. In fact, not many research on effect of Bottom Fuel Ash (BFA) on soft soils being carried out in the past. Nevertheless, there are some studies done to investigate the potential of BFA as the aggregate in road construction. BFA has been used as fine aggregate substitute in hot mix asphalt wearing surfaces, base courses, emulsified asphalt cold mix wearing surfaces and base courses. Because of the low durability nature of some bottom ash particles, bottom ash has been used more frequently in base courses than wearing surfaces. Apart from that, BFA has been used as concrete mixtures to improve workability and increase compressive strength of concrete. This porous surface structure also makes this material lighter than conventional aggregate and useful in lightweight concrete applications.

Bottom ash applications include its use as a:

i. Filler material for structural applications and embankments
ii. Aggregate in road bases, sub-bases, and pavement
iii. Feed stock in the production of cement
iv. Aggregate in lightweight concrete products
v. Snow and ice traction control material

For this research, it is intended to investigate the effect of BFA on shear strength of soft soils. Also, to check whether this stabilized soil can be used as subbase or subgrade material for construction of structures. The success of improving shear strength in this research can be evaluated from the improvement of soil's strength after series of laboratory experiments are conducted.

2.4 COMPACTION

The process of densifying, i.e, compacting soil is the oldest and most important method of soil stabilization. Compaction alone will often solve a particular soil problem and is usually the most economical of the techniques available. In addition to being used alone, compaction constitutes an essential part of a number of the other methods of stabilization. In compacting any particular soil, the moisture content, amount of compaction energy, and type of compaction can be varied. Compaction characteristics of the soil can also be varied by means of chemical additives. A considerable amount is known about the effect of moisture content and amount of compaction on the properties of the compacted soil. The most desirable combination of the placement variables depends on the particular soil and the particular set of properties desired. Benefits from compaction includes:

1. Densification, hence increase shear strength and stiffness, decrease compressibility of soil
2. Modify permeability
3. Reduce liquefaction potential
4. Control swelling and shrinking
5. Maintain material durability

2.5 SOIL CLASSIFICATION

Soil classification systems divide soils into groups and subgroups based on common engineering properties such as grain size distribution, liquid limit and plastic limit. There are several soil classification systems available. One of them, which is extensively being used is the Unified Soil Classification System (ASTM 2487 - 00).

2.6.1 Atterberg Limits (ASTM D 4318)

Atterberg Limits is used to determine whether the soil will act primarily as silt or clay, and whether it is considered highly plastic. The Plasticity Index is important in classifying fine-grained soils. It is fundamental to the Casagrande plasticity chart, which shows the relationship between Plasticity Index and Liquid Limit and provides information about the nature of cohesive soils.

2.6.2 Moisture Content Test (ASTM D 4643 – 00)

For many materials, water content is one of the most significant index properties used in establishing a correlation between soil behavior and its index properties. Water 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 particles.

2.6.3 Specific Gravity Test (ASTM 854 – 02)

The test covers the determination of specific gravity of soils that passes 4.75 mm sieve, by means of a water pyknometer. Specific gravity, G_s is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at 20°C.

2.6.4 Hydrometer Test (ASTM D 422 – 63)

Hydrometer analysis is based on the principle of sedimentation of soil particles in water. The soil is soaked in deflocculating agent, commonly solution of sodium hexametaphosphate for at least 16 hours. A hydrometer is placed in the cylinder to measure the specific gravity of the soil- water suspension. The sieve and hydrometer techniques may be combined for a soil having both coarse- grained and fine- grained soil constituents.

2.6.5 Compaction Test (ASTM D 698- 00)

Compaction test is used to determine the relationship between water content and dry unit weight of soils. Also, the dry density/ unit weight of a soil in the densest state of compactness that can be attained by using this procedure, which minimizes particle segregation and breakdown.

2.7 STRENGTH OF SOILS

Strength tests on soils can be done either by using unconfined compressive strength, vane shear or triaxial compression test. For this research, unconfined compressive strength test has been utilized to test the strength of soils.

2.7.1 Unconfined Compressive Strength Test (ASTM D 2166 -00)

The primary purpose of Unconfined Compressive Strength Test is to quickly obtain the approximate compressive strength of soils that possess sufficient cohesion to permit testing in the unconfined state. This test method provides an approximate value of the strength of cohesive soils in terms of total stresses.

CHAPTER 3

METHODOLOGY

Investigation the effect of bottom fuel ash on shear strength of soft soils requires proper techniques to ensure the accomplishment of the objective of the project. Two methods of problem- solving for this study are research- based approach and experimental/ analysis approach.

3.1 RESEARCH- BASED APPROACH

Research- based approach is the study of analytical, critical and objective review of written materials on the chosen topic and area. Published books and journals on soil stabilization were studied to enhance the knowledge and understanding of the topic. It provides the background information and identifies what other authors have said or discovered. Also, it contains all relevant theories, hypotheses, facts and data which are relevant to the objective and findings of the project.

3.2 EXPERIMENTAL/ ANALYSIS APPROACH

Results obtained through the various experiments using different testing machines and equipments were used as data in analyzing the shear strength of soil. The design of soil stabilization by additives consists of selecting the stabilizer, determining the method of application for soil testing and investigation of the optimum amount of stabilizer. These key properties are addressed by the following testing program:

3.2.1 Soil Classification

This process is used to screen the soil for identification of soft soil. The required information needed is based on common engineering properties such as grain size distribution, liquid limit and plastic limit. Soil classification can be done by using Unified Soil Classification System (ASTM 2487 - 00).

3.2.2 Soil strength Tests

Results obtained through a series of experiments using different percent of additive are used as data in analyzing the shear strength of soil. The samples were compacted first, before unconfined compressive strength tests were performed.

3.2.3 Optimum Percentage of Bottom Fuel Ash (BFA)

Different ratios (in percentage by weight) of bottom fuel ash, 0%, 2%, 4%, 5%, 10%, 15%, 20%, and 25% BFA are mixed with soft soil. The effects and trend of the different proportions of BFA on soft soils are studied to conclude the trend of changes in maximum dry density and optimum moisture content.

CHAPTER 4

RESULT AND DISCUSSION

4.1 PROPERTY TEST OF SOIL

The investigation on effect of Bottom Fuel Ash (BFA) on the shear strength of soft soils requires proper and complete laboratory testing for reliable results. Soil properties tests, namely moisture content, Atterberg limits, sieve analysis, hydrometer, specific gravity, compaction, organic content and chemical composition tests are crucial for soil classification. Also, gradation and chemical composition test for bottom fuel ash are of important to identify the physical and chemical properties of the additive. Most important, the main test, which is unconfined compressive strength test on soil with and without addition of BFA to study the effect of BFA as a stabilizer. The tests results are as follows:

4.1.1 Moisture Content Test

Water content is one of the most significant index properties used in establishing a correlation between soil behavior and its index properties. The soil tested was kept in microwave oven for at least 24 hours to remove its water content (moisture). After 24 hours, it is assumed that all water in the soil was removed and only dry soil is left. From the test, soil's average moisture content was **50.43 %**, with the range hover from **46** to **52%**. However, the soil tested maybe not at its natural condition. This is due to the experiment was handled one week after excavation from project site. Some moisture may have lost during the transportation from the site and there is possibility that some moisture loss during the storage. Therefore, it is anticipated that the natural moisture content is in between **55** to **60%**, or a little bit higher.

The soil is found to behave like **soft clay**. It contains high water and therefore high compressibility, which is of course, posses weak engineering properties. The typical moisture content for some soils and results of moisture content of soil are presented in Table A1 and A2 respectively in Appendix A.

4.1.2 Atterberg Limits Test

4.1.2.1 Liquid Limit Test

The Liquid Limit (LL) is the moisture content in percent, at which the soil changes from a liquid to a plastic state. By using cone penetration method, the liquid limits obtained for sample 1, 2 and 3 were **44.4%**, **45.4%** and **45.1%** respectively. Therefore, the average, which represents the liquid limit of soil was **45%**. The soil can be classified in group of **intermediate plasticity** ($35 \leq W_{LL} \leq 50$), based on Table A5 in Appendix A. The relationship of moisture content and cone penetration is presented in Figure 4.1 below. The results of the tests are as presented in Table A3 & A4 in Appendix A.

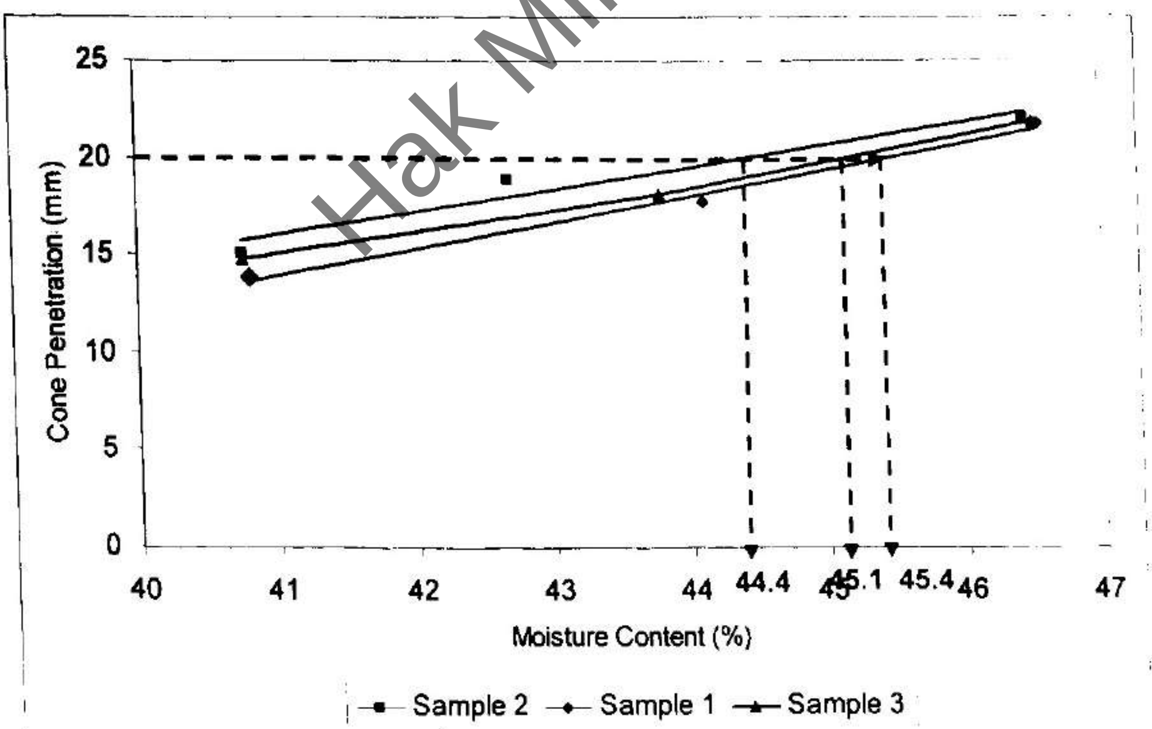

Figure 4.1 : Relationship between moisture content and cone penetration

4.1.2.2 Plastic Limit Test

The Plastic Limit (PL) is the water content, in percent, at which the soil changes from a plastic to a semisolid state. The soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling. From the test conducted, the moisture content for sample 1, 2 and 3 were **28.18%**, **28.39%** and **28.12%** respectively. Based on the three samples, the average, which represents the liquid limit was **28%**. The result of plastic limit test is presented in Table A6 in Appendix A.

The Plasticity Index (I_p) is the difference between liquid limit and plastic limit of a soil. From the tests conducted, the plasticity index was **17**. The soil can be classified to have **medium plasticity** ($10 \leq I_p \leq 20$) based on Table A7 in Appendix A.

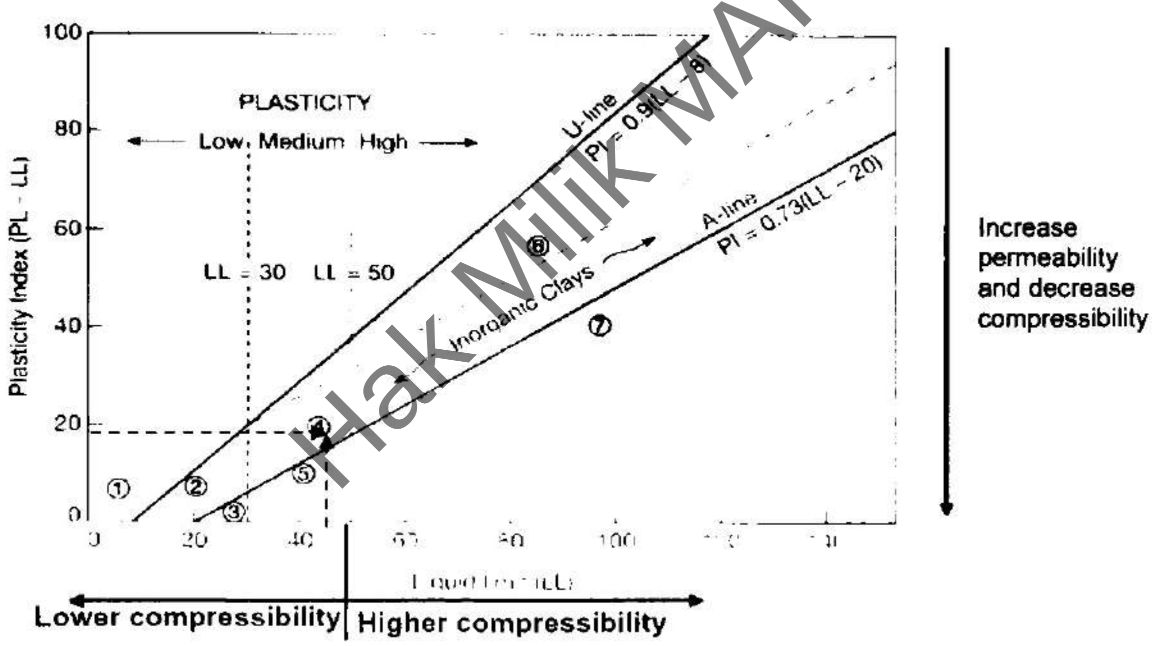

Figure 4.2 : Plasticity Index chart

For Plasticity Index of 17 and Liquid Limit of 45, therefore the soil falls under **Inorganic clay of medium plasticity** (Figure 4.2).

4.1.3 Specific Gravity Test

Specific gravity, G_s is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at 20°C (±2 °C). The specific gravity of soil solids is used to calculate the density of the soil solids by multiplying its specific gravity by the density of water at specific temperature. From Table A8 in Appendix A, the average specific gravity of soil tested were **3.14**, **2.71** and **2.34 g/cm^3** for three samples. Supposedly, the specific gravity of soil hovers around **2.65** to **2.75 g/cm^3**, based on 0% air void line in compaction test. From the compaction test, it is possible to check weather the value of specific gravity obtained is correct or not. The 0% air void line is parallel to, and should not intercept with dry density vs. moisture content curve. During the execution of lab works, careful attention should be made to ensure that no leakage of water from the pyknometers and no bubbles and froths trapped in the water.

4.1.4 Compaction Test

Mechanical compaction is one of the most common and cost effective means of stabilizing soils. Compaction test is used to determine the relationship between water content and dry unit weight of soils. The purpose of compaction of raw soils is to determine its optimum water content. The optimum water content is the water content that results in the greatest density for a specified compaction. Also, to verify weather the specific gravity tested previously is correct. It can be done by plotting 0, 10 and 20% air void lines on the same graph with compaction curve. Should the 0% air void line is above and not intersect with compaction curve to prove that the specific gravity is within the acceptable range.

From Figure 4.3, the maximum dry density of soil was **16.2 kN/m^3**, corresponding to its optimum moisture content of **20.5 %**. The value for both maximum dry density and optimum moisture content for sample 1 and 2 are almost similar and identical, whereas for sample 2, the maximum dry density of soil was **16.0 kN/m^3**, corresponding to its optimum moisture content of **20.0%**. Also shown in the figure are 0% air void (100% saturation) line, 10% air void (90% saturation) line and 20% air void (80% saturation) line with the assumption that the specific gravity, G_s of soil is **2.71** g/cm^3. The results of compaction test are as shown in Table A9 and A10 in Appendix A. Meanwhile, Table A11 shown the value of air void, corresponding to its moisture content

Raw soils were compacted in average internal diameter of 104.85 mm mould with a 24.4N rammer for 3 layers, 27 times of blow per layer. Mass of mould and base plate was 6.38 kg, with the average height of mould equal to 115.5 mm. The volume of mould was therefore 997.26 cm^3.

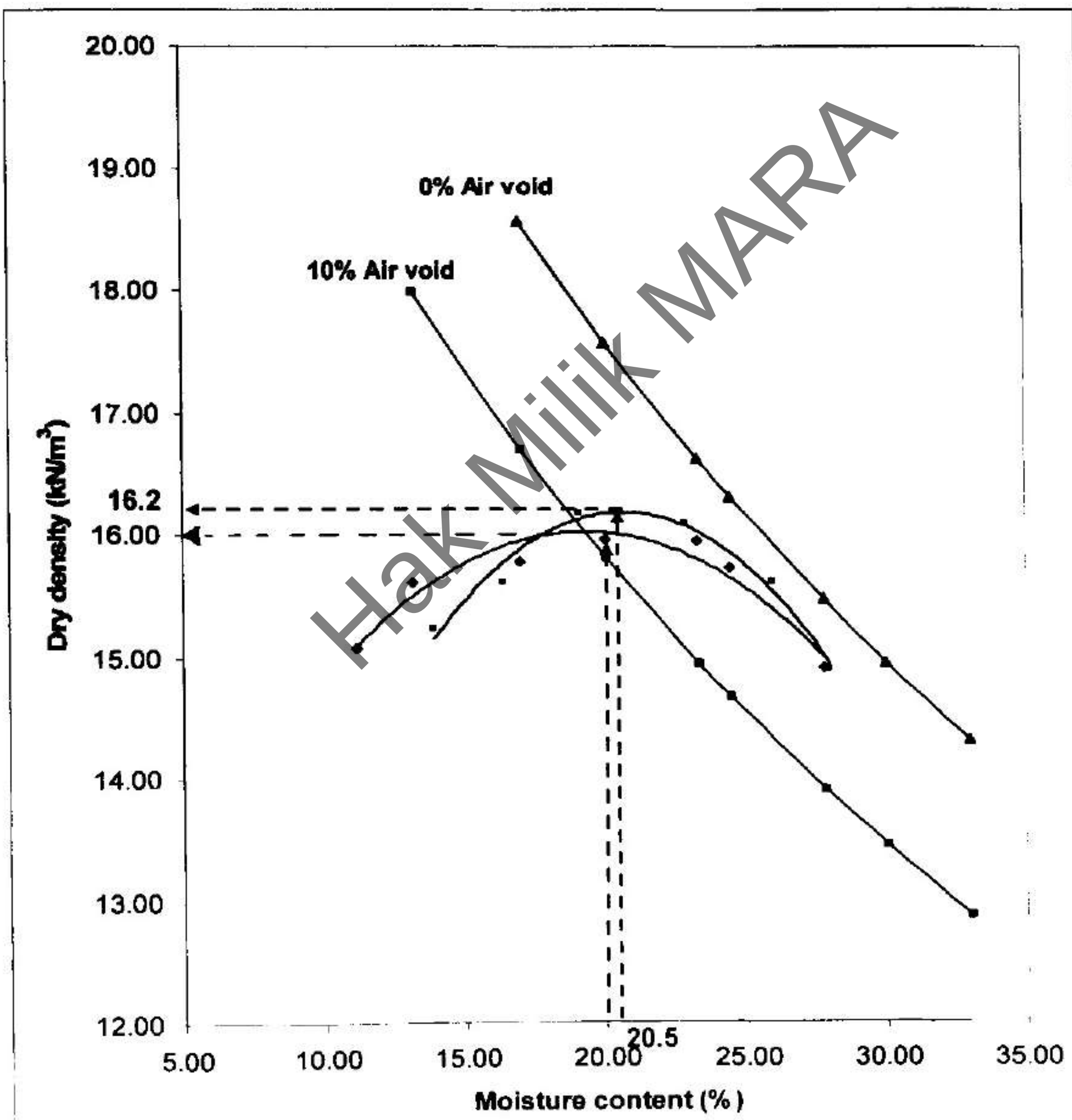

Figure 4.3 : Variation of dry densities with moisture contents

are as shown in Table A9 and A10 in Appendix A. Meanwhile, Table A11 shown the value of air void, corresponding to its moisture content

Raw soils were compacted in average internal diameter of 104.85 mm mould with a 24.4N rammer for 3 layers, 27 times of blow per layer. Mass of mould and base plate was 6.38 kg, with the average height of mould equal to 115.5 mm. The volume of mould was therefore 997.26 cm^3.

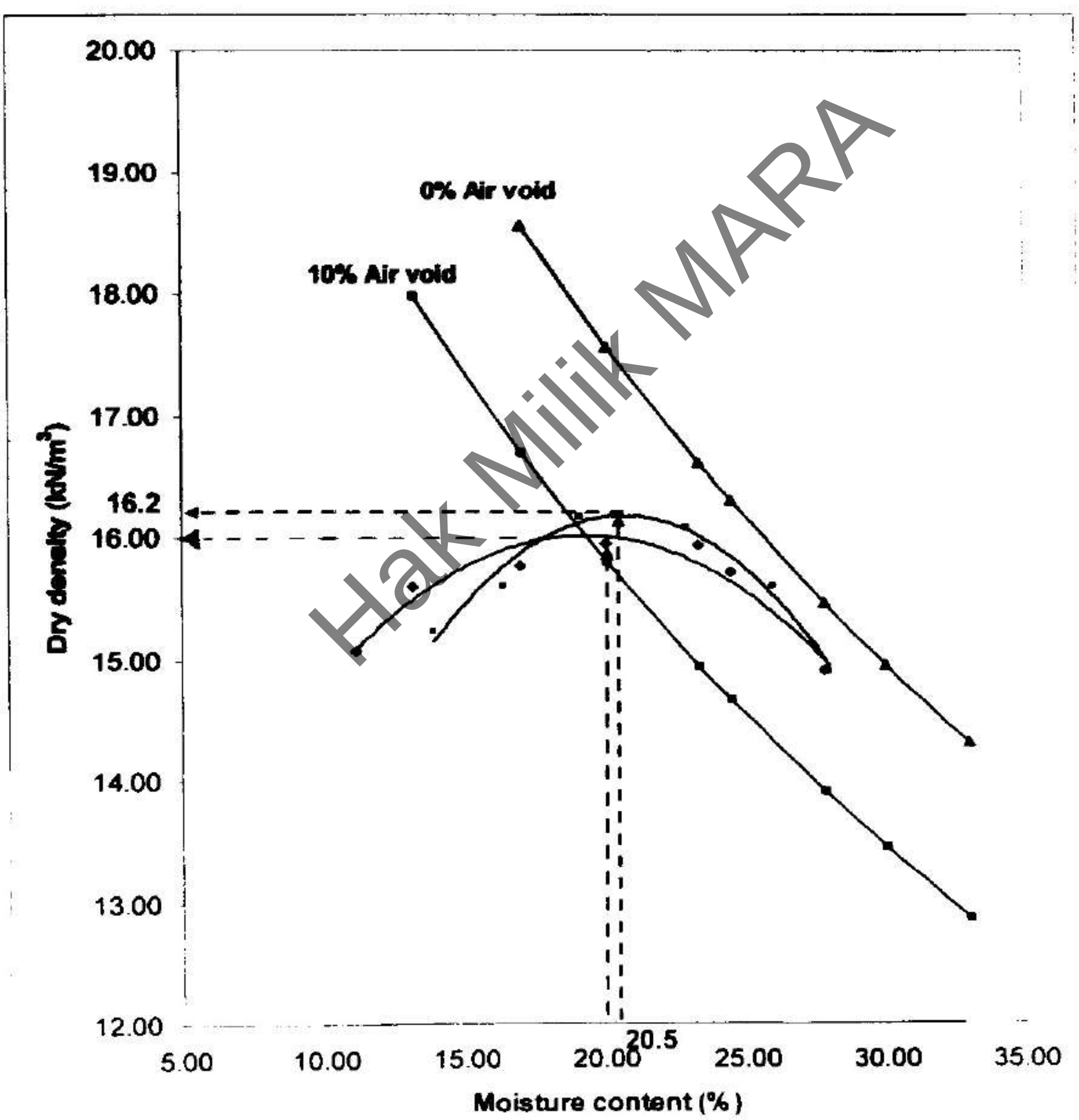

Figure 4.3 : Variation of dry densities with moisture contents

4.1.5 Organic Matter Content Test

Ash content of organic soil sample is determined by igniting the oven- dried sample from the moisture content determination in a muffle furnace at 440°C. The percent of organic matter is important in classifying an organic soil.

The organic matter for sample 1, 2, 3, 4, 5 and 6 were **5.04**, **5.81**, **5.13**, **4.73**, **9.68** and **10.5%** respectively, as presented in Table A12 in Appendix A. it is anticipated that the organic matter of the soil is in the range of **4.5** to **11%**, based on the test results. The organic matter in foil 4 is slightly lower than the other three samples may be due to the sample was taken at the spot which contains less humus and organic matter. Meanwhile, organic matter contained in foil 5 and 6 are much higher than the others may be due to the samples were taken at the spot that contain a whole lot more organic matter and humus than the other sample of soils were taken. It can be seen whereas a lot of black in color particles, most likely plant's roots and dead barks are mixed in the soils. This variation of organic content shows the variation of distribution of organic matter in the soil taken.

4.1.6 Chemical Composition

The chemical composition test for soil has been carried by using X- Ray Fluorescence (XRF) test. The XRF method is widely used to measure the elemental composition in weight percentage of materials. This method is fast and non-destructive to the sample. From the test result in Table A13 in Appendix A, it can be seen that Silica Oxide, SiO_2 (55.1%) is the biggest composition among all composition, followed by Aluminium Oxide, Al_2O_3 (36.7%). Pottasium Oxide, K_2O contained is 3.15% and Ferum Oxide, Fe_2O_3 is 2.62%. Meanwhile, the other compositions are lower than 1%.

4.2 BOTTOM FUEL ASH PROPERTIES TESTS

4.2.1 Gradation Test (Sieving Analysis)

Particle size analysis (sieving) gives the quantitative determination of the distribution of particle sizes by screening a known weight of the soil through a stack of sieves with different openings. The sievers are arranged in sequence from biggest to smallest sieve openings (3.35 mm, 2 mm, 1.18 mm, 600 μm, 425 μm, 300 μm, 212 μm, 150 μm, 63 μm), top to bottom, and followed by a pan.

Particle size distribution curve for sample 1 and 2 of bottom fuel ash are plotted on semilogarithmic graph as shown in Figure 4.4 below. From the curve, particle diameter, corresponding to its percentage finer, D_{10}, D_{30} and D_{60} are 0.06, 0.18 and 0.35 respectively. Two parameters can be determined from particle- size distribution curves of coarse-grained particles are the uniformity coefficient, C_u, and coefficient of gradation or coefficient of curvature, C_c which are **5.8** and **1.54** respectively.

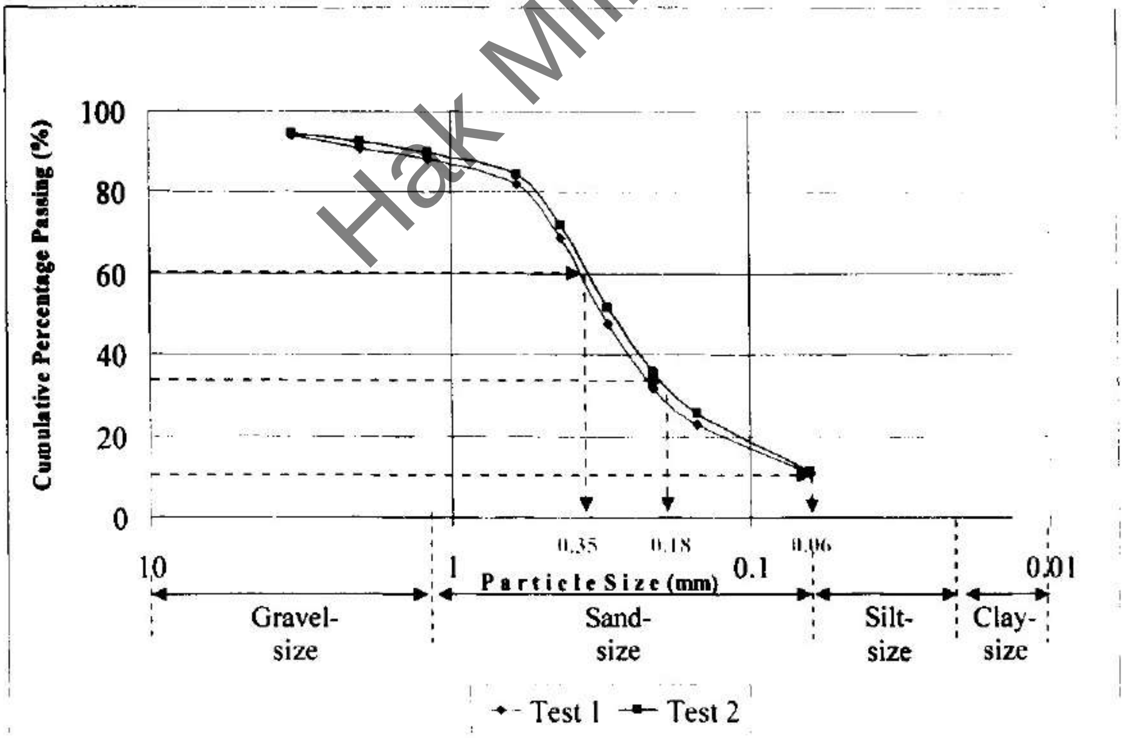

Figure 4.4 : Particle size distribution of bottom fuel ash

From the result in Table B1 and B2 in Appendix B, it can be observed that the average of gravel- size, sand- size, silt-size particles are **5.70, 83.29** and **11.01%** respectively. The summary of the result is as Table B3 in Appendix B. The bottom fuel ash particles can be classified as **well graded sand- like particles**, base on Unified Soil Classification System (ASTM, 2000).

4.2.2 Chemical Composition

The chemical composition test for bottom fuel ash has been carried by using X- Ray Fluorescence (XRF) test. The XRF method is widely used to measure the elemental composition in weight percentage of materials. This method is fast and non-destructive to the sample. From the test result in Table B4 in Appendix B, it can be seen that Silica Oxide, SiO_2 (57.1%) is the biggest composition among all composition. Silica Oxide acts as filler and binder to increase the strength of soils. Meanwhile, composition of Calcium Oxide, CaO is low, which is only 6.39%. Therefore, it is anticipated that the sample tested if of Class- F ash, which contains pozzolanic material properties and has the potential application to stabilize soft subgrade soil. Basically, there are two types of ash, which are class- F and class- C. Class- C ash contains lesser Silica, Aluminium and Ferum elements and contains cementation mixtures.

Apart from the test, chemical composition of ash provided by Tenaga Nasional Berhad (TNB) power plant, where the bottom fuel ash is obtained is a summarized in Table B5 and B6 in Appendix B.

4.3 STRENGTH TEST OF SOILS

4.3.1 Unconfined Compressive Strength Test

The purpose of Unconfined Compressive Strength Test is to obtain the approximate compressive strength of soils that possess sufficient cohesion to permit testing in the unconfined state. This test method provides an approximate value of the strength of cohesive soils in terms of total stresses. The relationship between axial stress and axial strain is shown in the Figure 4.5 as herein below:

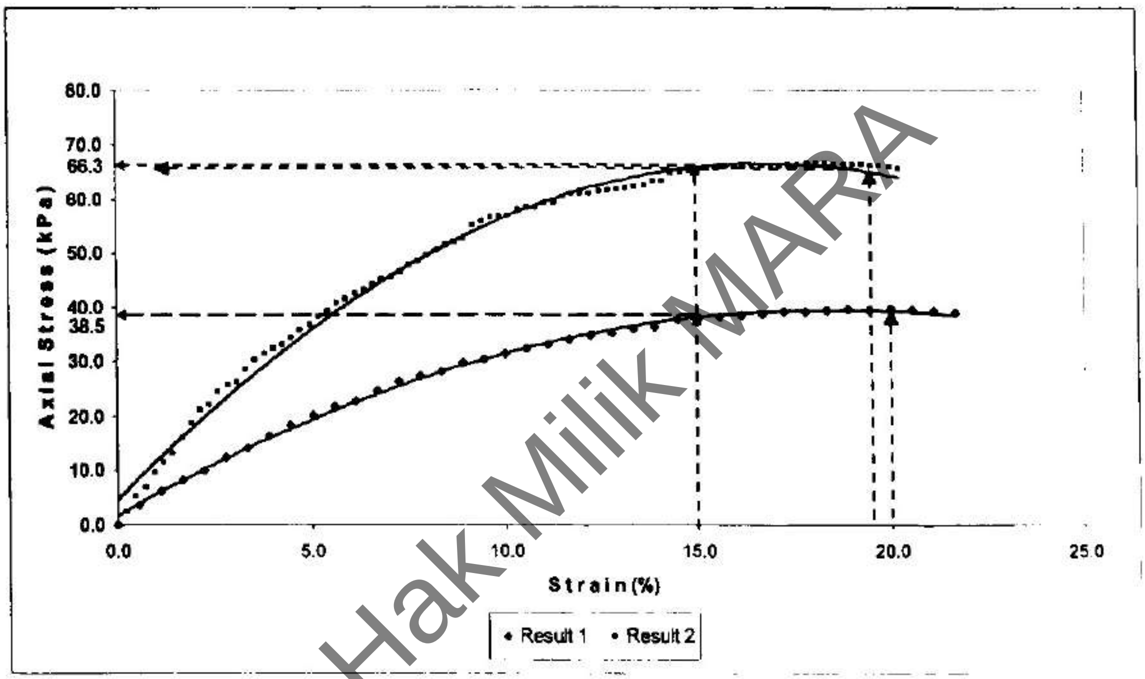

Figure 4.5 : Relationship Between Axial Stress and Strain for Undisturbed Sample

For sample 1, the value of unconfined compressive strength, q_u at 15% strain was 38.5kN/m^2 at moisture content of 52.5%. The maximum strength was 40kN/m^2 at maximum strain of 20%. However, ASTM D 2166 indicated that should the maximum value of compressive stress, or the compressive strength at 15% axial strain is selected, whichever attained first. Therefore, the compressive stress at which an unconfined soil fails was **38.5kN/m^2**. Meanwhile, shear strength, s_u for unconfined compressive strength of soil is ½ of the compressive stress at failure. Therefore, the shear strength of soil tested was **19.3kN/m^2**.

For sample 2, at moisture content of 37.5%, the value of unconfined compressive strength, q_u at 15% strain was 66.3kN/m^2. Meanwhile, the maximum strength was also 66.3kN/m^2 at maximum strain of 20%. Should the compressive strength at 15% axial strain is selected, as it is achieved first. Therefore, the compressive stress at which an unconfined soil fails was **66.3kN/m^2**. Meanwhile, shear strength, s_u for unconfined compressive strength of soil is ½ of the compressive stress at failure. Hence, the shear strength of soil tested was **33.2kN/m^2**. For both two tests conducted, it can be concluded that the higher moisture content of soils, the lower the compressive strength, thus lower the shear strength of that particular soils.

For unconfined compressive strength test, the cylinder specimen shall have a minimum diameter of 30mm and the largest particle contained within the test specimen shall be smaller than one tenth of the specimen diameter. The high-to-diameter ratio shall be between 2 and 2.5. Take a minimum of three height measurements (120° apart), and at least three diameter measurements at the quarter points of the height. The description of soil samples are presented in Table C1 and C2, meanwhile the results of Unconfined Compressive Strength tests are as Table C3 & C4 in Appendix C.

4.4 ADDITIONAL TEST

4.4.1 Wet sieve

This test was performed to find out how many percentage of sand contained in soils for unconfined compressive strength test. A known weight of dried soils was placed on 63μm sieve and allows flowing tap water run on it. The dry weight of remaining sample was weight, which represent the amount of sand contained in the soils. The test provided that the percentage of sand was 7.34%. This amount is a little bit high, but still acceptable for unconfined compressive strength test. If too high percentage of sand contained in the soils, it means that it requires more compressive force for soils to fail because of the sand, and the result is not precise as it should be. The test result is as presented in Table C7 in Appendix C.

4.5 MAIN TESTS

4.5.1 Compaction Tests for Soils with Additives

Compaction test is used to determine the relationship between water content and dry unit weight of soils. Moisture-density relationship was measured via appropriate compaction test, Standard Proctor test. The tests are performed on all the nine soils at 0%, 2%, 4%, 5%, 10%, 15%, 20%, 25% and 30% bottom ash content. Moisture content of air-dried soils is obtained before performing the compaction tests. This starting moisture content is used for calculating the amount of water to be added to each compaction sample. The desired amount of BFA, measured as percent of dry soil by weight, is then mixed thoroughly to produce a homogeneous soil-bottom ash blend. Wetting of the sample soil is accomplished by spreading the soil-fly ash blend in a fairly large pan. The calculated amount of water is weighed and added very evenly over the surface. All compactions are commence immediately after the addition of water to the blend and completed within 1 hours.

The investigation of the pozzolanic reaction of bottom fuel ash in soil provided that the maximum dry density for **soil + 0% BFA, soil + 2% BFA, soil + 4% BFA, soil + 5% BFA, soil + 10% BFA, soil + 15% BFA, soil + 20% BFA, soil + 25% BFA** and **soil +30% BFA** were **16.18, 16.05, 15.95, 15.90, 15.40, 14.68, 14.45, 13.95** and **13.85 kN/m^3** respectively and optimum moisture content were **20.50, 21.00, 21.50, 21.50, 23.00, 26.0, 26.0, 28.0** and **27.50%** respectively. The curves of dry densities vs. moisture contents are as illustrated in Figure 4.6 on the next page. From the curves, it can be concluded that up to 25% of bottom ash, the optimum moisture contents were increased. However, the moisture content was slightly decreased with the addition of 30% BFA. Meanwhile, the maximum dry densities were decreased all the way from 0% through 30% upon addition of BFA. It is mostly due to specific gravity of BFA is much lesser, which is 1.56 g/cm^3, as compared to soils, which is 2.71 g/cm^3.

Figure 4.6 shows the effects of compaction on soil structure. Cernica, 1995 verified that a possible explanation for the change in structure is tied to the "change in electrolyte concentration. At small water content such as at dry-of-optimum, the concentration of electrolytes is relatively high; this impedes the diffuse double layer of irons surrounding each clay particle from full development. The result is low interparticle repulsion and subsequent flocculation of the colloids and thereby a lack of significant particle orientation of the compacted clay. On the other hand, the water content is increased, to a point of wet-of-optimum, the electrolyte concentration is reduced, and there is an increase in repulsion between clay particles, a reduction of flocculation, and thus an increase in particle orientation."

For the compaction of soils with the addition of additive, the samples were compacted in average internal diameter of 104.66 mm mould with a 24.4 N rammer for 3 layers, 27 blows per layer. Mass of mould and base plate was 5.08 kg, with the average height of mould equal to 115.79 mm. The volume of mould was therefore 996.14 cm^3. The results were presented in Table D1 to D13 and summarized in Table D14 in Appendix D.

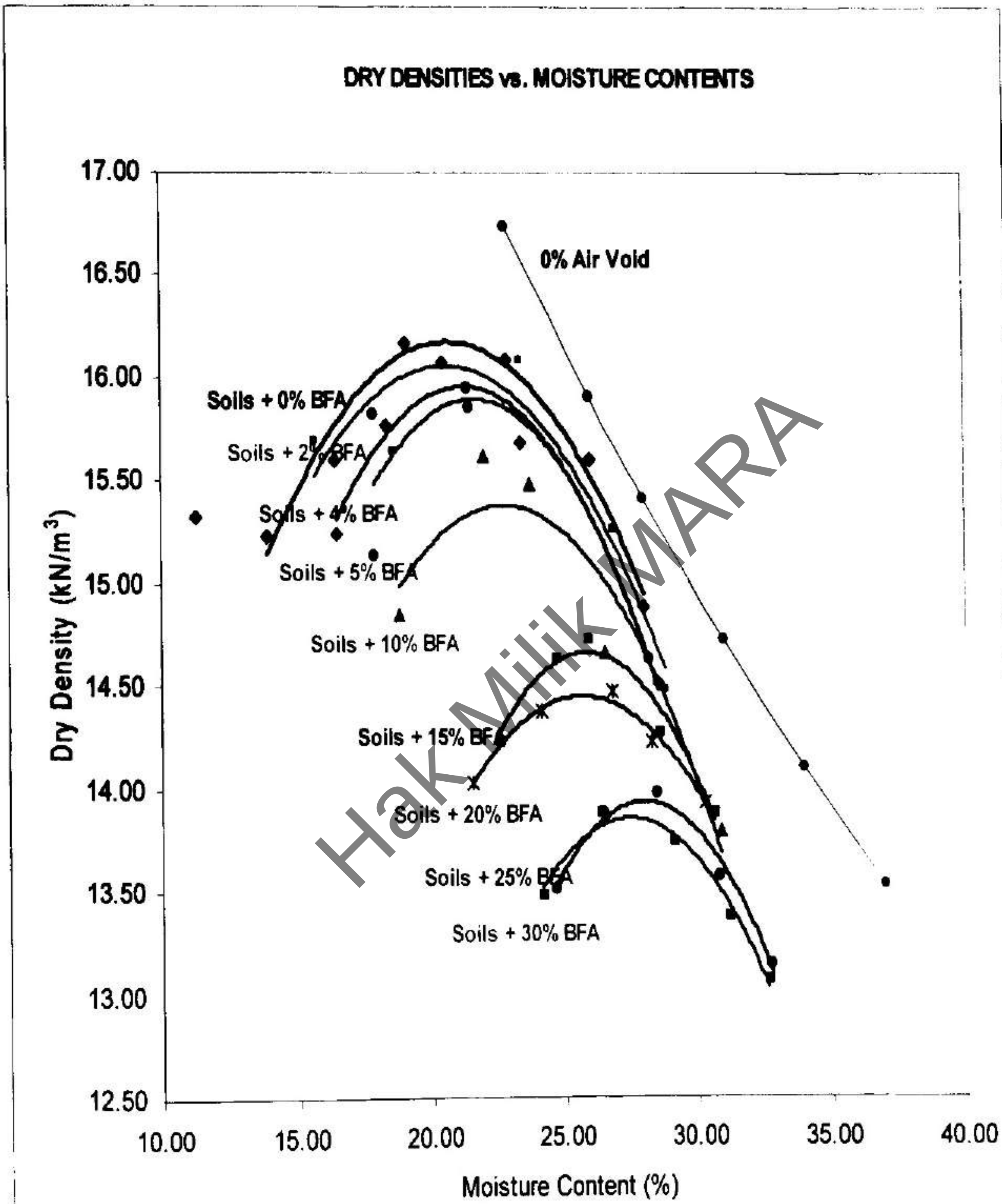

Figure 4.6 : Compaction curves for soils with different proportions of additive

4.5.2 Unconfined Compressive Strength Test

The unconfined compressive strength is often taken to be the parameter by which the mechanical characteristics of a subgrade are judged. The purpose of unconfined compressive strength test is to obtain the approximate compressive strength of soils that possess sufficient cohesion to permit testing in the unconfined state. This test method provides an approximate value of the strength of cohesive soils in terms of total stresses. The test is performed on compacted soil-fly ash blend sample. The unconfined compressive strength is obtained at the highest axial stress point from stress-strain curve.

The unconfined compressive strength was tested immediately after compaction and also after one day curing. The samples were wrapped in a plastic and cured in a vacuum dessicator to avoid moisture lost and to keep the moisture at its optimum, the point of greatest density. For immediate test, it was found out that the axial stress for **soil + 0% BFA, soil + 2% BFA, soil + 4% BFA, soil + 5% BFA, soil + 10% BFA, soil + 15% BFA, soil + 20% BFA** and **soil + 25% BFA** were **1153.30, 947.50, 1000.00, 722.50, 482.50, 214.00, 263.30** and **171.70 kN/m^2** respectively, thus gave shear strength of 576.7, **473.8, 500.0, 361.3, 241.3, 107.0, 131.7** and **85.9 kN/m^2** respectively. Meanwhile, for one day curing, the test results provided that the axial stress for **soil + 2% BFA, soil + 4% BFA, soil + 5% BFA, soil + 10% BFA, soil + 15% BFA, soil + 20% BFA** and **soil + 25% BFA** were **1035.00, 925.00, 702.50, 327.00, 471.00, 334.70** and **292.50 kN/m^2** respectively, which gave shear strength of **517.50, 462.50, 351.25, 163.50, 235.50, 167.35** and **146.25 kN/m^2** respectively. Shear strength, s_u for unconfined compressive strength of soil is ½ of the compressive stress at failure. For immediate test, and one day curing samples, the results showed that raw soils (soil + 0% BFA) have the greatest shear strength, as compared to soils added with BFA. For the case of immediate test, the trend of shear strength is quite wavering. The shear strength decreased as 2% of BFA was added to the soils, and slightly increased when 4% of BFA was added. Then it decreased with addition of BFA up to 15%. At 20% BFA, shear strength rose up and again, decreased at 25% BFA. However, the trend can be clearly observed when the shear strengths were plotted against percentage of BFA at its best fit curve as illustrated in Figure 4.8. From the curve, it can be concluded that the shear strength was decreased as more BFA was added to soils to 20%, and then it rose a little bit higher at 25% addition of BFA.

Shear strength test, prepared after one day curing showed better trend, whereas the shear strength decreased as more BFA was added to the soils, up to 15%, and then it slightly increased at 20% BFA, and decreased at 25% BFA. Shear strength versus percentage of BFA was plotted in best fit curve as illustrated in Figure 4.8, which clearly showed that the shear strengths were decreased with more addition of BFA, up to 20% and increasedsed at 25% BFA.

Samples with 2%, 15%, 20% and 25% BFA placed under one day curing demonstrated higher shear strength, as compared to samples tested immediately. However, samples with 4%, 5% and 10% BFA samples under one day curing provided lower shear strength as compared to samples tested immediately after the compaction. Rationally, samples tested after one day curing should give greater shear strength, as compared to samples tested immediately after compaction. It is due to time allowance for pozzolanic reaction to occur.

The relationship between axial stress and axial strain for raw soils tested immediately is presented in Figure 4.7. Meanwhile, the rest are presented in Figure E1 trough E7 (immediate test) in Appendix E and Figure F1 through F7 (one day curing) in Appendix F. Bottom ash content has a significant influence on the strengths of soil-bottom ash blends. The relationship between shear strength and percentage of BFA was presented in Figure 4.8.

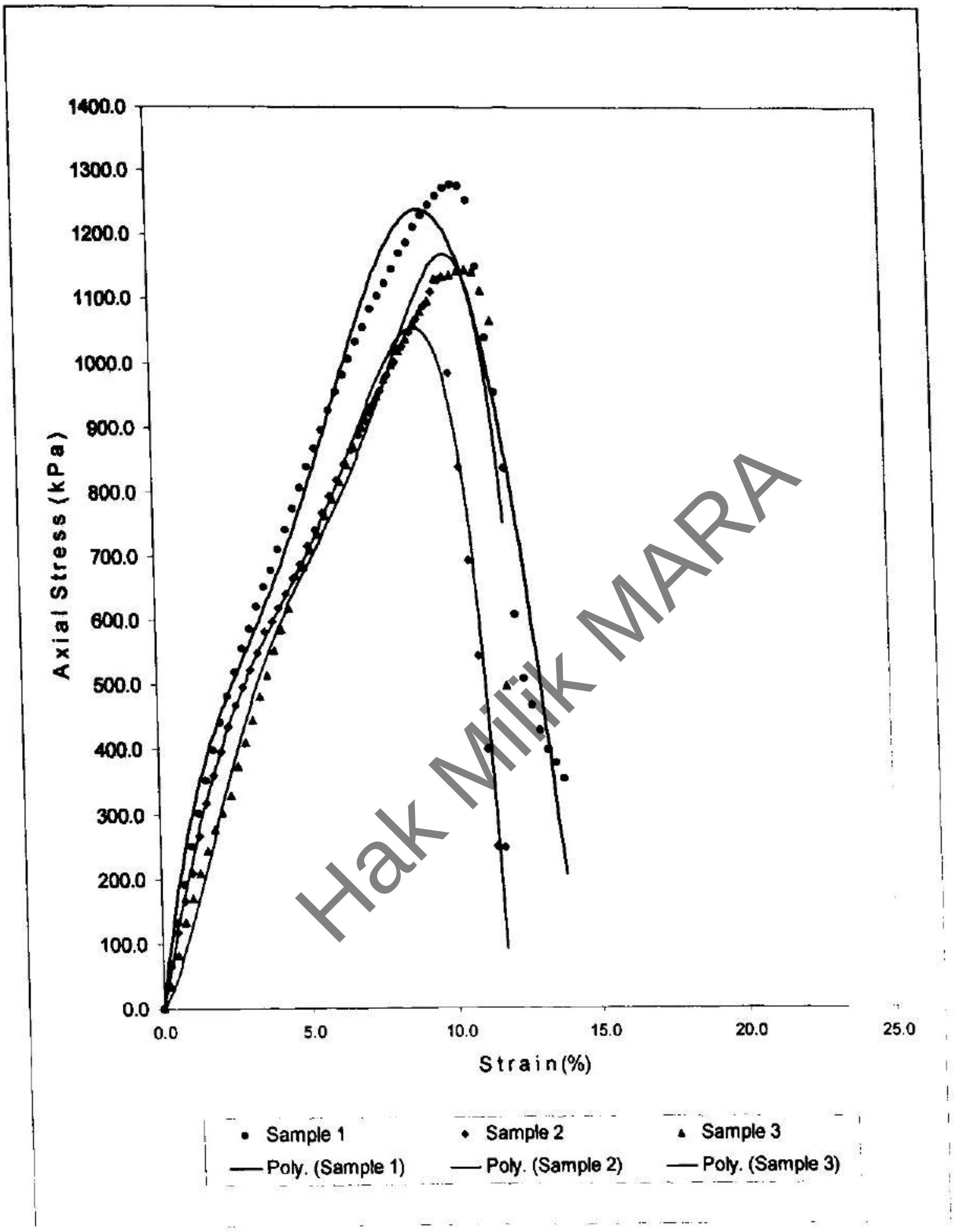

Figure 4.7 : Graph of axial stress vs. strain for raw soils (immediate test)

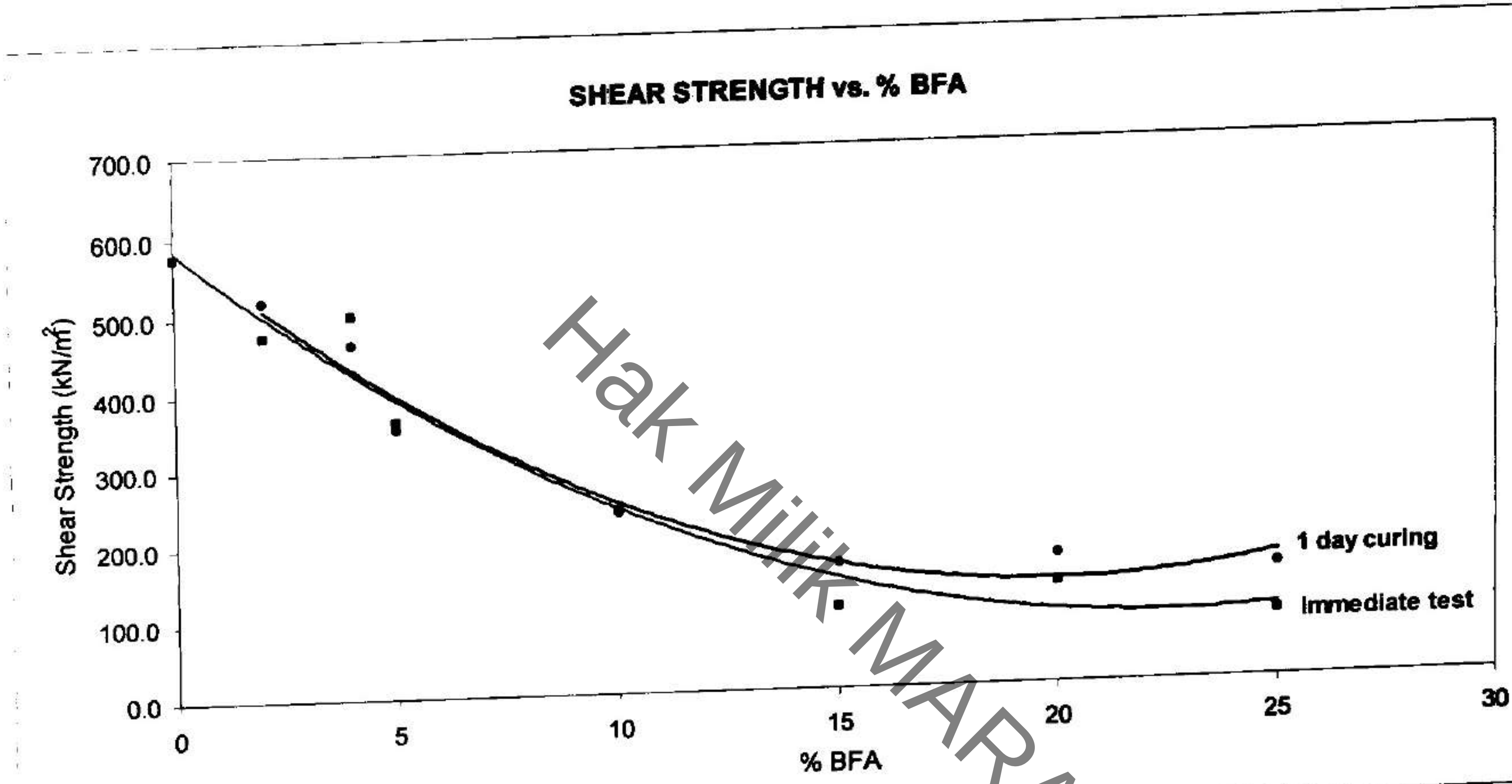

Figure 4.8 : Shear strength of soils with different proportions of bottom fuel ash

CHAPTER 5

CONCLUSIONS

Two compaction tests for raw soils and eight tests for soils with addition of 2, 4, 5, 10, 15, 20, 25 and 30% Bottom Fuel Ash (BFA) were carried out. Upon addition of BFA, the compaction characteristics were improved. The maximum dry densities decreased and optimum moisture contents increased with an increase in percent of bottom fuel ash.

For unconfined compressive strength test, eight sets, two tests per set – immediate testing and one day curing were done to study the effect of different proportion of BFA with soils. The test results provided that raw soils gave highest shear strength, as compared to soils added with proportions of BFA. For immediate test, the shear strength was decreased as more BFA was added to soils to 20%, and then it rose a little bit higher when 25% of BFA was added. The samples prepared after one day curing resulted almost similar trend to immediate test samples, whereas the shear strengths were also decreased with more addition of BFA, up to 20% and increased at 25% BFA. The trend clearly tells that addition of BFA up to 20% does not improve the shear strength of treated soil. But the improvement can be seen at 25% BFA. Maybe, the development will be more significant with the addition of more percentage of BFA. But somehow, it is not effective to utilize more BFA compared to composition of soil for soil stabilization.

Samples with 2, 15, 20 and 25% BFA placed under one day curing demonstrated higher shear strength, as compared to samples tested immediately after the compaction, at the same percentage. However, samples with 4, 5 and 10% BFA samples under one day curing gives lower shear strength as compared to samples tested immediately. Should samples prepared under one day curing exhibited higher shear strength, weigh against samples tested immediately after the compaction. This unconsistency results may due to different time needed for different percentage of BFA to react. For one day curing, samples were wrapped in plastic, and placed in a vacuum dessicator to provide some time for pozzolanic reaction to occur, as well as to avoid moisture lost.

The test results obtained shown an adverse impact on treated soils and do not tally with the literature reviews, which say that the shear strength of soft soils will increase as more BFA was added. In fact, the results demonstrated that raw soils, with no BFA added possess greatest shear strength.

Existing research has shown that Class F fly ash can be effectively used in stabilizing pavement subgrades and soils with poor bearing capacities (Vishwanathan et al.1997; Qubain et al. 2000; Acosta 2002). Moreover, Misra (2000) evaluated the Utilization of Western Coal Fly Ash in Construction of Highways in the Midwest. She strongly believed that the unconfined compressive strength shows a strong dependence on moisture content. The strength increases up to a certain moisture content and then decreases, such that an optimum moisture content, corresponding to the maximum unconfined compressive strength for the soil-fly ash blend, may be defined. Nicholson and Kashyap (1993) evaluated the effect of fly ash on the engineering properties of tropical soils from Hawaii. The addition of fly ash decreased the liquid limit and plasticity index, and increased the unconfined compressive strength. Even though they were using fly ash as the additive, but x-ray fluorescence proved that the chemical composition of bottom and fly ash are almost the same. It is just the size of bottom ash is very much greater than fly ash. The same goes to Jaturapitakkul and Cheerarot (2003). They came into conclusion that the addition of bottom ash into concrete developed higher compressive strength than that of the concrete without bottom ash. As a result of compressive strength, it was concluded that ground bottom ash could be used as a good pozzolanic material.

The possible explanations to the contradiction are that maybe the time allowance for samples tested immediately and one day curing after the compaction was not sufficient for pozzolanic reaction to occur. Since the effectiveness of BFA react with time, perhaps, should the samples were tested on 3, 7, 14, 21 and 28 days to observe the performance of the samples, just like what they do to concrete. Or, maybe the soils are not suitable for soil stabilization by using bottom fuel ash. However, more thorough analysis and lab tests must be conducted to investigate the potential of BFA for soil stabilization.

CHAPTER 6

RECOMMENDATIONDS

The test results showed an adverse impact on treated soils. It is most probably due to the pozzolanic reaction of BFA is quite slow, because of bigger size particles of BFA. Also, for the reason of low composition of Calcium Hydroxide, CaO, as well as class of BFA itself. Therefore, for improvement of this research, some recommendations are listed as herein below:

Even though BFA do not give an improvement to treated soils, it does not mean that BFA is not a good pozzolan at all. Maybe, it needs some activator, such as lime and cement, mixed with BFA to give positive impact on treated soils at faster rate and more effective way.

Also, effectiveness of BFA react with time, perhaps, should the samples are given more time to allow sufficient time for pozzolanic reaction to occur. Therefore, should the samples are tested on 3, 7, 14, 21 and 28 days and observe the performance of the samples.

More important, to utilize class C BFA, which contains more percentage of calcium and the silica, alumina and iron as compared to class F of BFA. In addition of having pozzolanic properties, Class C BFA, which contains significant amount of Calcium Hydroxide, CaO or lime also has some cementitios properties (ASTM C 618-03).

One of another crucial parameter to weigh the quality of BFA is the physical properties of BFA itself. It is strongly recommended to grind the BFA into much finer particles, until the particle size retained on sieve 325 was less than 5% by weight, for instance. The particle of bottom ash was large, porous and irregular shapes. The grinding process will reduce the particle size as well as porosity of the bottom ash.

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APPENDICES

APPENDIX A - SOILS

MOISTURE CONTENT TEST

Table A1 : Typical Moisture Content for Some Soils

Type of Soil	Natural Moisture Content (%)
Loose uniform sand	30
Dense uniform sand	16
Loose angular- grained silty sand	25
Dense angular- grained silty sand	15
Stiff clay	21
Soft clay	30 - 50
Loess	25
Soft organic clay	90 - 120
Glacial till	10

Table A2 : Result of moisture content test

Container No.	1	2	3	4	5	6	7	8	9	10
Mass of wet soil + container (g)	100.68	101.12	71.76	93.38	121.8	112.50	85.59	121.86	93.56	91.96
Mass of dry soil + container (g)	77.84	77.85	57.16	73.02	90.1	84.40	66.79	89.23	71.90	70.71
Mass of container (g)	29.17	29.19	29.20	29.59	29.5	29.20	29.67	29.35	29.26	29.19
Mass of moisture (g)	22.84	23.27	14.60	20.36	31.7	28.10	18.8	32.63	21.66	21.25
Mass of dry soil (g)	48.67	48.66	27.96	43.43	60.6	55.20	37.12	59.88	42.64	41.52
Moisture content (%)	**46.93**	**47.82**	**52.22**	**46.89**	**52.30**	**51.00**	**50.65**	**54.49**	**50.80**	**51.18**
Average moisture content (%)	**50.43**									

ERBERG LIMITS TEST

id Limit Test

le A3 : Result for liquid limit test (sample 1)

Test Method : BS 1337						
Test No.	1		2		3	
Initial dial gauge reading (mm)	0	0	0	0	0	0
Final dial gauge reading (mm)	13.60	14.10	17.90	17.70	21.70	21.80
Average penetration (mm)	13.85		17.80		21.75	
Mass of wet soil + container (g)	50.88	48.17	48.48	57.59	86.83	81.73
Mass of dry soil + container (g)	44.62	42.68	42.63	48.93	68.59	67.75
Mass of container (g)	29.22	29.28	29.39	29.26	29.65	37.53
Mass of moisture (g)	6.26	5.49	5.85	8.66	18.24	13.98
Mass of dry soil (g)	15.40	13.40	13.24	19.67	38.94	30.22
Moisture content (%)	40.65	40.97	44.18	44.03	46.84	46.26
Average moisture content (%)	40.81		44.11		46.55	

Table A4 : Result for liquid limit test (sample 2)

Test Method : BS 1337						
Test No.	1		2		3	
Initial dial gauge reading (mm)	0	0	0	0	0	0
Final dial gauge reading (mm)	15.40	14.90	19.20	18.70	22.00	22.10
Average penetration (mm)	15.15		18.95		22.05	
Mass of wet soil + container (g)	48.27	44.94	75.29	73.01	65.13	65.34
Mass of dry soil + container (g)	42.79	40.38	61.50	59.98	53.84	53.88
Mass of container (g)	29.27	29.25	29.18	29.48	29.49	29.25
Mass of moisture (g)	5.48	4.56	13.79	13.03	11.29	11.46
Mass of dry soil (g)	13.52	11.13	32.32	30.50	24.35	24.63
Moisture content (%)	40.53	40.97	42.67	42.72	46.37	46.53
Average Moisture content (%)	40.75		42.70		46.45	

Table A5 : Description of Liquid Limit, W_{LL}

Description	W_{LL}
Low Plasticity	< 35
Intermediate Plasticity	35 – 50
High Plasticity	50 – 70
Very High Plasticity	70 – 90
Extremely High Plasticity	> 90

astic Limit Test

able A6 : Result of plastic limit test

Test Method : BS 1337						
Sample No.	1		2		3	
Container no	1	2	3	4	5	6
Mass of wet soil + container (g)	41.83	41.63	42.35	41.34	40.82	40.91
Mass of dry soil + container (g)	39.09	38.92	39.43	38.68	38.33	38.42
Mass of container (g)	29.33	29.34	29.2	29.26	29.51	29.53
Moisture content (%)	28.07	28.29	28.54	28.24	28.23	28.01
Average Moisture content (%)	28.18		28.39		28.12	

Plasticity Index

Table A7 : Description of Plasticity Index, I_p

Description	I_p
Non-plastic	0
Slightly Plastic	1-5
Low Plasticity	5-10
Medium Plasticity	10-20
High Plasticity	20-40
Very High Plasticity	>40

SPECIFIC GRAVITY TEST

Table A8 : Result of specific gravity test

Test Method : ASTM D 854 - 02			
Sample no.	1	2	3
Mass of jar + cap (g)	537.33	536.55	536.65
Mass of jar + cap + soil (g)	937.38	936.47	937.08
Mass of jar + cap + soil+ water (g)	1788.87	1804.95	1792.77
Mass of jar + water (g)	1516.32	1552.54	1563.45
Particle Density, ρ_s (g/cm^3)	3.14	2.71	2.34

COMPACTION TEST

Table A9 : Result of compaction for raw soils (sample 1)

Sample no.	For Moisture content (%)	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	10	11.26	1.70	17.05	15.32
2	13	13.89	1.73	17.35	15.23
3	16	16.37	1.81	18.15	15.60
4	19	19.10	1.92	19.25	16.17
5	22	22.86	1.97	19.75	16.08
6	25	25.99	1.96	19.65	15.60
7	28	28.00	1.90	19.05	14.88

Table A10 : Result of compaction for raw soils (sample 2)

Sample no.	For Moisture content (%)	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	10	11.13	1.67	16.75	15.07
2	13	13.14	1.76	17.65	15.60
3	16	17.01	1.84	18.45	15.77
4	19	20.05	1.91	19.15	15.95
5	22	23.34	1.96	19.65	15.93
6	25	24.45	1.95	19.55	15.71
7	28	27.80	1.90	19.05	14.91

Table A11 : Value of air void, corresponding to its moisture content

Moisture content	0% air voids	10% air voids	20% air voids
11.13	20.82	18.74	16.66
13.14	19.98	17.99	15.99
17.01	18.55	16.70	14.84
20.05	17.56	15.80	14.05
23.34	16.60	14.94	13.28
24.45	16.30	14.67	13.04
27.80	15.46	13.91	12.37

ORGANIC MATTER CONTENT

Table A12 : Result of organic matter test

Test Method : ASTM 2974 - 00						
Foil No.	1	2	3	4	5	6
Mass of oven- dried soil + foil (g)	16.07	13.92	21.47	23.82	4.09	4.00
Mass of ash + foil (g)	15.32	13.18	20.43	22.75	3.81	3.70
Mass of foil (g)	1.18	1.18	1.18	1.18	1.19	1.17
Mass of ash (g)	14.14	12.00	19.25	21.57	2.61	2.53
Mass of oven- dried soil (g)	14.89	12.74	20.29	22.64	2.89	2.83
Ash content (%)	94.96	94.19	94.87	95.27	90.32	89.5
Organic matter (%)	5.04	5.81	5.13	4.73	9.68	10.5

CHEMICAL COMPOSITION TEST

Table A13: Elemental composition in weight percentage of soil

Chemicals	Composition (%)
Na_2O	0.205
MgO	0.596
Al_2O_3	36.700
SiO_2	55.100
P_2O_5	0.133
SO_3	0.173
K_2O	3.150
CaO	0.401
TiO_2	0.697
MnO	0.024
Fe_2O_3	2.620
NiO	0.00316
Rb_2O	0.03120
SrO	0.00394
Y_2O_3	0.00998
ZrO_2	0.02430
Nb_2O_5	0.00926
Re	0.05770

APPENDIX B – BOTTOM FUEL ASH

GRADATION TEST (MECHANICAL SIEVING)

Table B1 : Sample 1 – Particle size distribution of bottom fuel ash

Test Method : BS 1337					
Sieve	Mass of Sieve (g)	Ash Retained + Sieve (g)	Ash Retained on Sieve (g)	Percentage Retained, %	Cumulative Percentage Passing, %
3.35 mm	484.37	500.71	16.34	5.45	94.55
2.00 mm	389.38	394.86	5.48	1.83	92.73
1.18 mm	434.95	443.47	8.52	2.84	89.89
600 μm	405.46	421.26	15.80	5.27	84.62
425 μm	296.46	334.29	37.83	12.61	72.01
300 μm	286.16	346.59	60.43	20.14	51.87
212 μm	275.93	324.6	48.67	16.22	35.64
150 μm	269.33	298.85	29.52	9.84	25.80
63 μm	327.97	371.95	43.98	14.66	11.14
Pan	245.82	279.25	33.43	11.14	0.00
Total			300.00	100.00	

Mass of bottom fuel ash = 300

Table B2 : Sample 2 – Particle size distribution of bottom fuel ash

Test Method : BS 1337					
Sieve	Mass of Sieve (g)	Ash Retained + Sieve (g)	Ash Retained on Sieve (g)	Percentage Retained (%)	Cumulative Percentage Passing (%)
3.35 mm	484.37	502.23	17.86	5.95	94.05
2.00 mm	389.33	398.06	8.73	2.91	91.14
1.18 mm	434.78	443.04	8.26	2.75	88.38
600 μm	405.38	423.82	18.44	6.15	82.24
425 μm	296.47	336.05	39.58	13.19	69.04
300 μm	286.17	349.47	63.30	21.10	47.94
212 μm	275.88	323.75	47.87	15.96	31.99
150 μm	269.16	296.19	27.03	9.01	22.98
63 μm	327.84	364.15	36.31	12.10	10.87
Pan	245.83	278.45	32.62	10.87	0.00
Total			300.00	100.00	

Mass of bottom fuel ash = 300 g

Table B3 : Summarize of particle size distribution of bottom fuel ash

Size	Sample 1	Sample 2	Average
Gravel-like size (%)	5.45	5.95	5.70
Sand- like size (%)	83.41	83.17	83.29
Silt-like size (%)	11.14	10.87	11.01
Total (%)	100	100	100

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.35}{0.06} = 5.8$$

$$Cc = \frac{(D_{30})^2}{(D_{60})(D_{10})} = \frac{0.18^2}{(0.35)(0.06)} = 1.54$$

CHEMICAL COMPOSITION TEST

Table B4: Elemental composition in weight percentage of bottom fuel ash

Chemicals	Composition (%)
Na_2O	0.202
MgO	0.983
Al_2O_3	20.100
SiO_2	57.100
P_2O_5	0.259
SO_3	2.220
K_2O	1.400
CaO	6.390
TiO_2	1.820
MnO	0.059
Fe_2O_3	8.870
SrO	0.164
Y_2O_3	0.027
ZrO_2	0.166
Tb_4O_7	0.035
Re	0.202

Table B5 : Chemical Composition of class- F ash (TNB power plant)

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No. :

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Certificate of Sampling and Analysis

ULTIMATE ANALYSIS :

- Carbon	(dry ash free basis)	:	74.2	%
- Hydrogen	(dry ash free basis)	:	5.11	%
- Nitrogen	(dry ash free basis)	:	0.90	%
- Sulfur	(dry ash free basis)	:	0.12	%
- Oxygen	(dry ash free basis)	:	19.7	%
- Phosphorus	(dry basis)	:	0.002	%
- Chlorine	(dry basis)	:	< 0.01	%
- Sodium in Ash	(dry basis)	:	0.30	%
- Hardgrove Grindability Index		:	49	
- Abrasive Index		:	3.62	mg metal/kg coal

Ash Fusion Temperature :		Reducing Atmosphere			Oxidizing Atmosphere	
- Deformation	:	1120	°C	:	1260	°C
- Hemisphere	:	1140	°C	:	1290	°C
- Flow	:	1250	°C	:	1320	°C

Ash Analysis :

- SiO_2	(dry basis)	:	30.03	%
- Al_2O_3	(dry basis)	:	20.26	%
- Fe_2O_3	(dry basis)	:	20.52	%
- TiO_2	(dry basis)	:	0.82	%
- CaO	(dry basis)	:	13.70	%
- MgO	(dry basis)	:	3.63	%
- K_2O	(dry basis)	:	0.59	%
- Na_2O	(dry basis)	:	0.30	%
- P_2O_5	(dry basis)	:	0.31	%
- Mn_3O_4	(dry basis)	:	0.31	%
- SO_3	(dry basis)	:	9.07	%
- BaO	(dry basis)	:	0.19	%
- Undetermined	(dry basis)	:	0.27	%

Size Distribution :

0 x 50 mm	:	97.1	%
0 x 3 mm	:	21.8	%
0 x 0.5 mm	:	7.2	%

September 25, 2003
Jakarta

Signed for and on behalf of

PT. SUCOFINDO

Ref. : BJM/35.10/ADR355A/EXP/03
hw.-

This inspection order has been accepted and this certificate/report is issued subject to the Standard General Conditions of the INTERNATIONAL FEDERATION OF INSPECTION AGENCIES (IFIA). The company's liability is limited under the terms of Article 10 thereof. Issuance of this certificate/report

Table B6: Chemical Composition of class- F ash (TNB power plant)

PT. GEOSERVICES LTD.

MARINE CARGO SURVEYING

Head Office : BANDUNG, Jl. Setiabudi No. 79-81, Ph. 022-[illegible], Fx. 022-[illegible]; Branch Office : BALIKPAPAN, Jl. M.T. Haryono No. [illegible], Ph. [illegible] 672188/672185, Fx. 0542-872181; SAMARINDA, Jl. Wijaya Kusumah No. 28A, Ph. 0541-203728, Fx. 0541-203727; BANJARBARU, Jl. Jend. A. Yani [illegible], Ph. 0511-772818 (Hunting), Fx. 0511-772324; KOTABARU, Jl. Jend. Sudirman No. 336AB, Ph./Fx. 0518-21648; TANJUNG REDEB-BERAU, Jl. Raya Bukit [illegible] Gunung Tabur, Ph. 0554-23138, Fx. 0554-23364; GUNUNG BAYAN, Mansu Laboratory, Mansu-Muara Pahu, Kutai Barat, Ph. 0542-[illegible]

ULTIMATE ANALYSIS (DRY ASH FREE)			
Carbon	%	79.12	ISO 609 ; 1996 (E)
Hydrogen	%	6.21	ISO 609 ; 1996 (E)
Nitrogen	%	1.35	ISO 333 ; 1996 (E)
Sulfur	%	1.02	ISO 351 ; 1996 (E)
Oxygen + Error	%	12.30	By Difference
ASH FUSION TEMPERATURE (REDUCING ATMOSPHERE)			
Initial Deformation	°C	> 1600	ISO 540 ; 1995 (E)
Hemispherical Deformation	°C	> 1600	ISO 540 ; 1995 (E)
Flow	°C	> 1600	ISO 540 ; 1995 (E)
ASH FUSION TEMPERATURE (OXIDIZING ATMOSPHERE)			
Initial Deformation	°C	> 1600	ISO 540 ; 1995 (E)
Hemispherical Deformation	°C	> 1600	ISO 540 ; 1995 (E)
Flow	°C	> 1600	ISO 540 ; 1995 (E)
PHYSICAL PROPERTIES			
Hardgrove Grindability Index		40	ISO 5074 ; 1994 (E)
Size Distribution			
0 x 50 mm	%	97.45	ISO 1953 ; 1994 (E)
0 x 3 mm	%	19.01	ISO 1953 ; 1994 (E)
0 x 0.5 mm	%	5.10	ISO 1953 ; 1994 (E)
ASH ANALYSIS			
SiO_2	%	47.23	ASTM D-3682 ; 2002
AL_2O_3	%	35.48	ASTM D-3682 ; 2002
Fe_2O_3	%	8.00	ASTM D-3682 ; 2002
CaO	%	2.89	ASTM D-3682 ; 2002
MgO	%	0.67	ASTM D-3682 ; 2002
Na_2O	%	0.52	ASTM D-3682 ; 2002
K_2O	%	0.35	ASTM D-3682 ; 2002
TiO_2	%	2.25	ASTM D-3682 ; 2002
Mn_3O_4	%	0.015	ASTM D-3682 ; 2002
SO_3	%	2.18	ASTM D-3682 ; 2002
P_2O_5	%	0.476	ASTM D-3682 ; 2002

Signed for and on behalf of
PT. Geoservices (Ltd)
(Gandawa)
Laboratory Manager

Date, November 18, 2003
Issuing Office : Banjarbaru Branch
Certificate No. 08/F-1353

APPENDIX C – STRENGTH TEST ON SOILS

UNCONFINED COMPRESSIVE STRENGTH TEST

Table C1 : Unconfined compressive strength test result(sample 1)

Test Method : ASTM D 2166-00			
Initial		Final	
Specimen Details	**Amount**	**Specimen Details**	**Amount**
Diameter, D	37.70 mm	Wet mass of soil	114.20 g
Area, A	1116.28 mm^2	Dry mass of soil	75.08 g
Height, H	71.94 mm	Moisture content	52.10 %
Volume, V	80.31 cm^3		
Mass	114.20 g		
Density	1.42 Mg/m^3		

Table C2: Unconfined compressive strength test result (sample 2)

Test Method : ASTM D 2166-00			
Initial		Final	
Specimen Details	**Amount**	**Specimen Details**	**Amount**
Diameter, D	36.93 mm	Wet mass of soil	157.13 g
Area, A	1071.15 mm^2	Dry mass of sol	114.27 g
Height, H	85 mm	Moisture content	37.50 %
Volume, V	91.05 cm^3		
Wet mass	157.13 g		
Density	1.73 Mg/m^3		

Table C3 : Unconfined Compression Test data (Sample 1)

Deformation gauge reading	Compression of specimen ΔL (mm)	Strain €=ΔL/L$_o$	% strain	Corrected area A =A$_o$/1- € (mm2)	Force gauge reading	Axial Force P	Axial stress 100P/A (kN/m^2)
0	0.0	0.000	0.00	1116.28	0.0	0.00	0.00
40	0.4	0.006	0.56	1122.52	3.0	4.05	3.61
80	0.8	0.011	1.11	1128.83	5.0	6.75	5.98
120	1.2	0.017	1.67	1135.22	7.0	9.45	8.32
160	1.6	0.022	2.22	1141.67	8.5	11.48	10.05
200	2.0	0.028	2.78	1148.20	10.5	14.18	12.35
240	2.4	0.033	3.34	1154.81	12.0	16.20	14.03
280	2.8	0.039	3.89	1161.49	14.0	18.90	16.27
320	3.2	0.044	4.45	1168.25	16.0	21.60	18.49
360	3.6	0.050	5.00	1175.08	17.5	23.63	20.10
400	4.0	0.056	5.56	1182.00	19.0	25.65	21.70
440	4.4	0.061	6.12	1189.00	20.0	27.00	22.71
480	4.8	0.067	6.67	1196.09	22.0	29.70	24.83
520	5.2	0.072	7.23	1203.25	23.5	31.73	26.37
560	5.6	0.078	7.78	1210.51	24.5	33.08	27.32
600	6.0	0.083	8.34	1217.85	25.4	34.29	28.16
640	6.4	0.089	8.90	1225.29	27.0	36.45	29.75
680	6.8	0.095	9.45	1232.81	28.0	37.80	30.66
720	7.2	0.100	10.01	1240.43	29.0	39.15	31.56
760	7.6	0.106	10.56	1248.14	30.0	40.50	32.45
800	8.0	0.111	11.12	1255.95	31.0	41.85	33.32
840	8.4	0.117	11.68	1263.85	32.0	43.20	34.18
880	8.8	0.122	12.23	1271.86	33.0	44.55	35.03
920	9.2	0.128	12.79	1279.97	33.5	45.23	35.33
960	9.6	0.133	13.34	1288.18	34.5	46.58	36.16
1000	10.0	0.139	13.90	1296.50	35.0	47.25	36.44
1040	10.4	0.145	14.46	1304.93	36.5	49.28	37.76
1080	10.8	0.150	15.01	1313.46	37.0	49.95	38.03
1120	11.2	0.156	15.57	1322.11	37.5	50.63	38.29
1160	11.6	0.161	16.12	1330.88	38.0	51.30	38.55
1200	12.0	0.167	16.68	1339.76	38.5	51.98	38.79
1240	12.4	0.172	17.24	1348.76	39.2	52.92	39.24
1280	12.8	0.178	17.79	1357.88	39.5	53.33	39.27
1320	13.2	0.183	18.35	1367.13	40.0	54.00	39.50
1360	13.6	0.189	18.90	1376.50	40.5	54.68	39.72
1400	14.0	0.195	19.46	1386.01	40.5	54.68	39.45
1440	14.4	0.200	20.02	1395.64	41.0	55.35	39.66
1480	14.8	0.206	20.57	1405.41	41.2	55.62	39.58
1520	15.2	0.211	21.13	1415.32	41.2	55.62	39.30
1560	15.6	0.217	21.68	1425.37	41.2	55.62	39.02

Table C4 : Unconfined Compression Test data (Sample 2)

Deformation gauge reading	Compression of specimen ΔL (mm)	Strain $\epsilon=\Delta L/L_o$	% strain	Corrected area $A = A_o/1- \epsilon$ (mm2)	Force gauge reading	Axial Force P	Axial stress (kN/m^2) =1000P/A
0	0.0	0.000	0.00	1071.15	0.0	0.00	0.00
20	0.2	0.002	0.24	1073.68	2.0	2.70	2.51
40	0.4	0.005	0.47	1076.21	4.0	5.40	5.02
60	0.6	0.007	0.71	1078.76	5.5	7.43	6.88
80	0.8	0.009	0.94	1081.33	7.5	10.13	9.36
100	1.0	0.012	1.18	1083.90	9.2	12.42	11.46
120	1.2	0.014	1.41	1086.49	10.5	14.18	13.05
140	1.4	0.016	1.65	1089.09	13.0	17.55	16.11
160	1.6	0.019	1.88	1091.70	15.0	20.25	18.55
180	1.8	0.021	2.12	1094.32	17.0	22.95	20.97
200	2.0	0.024	2.35	1096.96	18.0	24.30	22.15
220	2.2	0.026	2.59	1099.61	20.0	27.00	24.55
240	2.4	0.028	2.82	1102.27	21.0	28.35	25.72
260	2.6	0.031	3.06	1104.95	21.5	29.03	26.27
280	2.8	0.033	3.29	1107.64	23.5	31.73	28.64
300	3.0	0.035	3.53	1110.34	25.0	33.75	30.40
320	3.2	0.038	3.76	1113.05	26.0	35.10	31.53
340	3.4	0.040	4.00	1115.78	26.8	36.18	32.43
360	3.6	0.042	4.24	1118.52	27.5	37.13	33.19
380	3.8	0.045	4.47	1121.28	28.5	38.48	34.31
400	4.0	0.047	4.71	1124.05	29.8	40.23	35.79
420	4.2	0.049	4.94	1126.83	30.8	41.58	36.90
440	4.4	0.052	5.18	1129.62	32.0	43.20	38.24
460	4.6	0.054	5.41	1132.43	33.0	44.55	39.34
480	4.8	0.056	5.65	1135.26	34.2	46.17	40.67
500	5.0	0.059	5.88	1138.10	35.0	47.25	41.52
520	5.2	0.061	6.12	1140.95	35.8	48.33	42.36
540	5.4	0.064	6.35	1143.82	36.5	49.28	43.08
560	5.6	0.066	6.59	1146.70	37.5	50.63	44.15
580	5.8	0.068	6.82	1149.59	38.5	51.98	45.21
600	6.0	0.071	7.06	1152.50	39.0	52.65	45.68
620	6.2	0.073	7.29	1155.43	39.8	53.73	46.50
640	6.4	0.075	7.53	1158.37	41.0	55.35	47.78
660	6.6	0.078	7.76	1161.32	41.8	56.43	48.59
680	6.8	0.080	8.00	1164.29	42.6	57.51	49.39
700	7.0	0.082	8.24	1167.28	43.5	58.73	50.31
720	7.2	0.085	8.47	1170.28	44.5	60.08	51.33
740	7.4	0.087	8.71	1173.30	45.0	60.75	51.78
760	7.6	0.089	8.94	1176.33	45.8	61.83	52.56
780	7.8	0.092	9.18	1179.38	48.0	64.80	54.94
800	8.0	0.094	9.41	1182.44	48.8	65.88	55.72
820	8.2	0.096	9.65	1185.52	49.5	66.83	56.37
840	8.4	0.099	9.88	1188.61	50.0	67.50	56.79
860	8.6	0.101	10.12	1191.72	50.1	67.64	56.75

880	8.8	0.104	10.35	1194.85	51.2	69.12	57.85
900	9.0	0.106	10.59	1198.00	51.8	69.93	58.37
920	9.2	0.108	10.82	1201.16	52.0	70.20	58.44
940	9.4	0.111	11.06	1204.34	52.8	71.28	59.19
960	9.6	0.113	11.29	1207.53	53.0	71.55	59.25
980	9.8	0.115	11.53	1210.74	54.2	73.17	60.43
1000	10.0	0.118	11.76	1213.97	54.8	73.98	60.94
1020	10.2	0.120	12.00	1217.22	54.8	73.98	60.78
1040	10.4	0.122	12.24	1220.48	55.0	74.25	60.84
1060	10.6	0.125	12.47	1223.76	55.6	75.06	61.34
1080	10.8	0.127	12.71	1227.06	56.0	75.60	61.61
1100	11.0	0.129	12.94	1230.38	56.4	76.14	61.88
1120	11.2	0.132	13.18	1233.71	56.8	76.68	62.15
1140	11.4	0.134	13.41	1237.06	57.2	77.22	62.42
1160	11.6	0.136	13.65	1240.43	57.5	77.63	62.58
1180	11.8	0.139	13.88	1243.82	58.2	78.57	63.17
1200	12.0	0.141	14.12	1247.23	58.4	78.84	63.21
1220	12.2	0.144	14.35	1250.66	60.0	81.00	64.77
1240	12.4	0.146	14.59	1254.10	60.4	81.54	65.02
1260	12.6	0.148	14.82	1257.57	60.8	82.08	65.27
1280	12.8	0.151	15.06	1261.05	61.2	82.62	65.52
1300	13.0	0.153	15.29	1264.55	61.5	83.03	65.66
1320	13.2	0.155	15.53	1268.07	61.8	83.43	65.79
1340	13.4	0.158	15.76	1271.62	62.0	83.70	65.82
1360	13.6	0.160	16.00	1275.18	62.2	83.97	65.85
1380	13.8	0.162	16.24	1278.76	62.5	84.38	65.98
1400	14.0	0.165	16.47	1282.36	62.8	84.78	66.11
1420	14.2	0.167	16.71	1285.99	63.0	85.05	66.14
1440	14.4	0.169	16.94	1289.63	63.2	85.32	66.16
1460	14.6	0.172	17.18	1293.29	63.5	85.73	66.28
1480	14.8	0.174	17.41	1296.98	63.9	86.27	66.51
1500	15.0	0.176	17.65	1300.68	64.1	86.54	66.53
1520	15.2	0.179	17.88	1304.41	64.4	86.94	66.65
1540	15.4	0.181	18.12	1308.16	64.5	87.08	66.56
1560	15.6	0.184	18.35	1311.93	64.8	87.48	66.68
1580	15.8	0.186	18.59	1315.72	64.8	87.48	66.49
1600	16.0	0.188	18.82	1319.53	64.9	87.62	66.40
1620	16.2	0.191	19.06	1323.37	65.0	87.75	66.31
1640	16.4	0.193	19.29	1327.23	65.2	88.02	66.32
1660	16.6	0.195	19.53	1331.11	65.2	88.02	66.13
1680	16.8	0.198	19.76	1335.01	65.4	88.29	66.13
1700	17.0	0.200	20.00	1338.94	65.4	88.29	65.94
1720	17.2	0.202	20.24	1342.89	65.4	88.29	65.75

Table C5 : Consistency of clays in correlation with unconfined compressive strength

Unconfined Compressive Strength, q_u (kN/m^2)	Soil Type
< 25	Very soft
25 - 50	Soft
50 - 100	Medium (Firm)
100 - 200	Stiff
200 - 400	Very stiff
> 400	Hard

Table C6: Consistency in correlation with undrained shear strength

Undrained Shear Strength, s_u (kPa)	Soil Type
< 12.5	Very soft
12.5 - 25	Soft
25 - 50	Medium (Firm)
50 - 100	Stiff
100 - 200	Very stiff

Table C7 : Wet sieve for determination of amount of sand in unconfined compressive sample

Sample details	Amount
Weight of siever (g)	327.34
Weight of siever + dry soil (g)	427.34
Weight of siever + dry sand (g)	334.68
Percentage of sand (%)	7.34

APPENDIX D

Table D1 : Result of compaction for raw soils (sample 1)

Sample no.	For Moisture content (%)	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	10	11.26	1.70	17.05	15.32
2	13	13.89	1.73	17.35	15.23
3	16	16.37	1.81	18.15	15.60
4	19	19.10	1.92	19.25	16.17
5	22	22.86	1.97	19.75	16.08
6	25	25.99	1.96	19.65	15.60
7	28	28.00	1.90	19.05	14.88

Table D2 : Result of compaction for raw soils (sample 2)

Sample no.	For Moisture content (%)	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	10	11.13	1.67	16.75	15.07
2	13	13.14	1.76	17.65	15.60
3	16	17.01	1.84	18.45	15.77
4	19	20.05	1.91	19.15	15.95
5	22	23.34	1.96	19.65	15.93
6	25	24.45	1.95	19.55	15.71
7	28	27.80	1.90	19.05	14.91

Table D3 : Value of air void, corresponding to its moisture content

Moisture content	0% air voids	10% air voids	20% air voids
11.13	20.82	18.74	16.66
13.14	19.98	17.99	15.99
17.01	18.55	16.70	14.84
20.05	17.56	15.80	14.05
23.34	16.60	14.94	13.28
24.45	16.30	14.67	13.04
27.80	15.46	13.91	12.37

Table D4 : Result of compaction for soils + 2% BFA

Sample no.	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	15.66	1.81	18.15	15.69
2	18.61	1.85	18.55	15.64
3	23.41	1.98	19.85	16.09
4	26.86	1.93	19.35	15.26
5	28.78	1.86	18.65	14.48

Table D5 : Result of compaction for soils + 4% BFA

Sample no.	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	16.44	1.77	17.75	15.24
2	18.31	1.86	18.65	15.76
3	20.44	1.93	19.35	16.07
4	23.37	1.93	19.35	15.69
5	28.54	1.86	18.65	14.51

Table D6 : Result of compaction for soils + 5% BFA

Sample no.	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	17.89	1.78	17.85	15.14
2	17.87	1.86	18.65	15.82
3	21.38	1.93	19.35	15.94
4	21.49	1.92	19.25	15.85
5	28.19	1.87	18.75	14.63

Table D7 : Result of compaction for soils + 10% BFA

Sample no.	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	18.83	1.76	17.65	14.85
2	21.99	1.90	19.05	15.62
3	23.72	1.91	19.15	15.48
4	26.55	1.85	18.55	14.66
5	30.85	1.80	18.05	13.79

Table D8 : Result of compaction for soils + 15% BFA

Sample no.	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	22.53	1.74	17.45	14.24
2	24.68	1.82	18.25	14.64
3	25.87	1.85	18.55	14.74
4	28.51	1.83	18.35	14.28
5	30.61	1.81	18.15	13.90

Table D9 : Result of compaction for soils + 20% BFA

Sample no.	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	21.52	1.70	17.05	14.03
2	24.16	1.78	17.85	14.38
3	26.82	1.83	18.35	14.47
4	28.29	1.82	18.25	14.23
5	30.27	1.81	18.15	13.93

Table D10 : Result of compaction for soils + 25% BFA

Sample no.	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	24.66	1.68	16.85	13.51
2	26.46	1.75	17.55	13.88
3	28.48	1.79	17.95	13.97
4	30.78	1.77	17.75	13.57
5	32.76	1.74	17.45	13.14

Table D11 : Result of compaction for soils + 30% BFA

Sample no.	Calculated Moisture content (%)	Mass of compacted soil (kg	Bulk Density, ρ_m (kN/m^3)	Dry Density, ρ_d (kN/m^3)
1	24.15	1.67	16.75	13.49
2	26.33	1.75	17.55	13.89
3	29.05	1.77	17.75	13.75
4	31.12	1.75	17.55	13.38
5	32.59	1.73	17.35	13.08

Table D14 : Summary of results for compaction test

Sample	Dry Density (kN/m^3)	Moisture Content (%)
Raw soils	16.20	20.50
Soils + 2% BFA	16.05	21.00
Soils + 4% BFA	15.95	21.50
Soils + 5% BFA	15.90	21.50
Soils + 10% BFA	15.40	23.00
Soils + 15% BFA	14.68	26.00
Soils + 20% BFA	14.45	26.00
Soils + 25% BFA	13.95	28.00
Soils + 30% BFA	13.85	27.50

APPENDIX E

UNCONFINED COMPRESSIVE STRENGTH TEST (IMMEDIATE TEST)

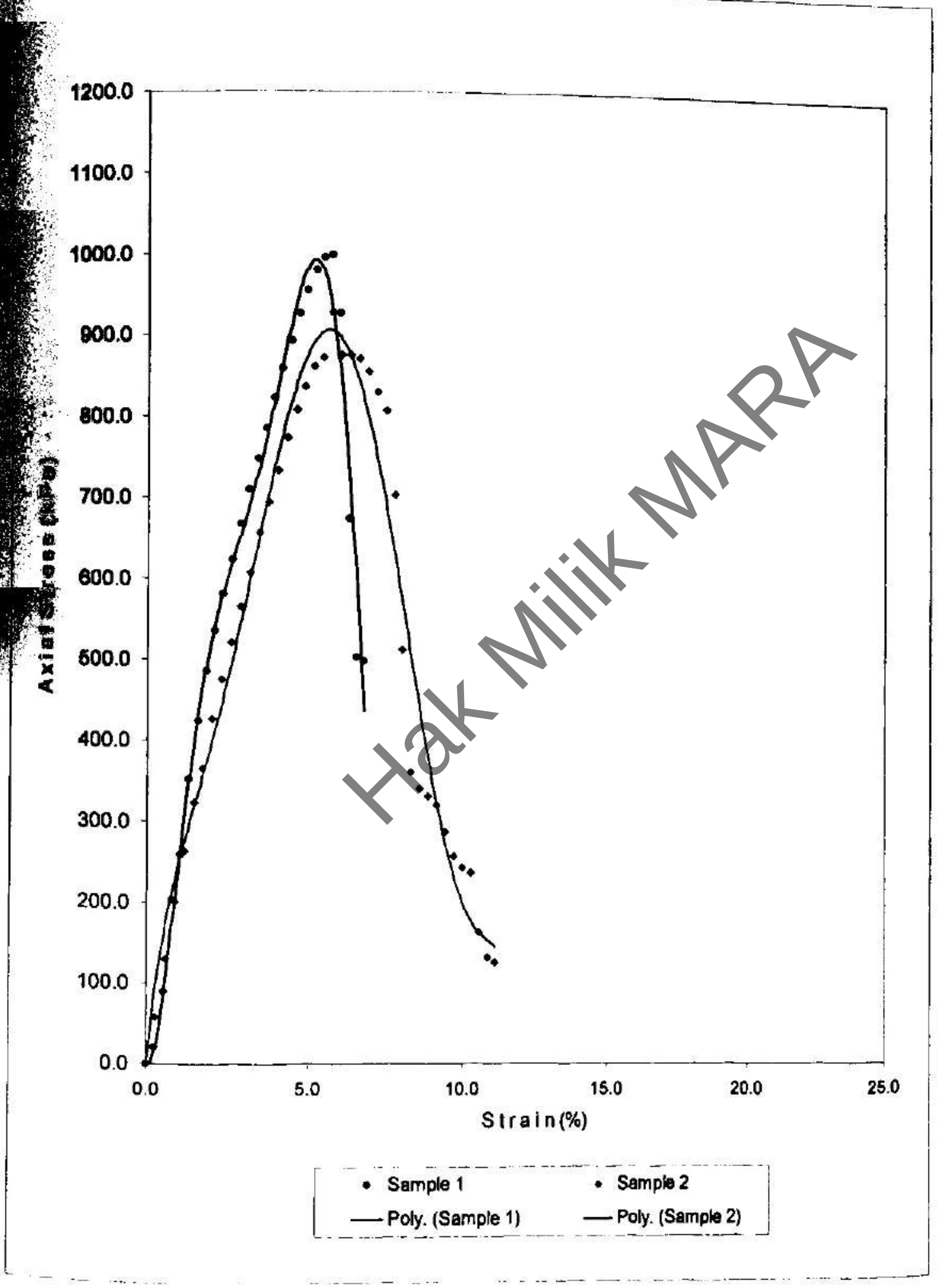

Figure E1 : Graph of axial stress vs. strain for soils + 2% BFA (immediate test)

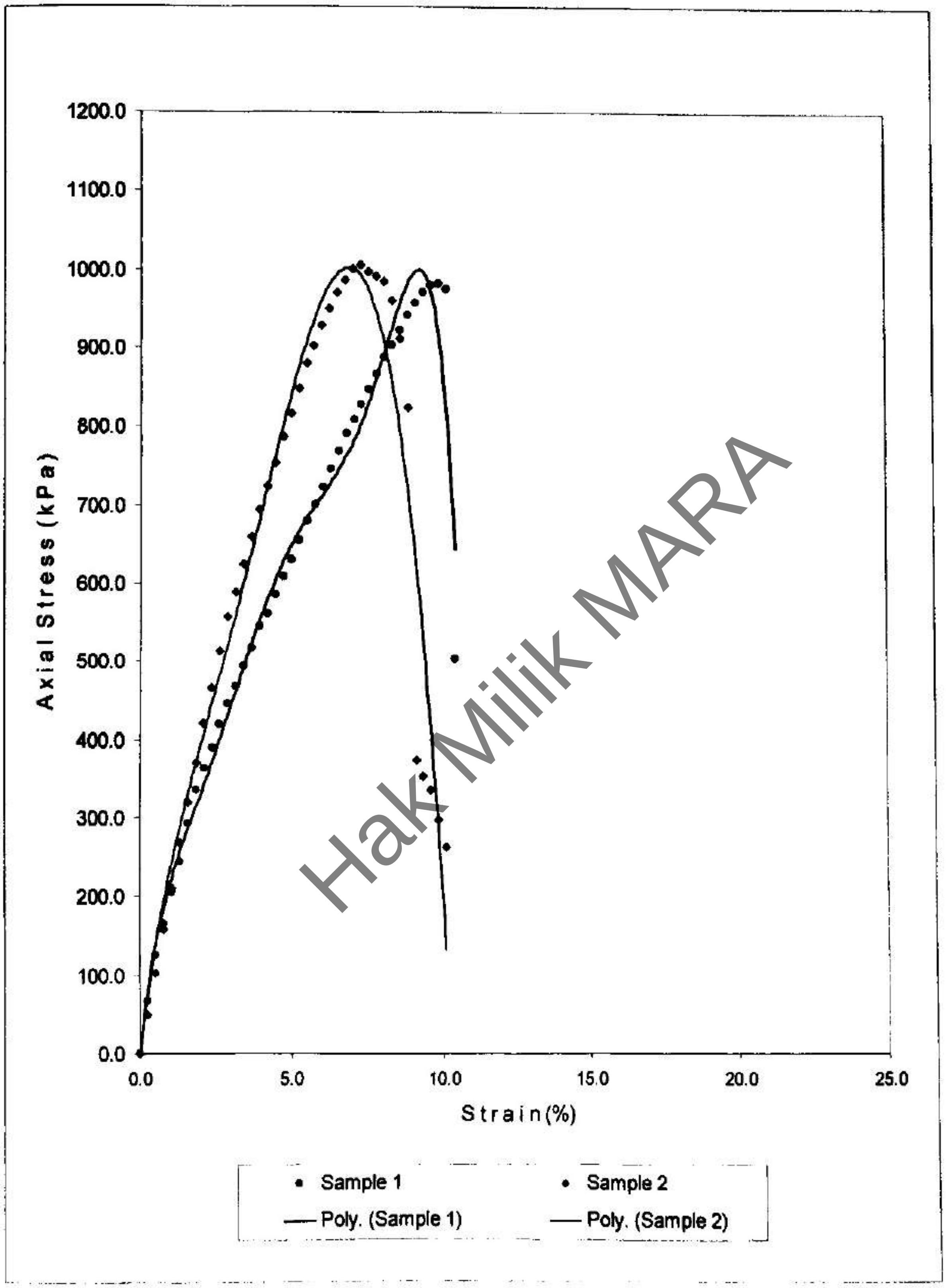

Figure E2 : Graph of axial stress vs. strain for soils + 4% BFA (immediate test)

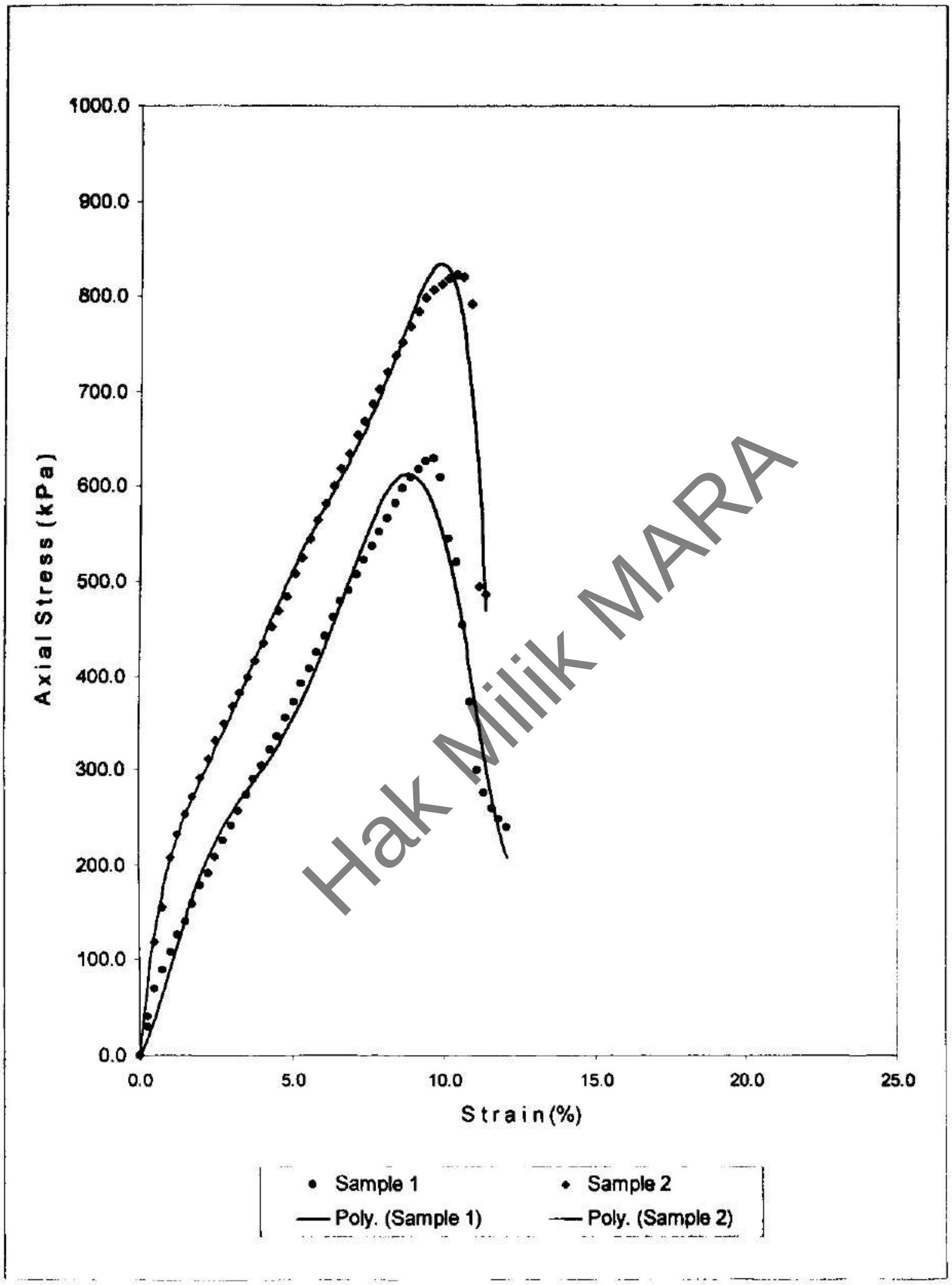

Figure E3 : Graph of axial stress vs. strain for soils + 5% BFA (immediate test)

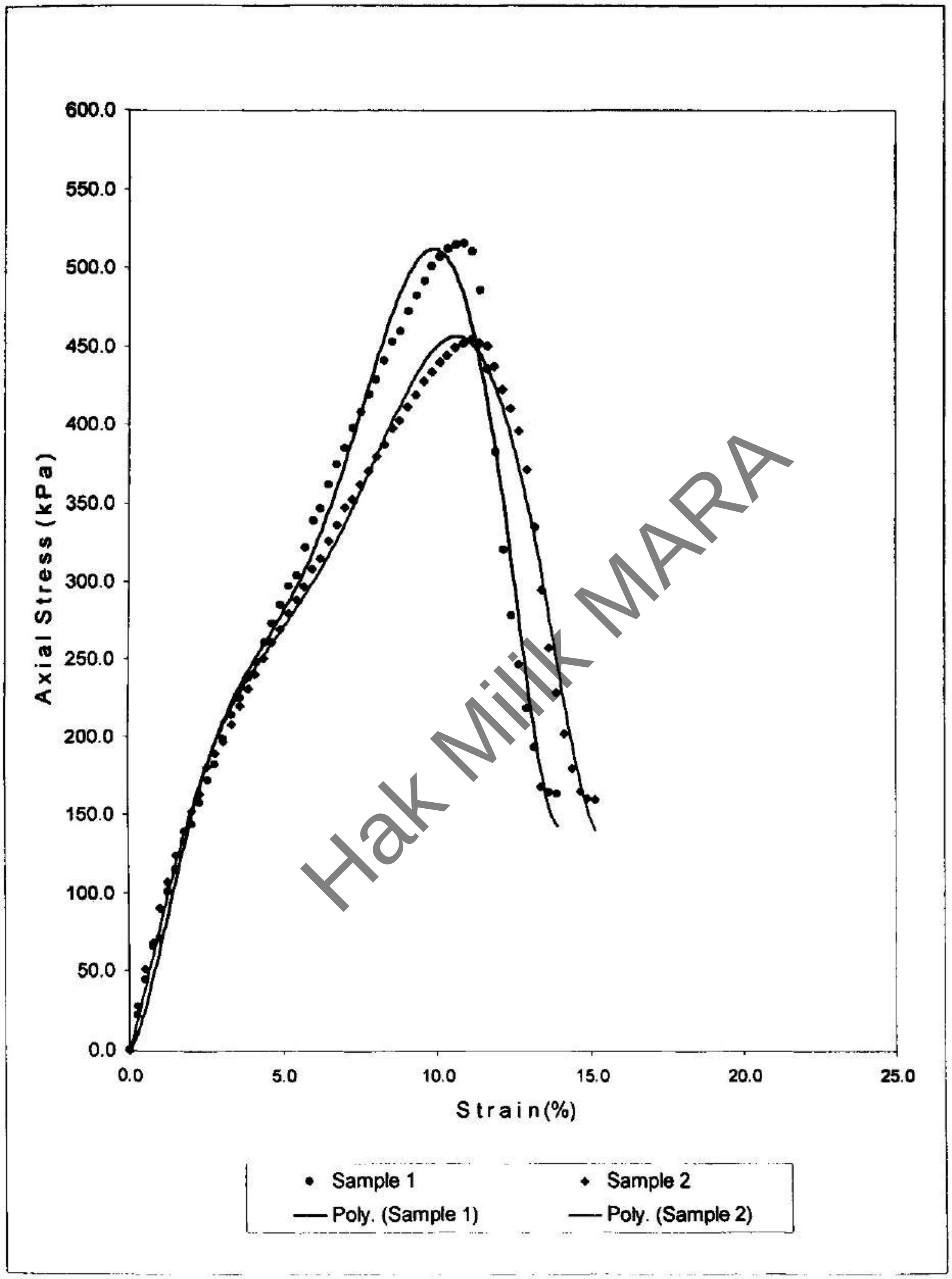

Figure E4 : Graph of axial stress vs. strain for soils + 10% BFA (immediate test)

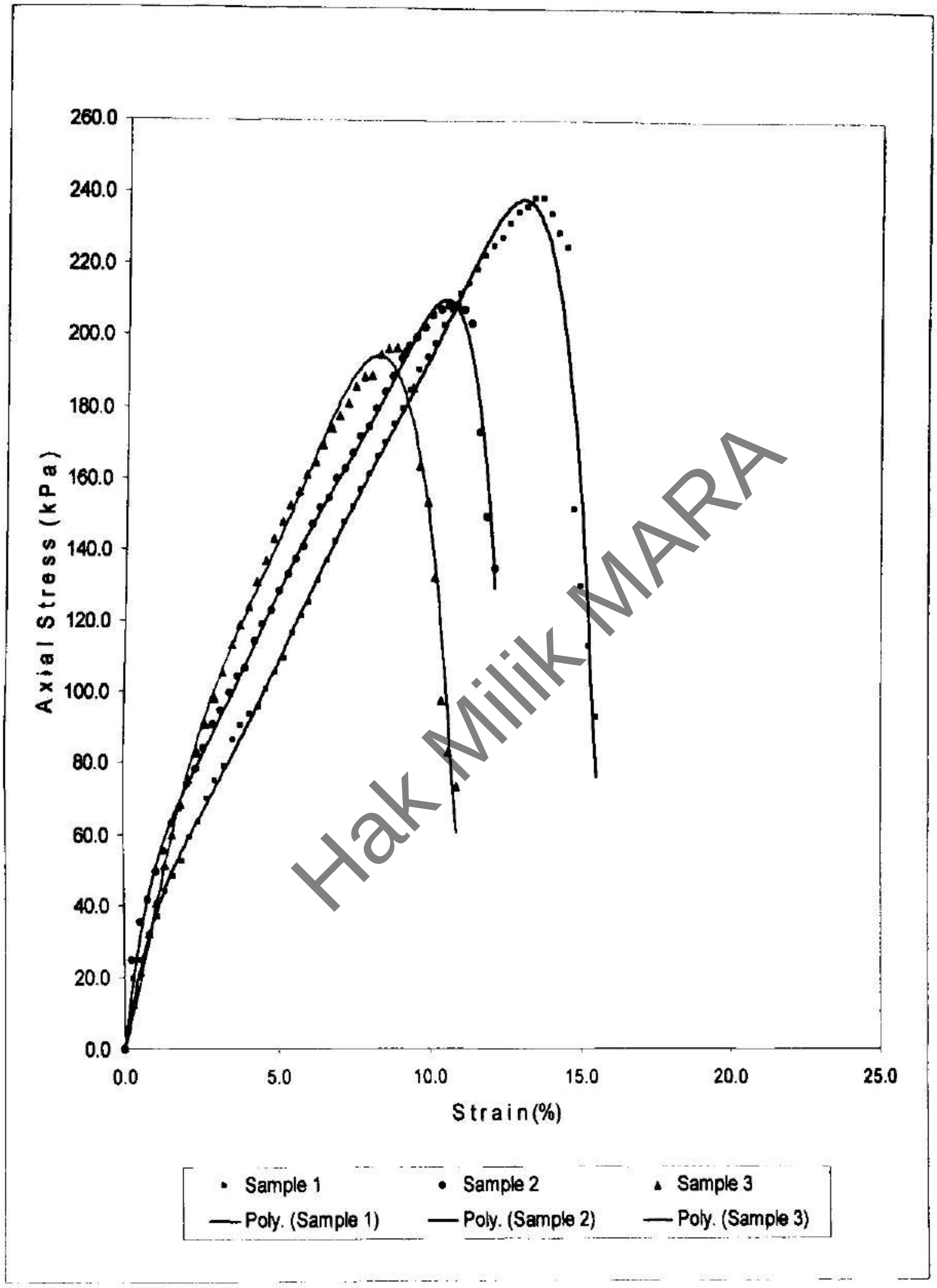

Figure E5 : Graph of axial stress vs. strain for soils + 15% BFA (immediate test)

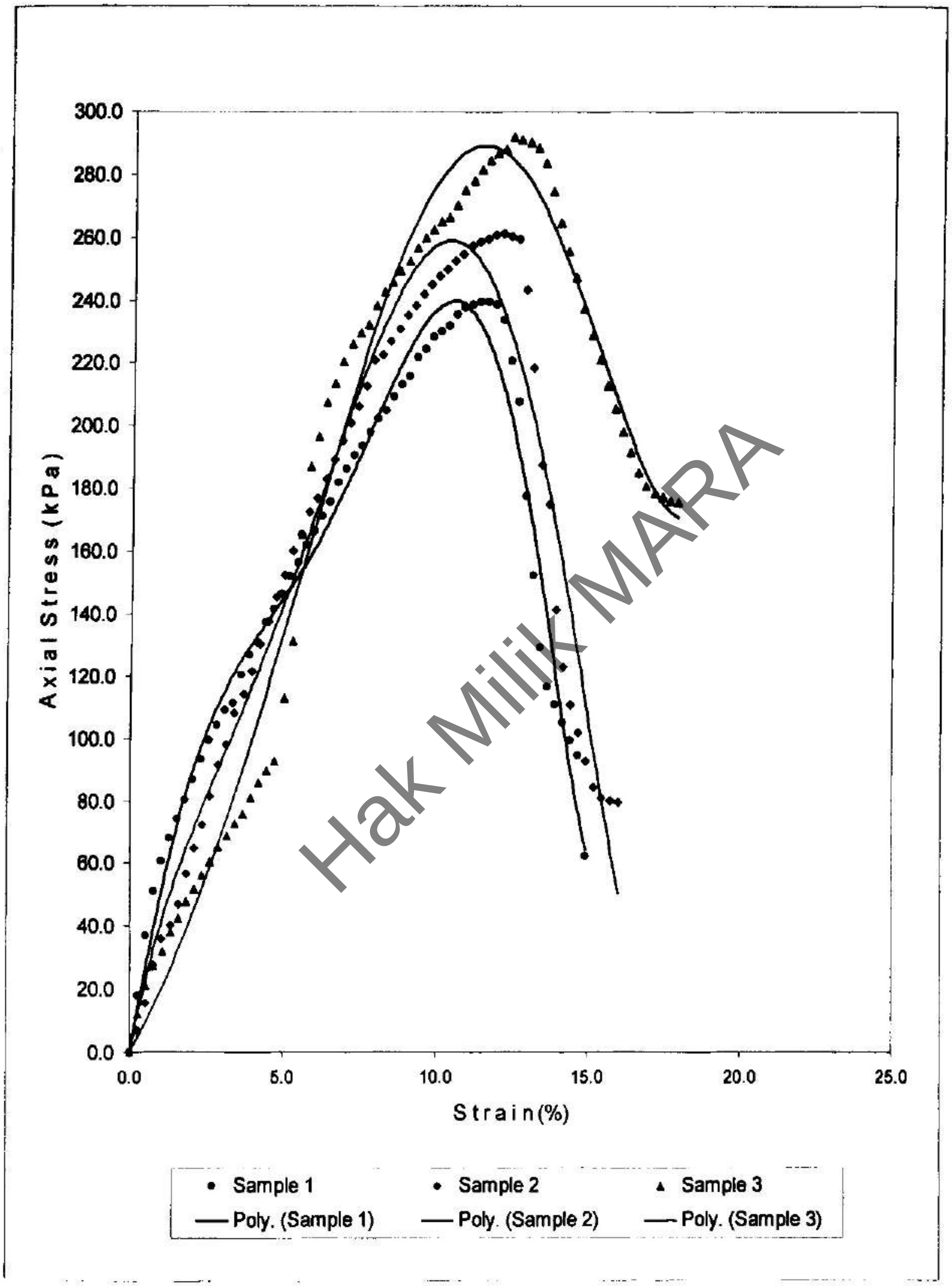

Figure E6 : Graph of axial stress vs. strain for soils + 20% BFA (immediate test)

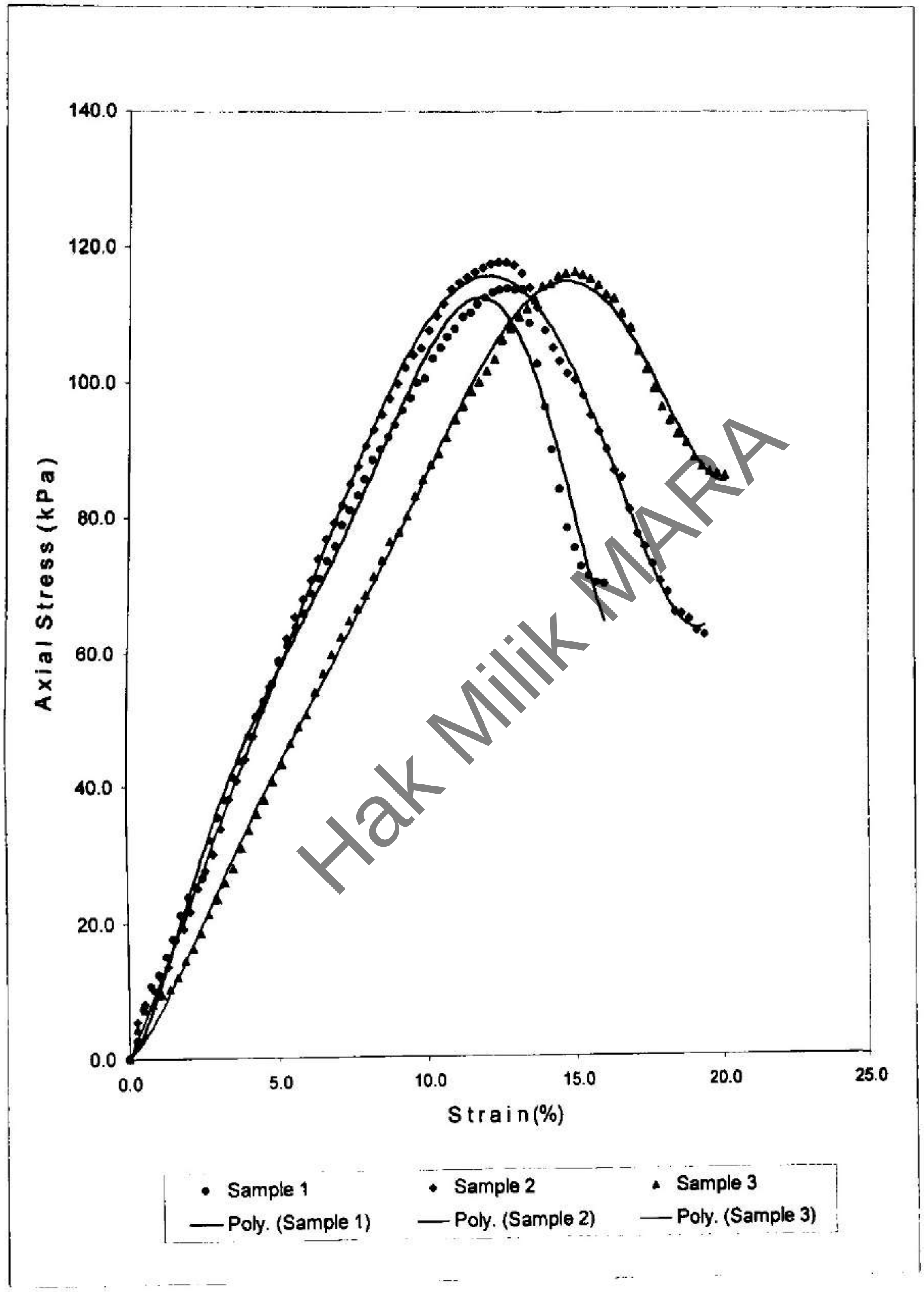

Figure E7 : Graph of axial stress vs. strain for soils + 25% BFA (immediate test)

APPENDIX F

UNCONFINED COMPRESSIVE STRENGTH TEST (1 DAY CURING)

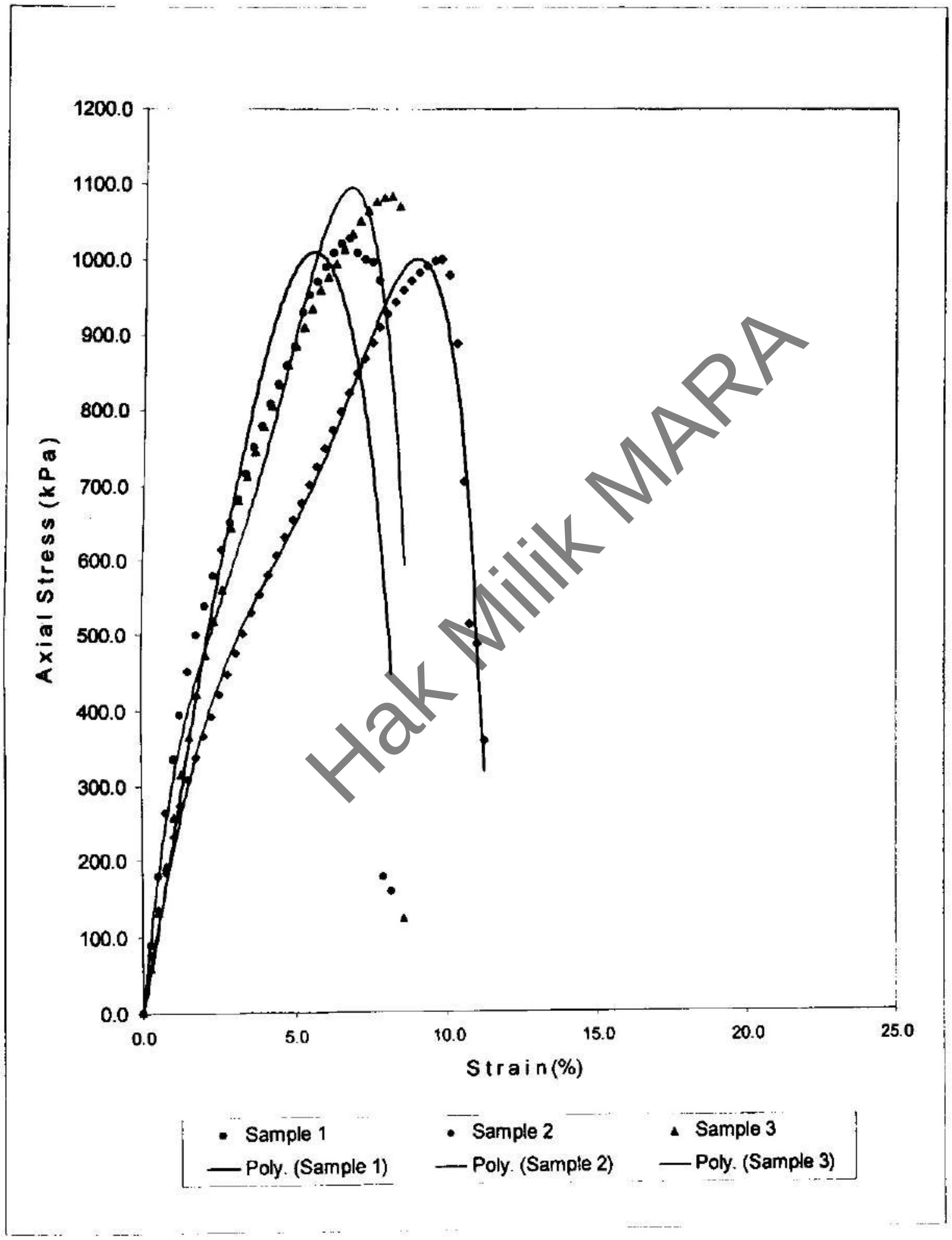

Figure F1 : Graph of axial stress vs. strain for soils + 2% BFA under 1 day curing

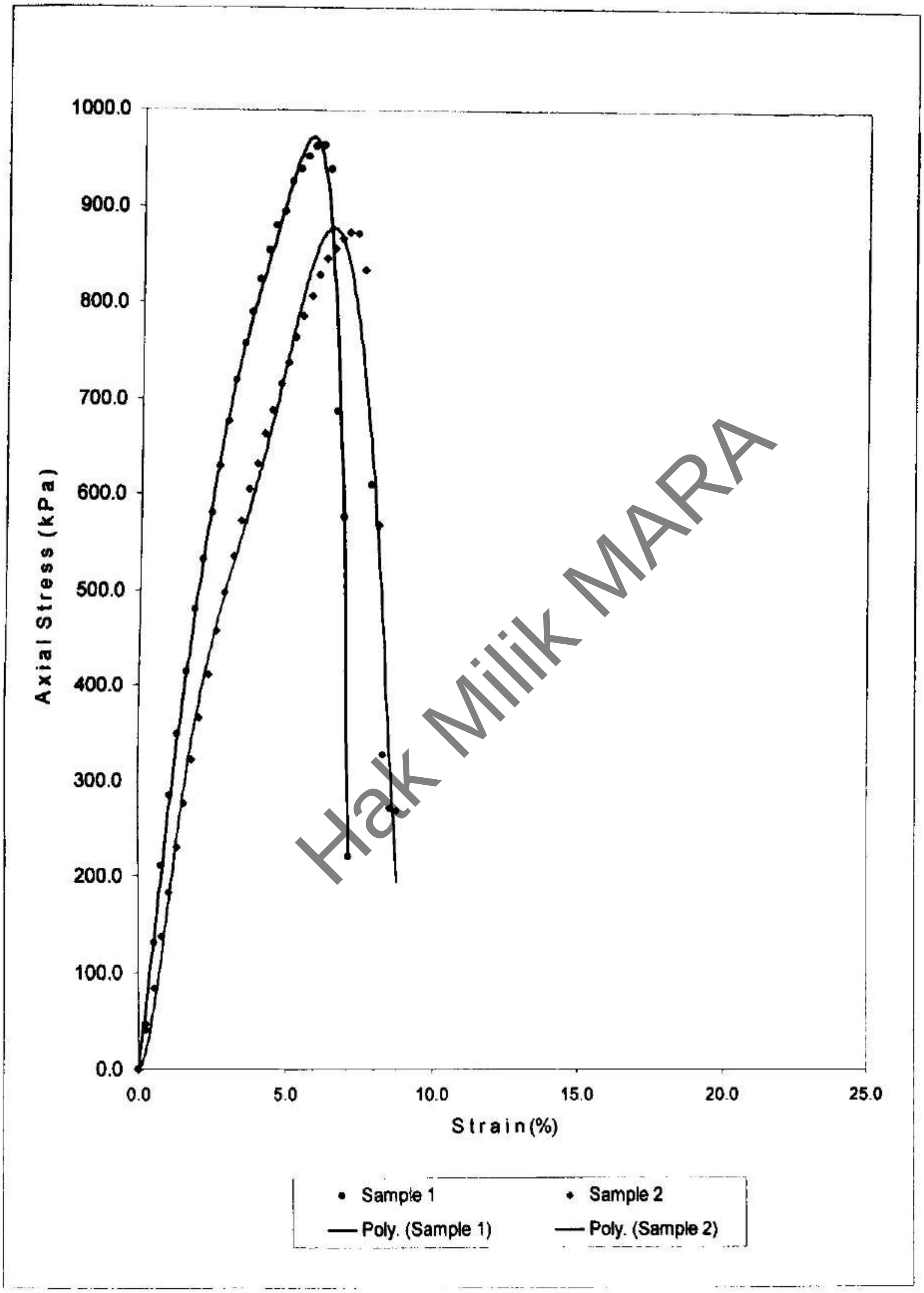

Figure F2 : Graph of axial stress vs. strain for soils + 4% BFA under 1 day curing

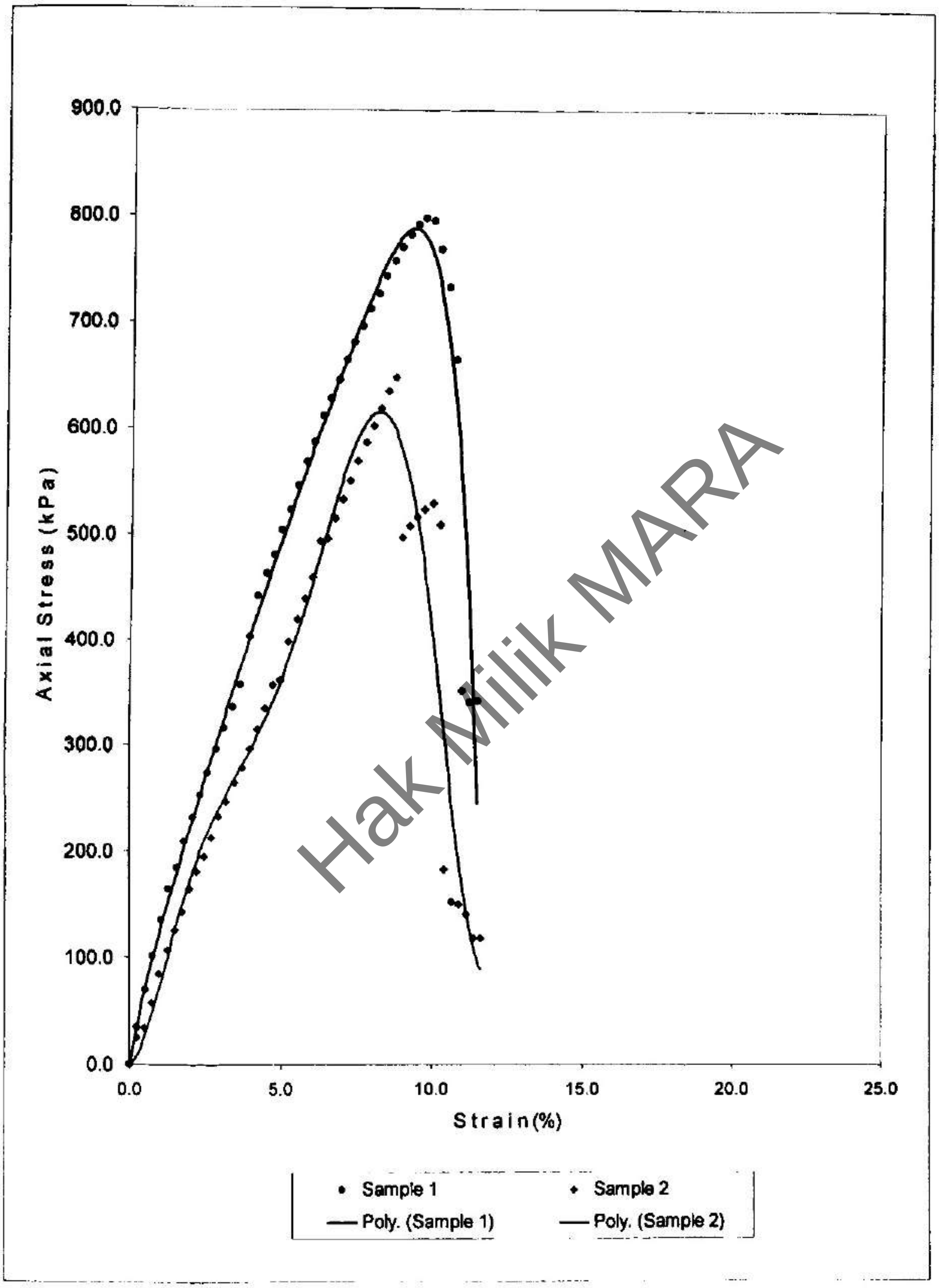

Figure F3 : Graph of axial stress vs. strain for soils + 5% BFA under 1 day curing

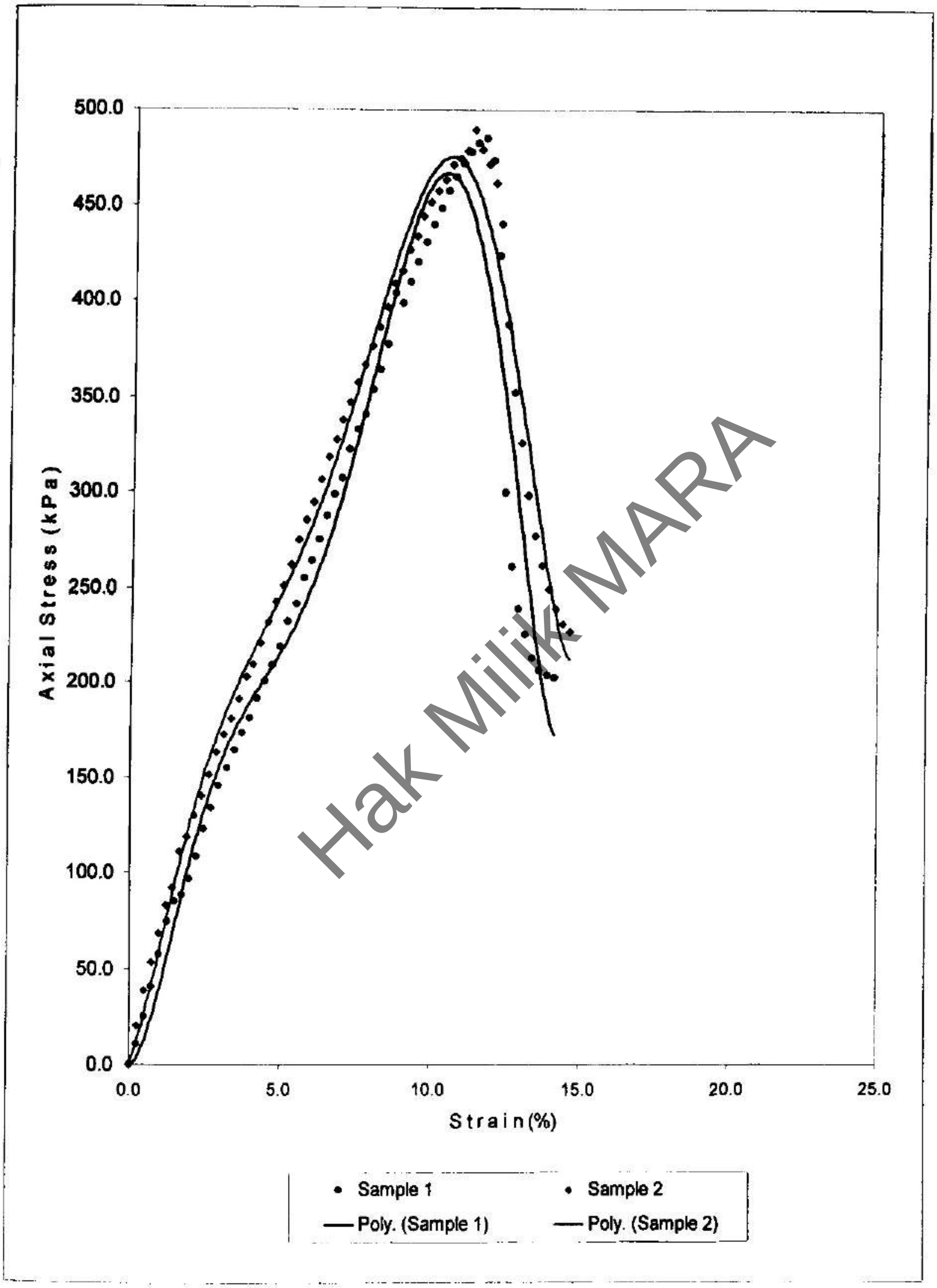

Figure F4 : Graph of axial stress vs. strain for soils + 10% BFA under 1 day curing

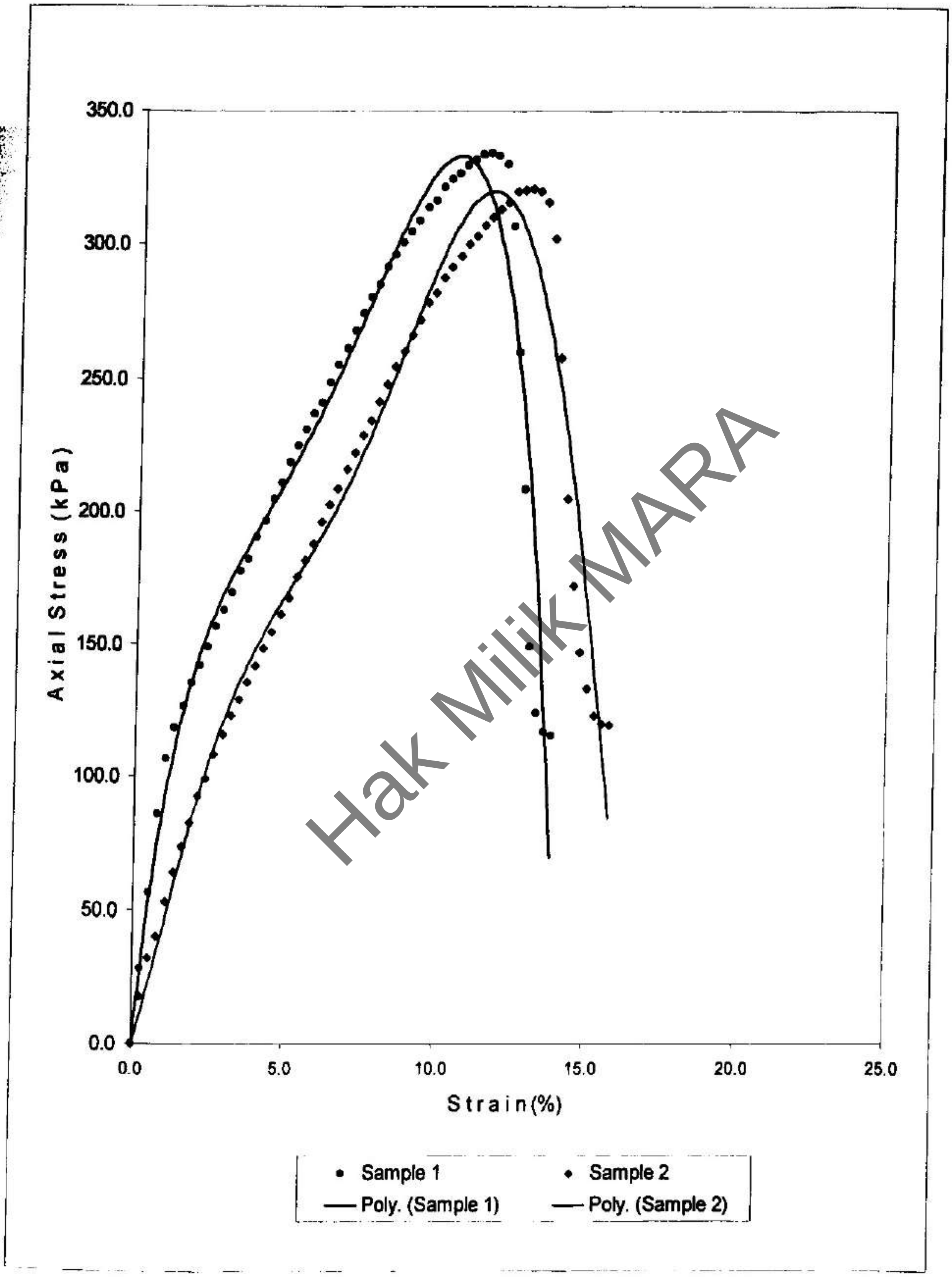

Figure F5 : Graph of axial stress vs. strain for soils + 15% BFA under 1 day curing

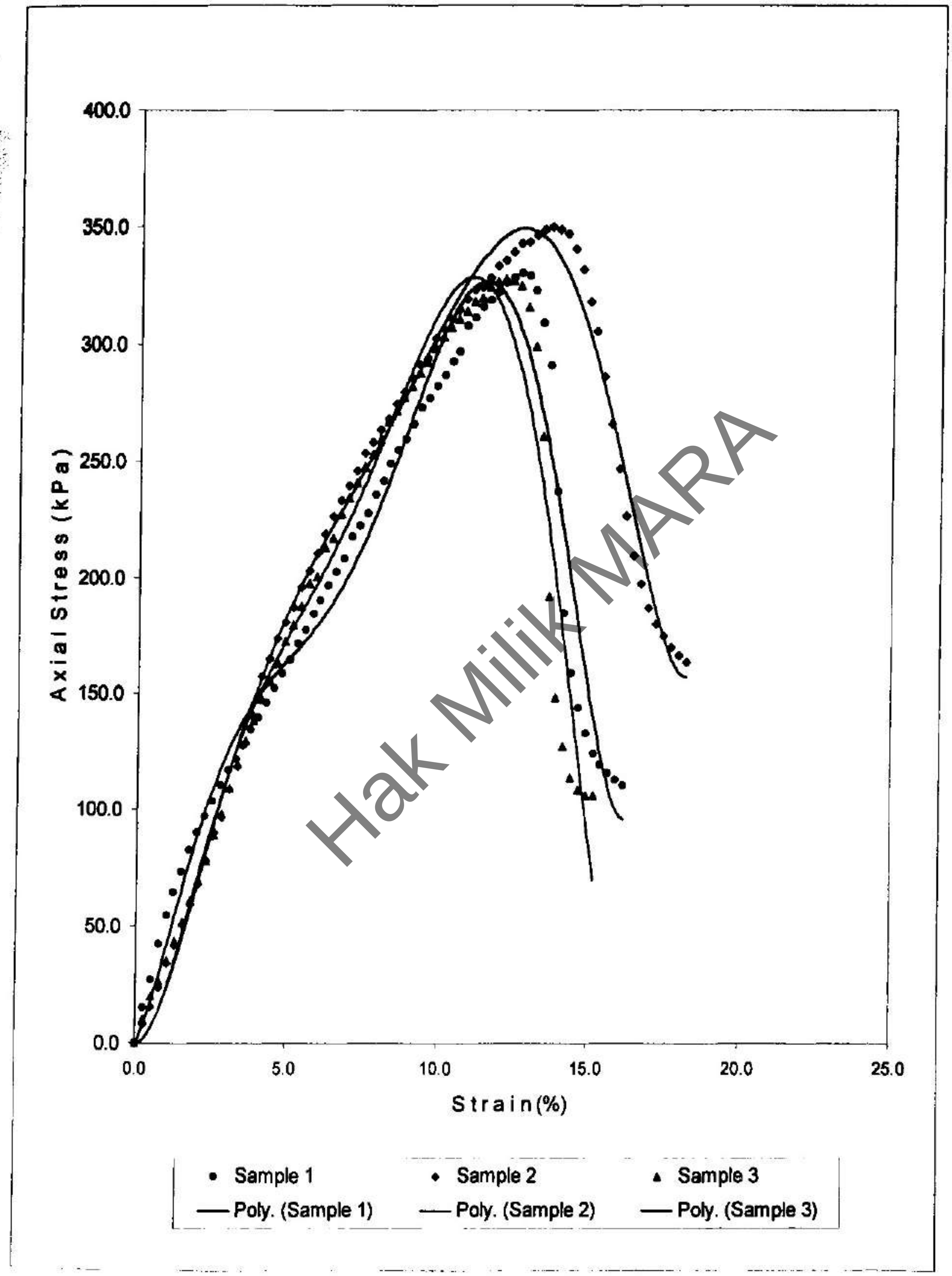

Figure F6 : Graph of axial stress vs. strain for soils + 20% BFA under 1 day curing

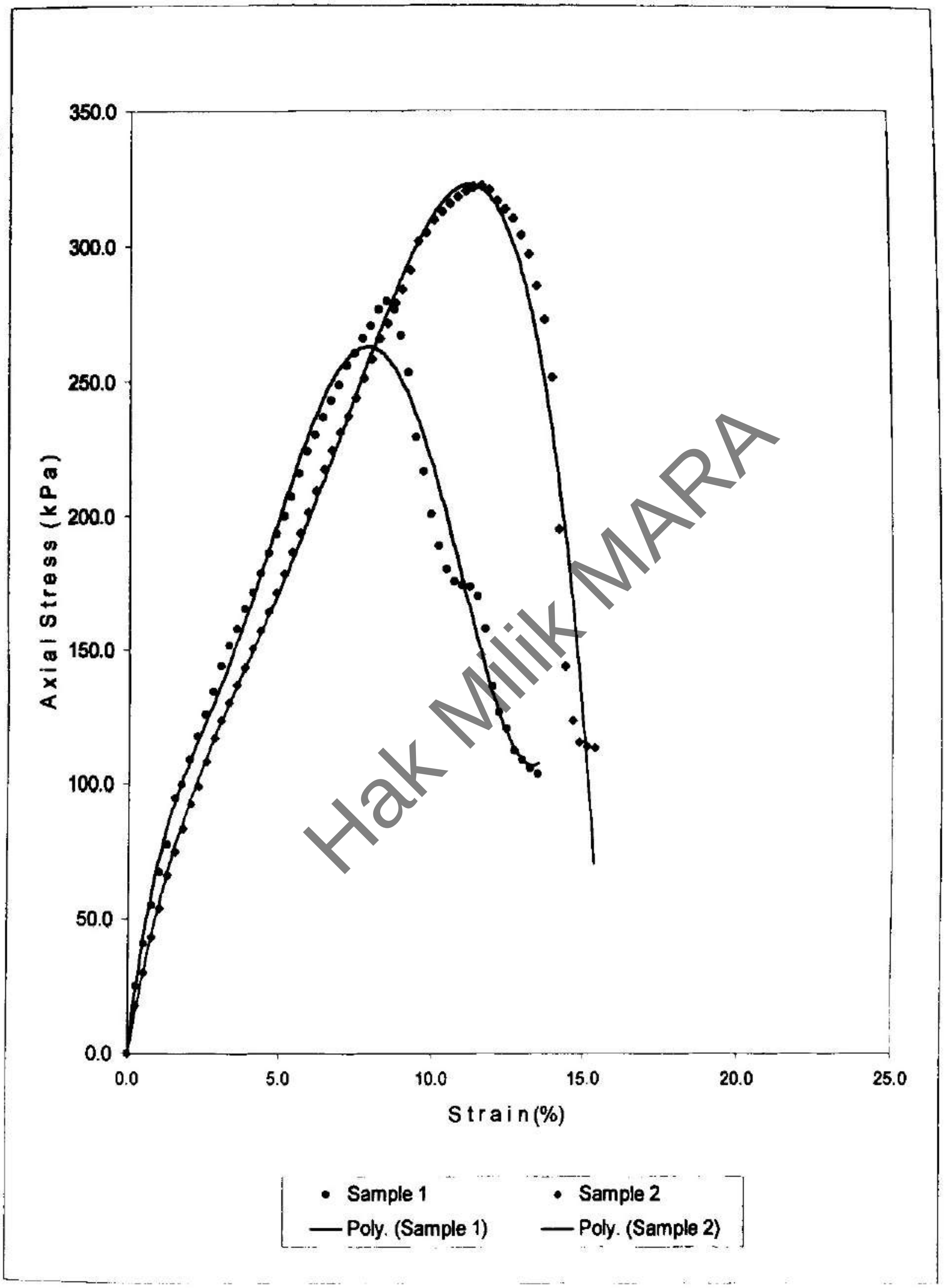

Figure F7 : Graph of axial stress vs. strain for soils +25% BFA under 1 day curing