TREATMENT AND IMPROVEMENT OF THE GEOTECHNICAL PROPERTIES OF DIFFERENT SOFT FINE-GRAINED SOILS USING CHEMICAL STABILIZATION

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Dedicated to my family

Abstract

In general, fly ash (a by-product from the burning of coal in the electric power plants) is currently in use for soil stabilization in some countries like USA, Japan, Scandinavian countries, India, and some other countries and has several recommendations and regulations. In Germany, however, fly ash is not used for soil-stabilization. The present study is an attempt to estimate how the use of fly ash (from a local electric power plant at Lippendorf, South of Leipzig city, Saxony, Germany), hydrated lime, and lime/fly ash could improve the geotechnical properties [including consistency limits, compaction properties, unconfined compressive strength (qu), elasticity modulus (E_{secant}), durability, California bearing ratio (CBR), indirect tensile strength (σt), and the hydraulic conductivity (K-value)] of three different soft fine-grained soils [tertiary clay, organic silt, and weathered soil] collected from Halle-city region, Saxony-Anhalt, Germany. One of the most significant objectives of the present study is to use the ultrasonic p-wave velocity testing as non-destructive method to evaluate the improvement of the geotechnical properties of the stabilized soils and to correlate the p-wave velocity values of the stabilized soils with the other geotechnical parameters (qu-, E_{secant}-, CBR-, and σt-value). In addition, the study is designed to evaluate the effect of lime-, fly ash-, and lime/fly ash-stabilization process on the microstructures and on the mineralogical composition of the three studied soils using scanning electron microscope (SEM)- and X-ray diffraction (XRD)-analysis, respectively. Furthermore, one of the objectives of this study is to estimate the heat flow of the soil-chemical additive mixtures and their hydration reactions using calorimetry-analysis. The results of the present study illustrated the following findings:

* The addition of lime, fly ash, and lime/fly ash to the three tested soils led to a reduction of the plasticity index and contributed to an increase in the optimum moisture content and a decrease in the maximum dry density. The moisture-density curves of the stabilized soils have typical flattened form compared to the natural soils. The qu-, E_{secant}-, CBR-, and the Vp-values increased slightly with an increment of the dry density of the untreated compacted soils (due to the compaction process) and strongly due to the addition of the chemical stabilizing agents (lime, fly ash, and lime/fly ash) whereas the formed cementitious compounds (as a result of the chemical reactions between the silica and the alumina and the additives) joined the soil particles.

* The optimum lime content (according to pH-method) of tertiary clay, organic silt, and weathered soil is 4.5, 3, and 5%, respectively. Tertiary clay is strongly reactive with lime. Unconfined compressive strength, California bearing ratio, indirect tensile strength, and p-wave velocity of the lime-stabilized tertiary clay increased continuously with the increase in lime content, because it contains a high amount of the clay particles (< 2µm = 47%) including kaolinite, montmorolinite, and halloysite where montmorolonite reacts strongly and fast with the additional lime. Both the organic silt and the weathered soil react weakly with lime where they contain relatively small amount of the clay particles including kaolinite (in weathered soil) and halloysite (in organic silt) which react slowly with the additional lime in comparison to montmorolinite in tertiary clay.

* The optimum fly ash content (according to pH-method) of tertiary clay, organic silt, and weathered soil is 16, 20, and 35%, respectively. The qu-, CBR-, σt-, and the Vp-values increased with an increase in the fly ash content in case of both the organic silt and the weathered soil. In the case of tertiary clay, the values increased with an increase in the fly ash content (from 8 to 20%) and decreased with continuous increase in the fly ash content (above 20%). The improvement of the geotechnical properties for both the organic silt and the weathered soil with fly ash is relatively smaller than the improvement for tertiary clay, at the same fly ash contents.

* The optimum lime/fly ash content (according to pH-method) of tertiary clay, organic silt, and weathered soil is (2.5%L+8%F), (2%L+12%F), and (3%L+20%F), respectively. The addition of lime and fly ash together to the three studied soils increased the qu-, CBR-, σt-, and the Vp-values strongly compared to the addition of lime and fly ash separately. Lime/fly ash-tertiary clay mixtures have qu-, CBR-, σt-, and Vp-values higher than the values of both lime/fly ash-organic silt and –weathered soil mixtures. The qu-, CBR-, σt-, and the Vp-values increased with an increase in the lime/fly ash ratio

and the maximum values of these parameters are at the optimum lime/fly ash-ratio, above the optimum lime/fly ash-ratio, the values decreased. The optimum lime/fly ash-ratio of tertiary clay and organic silt is 0.16 and 0.15, respectively (about 1 lime: 6 fly ash by weight) and the ratio of weathered soil is 0.14 (about 1 lime: 7 fly ash by weight).

* In case of the three studied stabilized soils, elasticity modulus (E_{secant}) increased and failure axial strain (ε_f) decreased as a consequence of either the separate or the joined effects of lime and fly ash contents. The E_{secant} increased and the failure axial strain decreased dramatically with the addition of both the lime and the fly ash together, especially in the case of tertiary clay. The mechanical behavior of the three studied soils was changed from ductile to brittle. This development was relatively weak in case of the weathered soil. The development of the mechanical behavior from ductile to brittle of the three stabilized soils was strong through the long-term curing except for the stabilized weathered soil. The influence of curing time was strong on the lime/fly ash-stabilization process compared to the effect on the fly ash-stabilization process, especially in the case of tertiary clay whereas the improvement of the lime/fly ash tertiary clay mixtures with the long-term curing was dramatic. The effect of long-term curing on fly ash- and lime/fly ash-stabilized weathered soil was weaker than the effect on both fly ash and lime/fly ash stabilized-tertiary clay and -organic silt.

* The correlation between qu-, CBR-, and σt-measurement (on one hand) and Vp-measurement (on the other hand) for the three tested stabilized soils showed that the variation of Vp-values of the three studied soils [due to the addition of lime, fly ash, and lime/fly ash (cured at 7 days)] is relatively similar to the variation of qu-, CBR-, and σt-values. The correlation between Vp-, qu-, and E_{secant} -measurement of the three tested lime-, fly ash-, and lime/fly ash-stabilized soils with long-term curing provided that the variation of Vp-values with curing time is similar to the variation of both the unconfined compressive strength (qu) and the elasticity modulus (E_{secant}) values. The ultrasonic testing method is a practical, simple, and fast method to evaluate lime-, fly ash-, and lime/fly ash-stabilized soil characteristics and the soil stabilization process.

* The compaction process without chemical additives can be contributed to a reduction of the hydraulic conductivity (K-value) of the three tested soils compared to the K-value of the natural soils. The K-value of organic silt was strongly affected by the compaction process compared to both the tertiary clay and the weathered soil. In the case of both fly ash- and lime/fly ash-stabilization process, the fly ash- and lime/fly ash-addition to the three tested soils resulted in an increment of the hydraulic conductivity in comparison to the untreated compacted soils. The maximum increase in K-value was at 28 days in the case of both fly ash and lime/fly ash stabilized soils (except, the K-values of fly ash-stabilized weathered soil after 7 days were higher than the K-values after 28 days). With an increase in the curing time, 56 and 180 days, the hydraulic conductivity reduced.

* The influence of lime-, fly ash-, and lime/fly ash-addition to the studied soils on the geotechnical properties is unique for each soil and chemical additive. The presence of sulfate (in case of the weathered soil) led to a formation of ettringite crystals (after the compaction) which resulted in a destruction of the compacted soil structure and, subsequently, a reduction of the strength gain development especially with the long-term curing. All the tested stabilized mixtures passed successively in the freeze-thaw durability test. Scanning electron microscope studies indicated that the microstructures of the tested soils changed due to lime-, fly ash- and lime/fly ash-stabilization process and developed with the long-term curing. Additionally, the SEM-micrograph of fly ash- and lime/fly ash-stabilized weathered soil showed rod-like crystals (ettringite) and XRD-analysis confirmed the formation of ettringite.

* The calorimetry-analysis illustrated that the high value of CaO-content and the presence of calcite mineral in the natural organic silt contributed to an acceleration of the hydration reaction of the optimum lime- and the lime/fly ash-organic silt mixtures. Finally, Lippendorf fly ash can be utilized to treat and stabilize the soft fine grained soils as economical (cheaper) alternative to Portland cement and other (expensive) chemical stabilizers. The use of fly ash for stabilization applications is an environmental solution of the problems associated with its disposal process.

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Introduction

1.1 General description

Geotechnical properties of problematic soils such as soft fine-grained and expansive soils are improved by various methods. The problematic soil is removed and replaced by a good quality material or treated using mechanical and/or chemical stabilization.

Different methods can be used to improve and treat the geotechnical properties of the problematic soils (such as strength and the stiffness) by treating it *in situ*. These methods include densifying treatments (such as compaction or preloading), pore water pressure reduction techniques (such as dewatering or electro-osmosis), the bonding of soil particles (by ground freezing, grouting, and chemical stabilization), and use of reinforcing elements (such as geotextiles and stone columns) (William Powrie, 1997).

The chemical stabilization of the problematic soils (soft fine-grained and expansive soils) is very important for many of the geotechnical engineering applications such as pavement structures, roadways, building foundations, channel and reservoir linings, irrigation systems, water lines, and sewer lines to avoid the damage due to the settlement of the soft soil or to the swelling action (heave) of the expansive soils.

Generally, the concept of stabilization can be dated to 5000 years ago. McDowell (1959) reported that stabilized earth roads were used in ancient Mesopotamia and Egypt, and that the Greek and the Romans used soil-lime mixtures. Kézdi (1979) mentioned that the first experiments on soil stabilization were achieved in the USA with sand/clay mixtures round 1906. In the 20th century, especially in the thirties, the soil stabilization relevant to road construction was applied in Europe.

In Germany, Vosteen (1998 & 1999) reported that the use of cement or lime for the stabilization of pavement bases (during the past few decades) was investigated and developed into practical construction procedures. These practical procedures have been improved and covered periodically by the technical standards for road and traffic. Fly ash-soil stabilization for road construction is applied in USA, Japan, Scandinavian countries, and some other countries like India. In Germany, fly ash-soil stabilization for road construction is not applied and there are no German recommendations and regulations for soil stabilization using fly ash. The Engineers are often faced with the problem of constructing roadbeds on or with soils (especially soft clayey and expansive soils). These problematic soils do not possess enough strength to support the wheel loads upon them either in construction or during the service life of the pavement. These soils must be, therefore, treated to provide a stable sub-grade or a working platform for the construction of the pavement. One of the strategies to achieve this is *soil stabilization*. *The soil stabilization* includes both physical stabilization [such as dynamic compaction] and chemical stabilization [such as mixing with cement, fly ash, lime, and lime by-Products, etc] (Materials & Tests Division, Geotechnical Section, Indiana, 2002).

Chemical stabilization involves mixing chemical additives (binding agents) with natural soils to remove moisture and improve strength properties of the soil (sub-grade). Generally, the role of the stabilizing (binding) agent in the treatment process is either reinforcing of the bounds between the particles or filling of the pore spaces. Most of these chemical stabilizing agents are not available in Egypt, except cement and lime which are well- known. The chemical stabilizing agents are relatively expensive compared with other methods of stabilization, so that the soil stabilization technique is an open-field of research with the potential for its use in the near future (Egyptian Code, 1995).

There are two types of chemical stabilization depending to the depth of the problematic soil and the type of geotechnical application: surface or deep stabilization. The traditional surface stabilization begins by excavating and breaking up the clods of the soil followed by the addition of stabilizing agent (additive). Soil and additives are mixed together with known amounts of water and compacted. Depths of the order of 150 to 250 mm can be strengthened by this surface method. The depth of the stabilized and strengthened zone may be increased up to one meter by using heavy equipment with appropriate modification. These methods are used extensively to stabilized bases and sub-bases of highways and airfield pavements (Nagaraj & Miura, 2001).

The following general terminology is typically used in the pavement and stabilization applications.

Additives refer to manufactured commercial products that, when added to the soil and thoroughly mixed, will improve the quality of the soil. Examples of additives include Portland cement, lime, fly ash, bitumen, and any combination of the cement, lime, and fly ash materials (Tensar Technical Note, TTN, BR10, 1998).

Chemical (Additive) soil stabilization is achieved by the addition of proper percentages of cement, lime, fly ash, bitumen, or combinations of these materials to the soil. The selection of the type and the determination of the percentage of the additive to be used are dependent upon the soil classification and the degree of improvement in soil quality desired. In general, smaller amounts of additives are required when it is simply desired to modify soil properties such as gradation, workability, and plasticity. When it is desired to improve the strength and durability significantly, larger quantities of additive are used. After the additive has been mixed with soil, spreading and compaction are achieved by conventional means (U.S. Army, Air Force & Navy, 2005).

Soil modification refers to the chemical stabilization process that results in improvements of some properties of the soil for improved constructability, but does not provide the designer with a significant increase in soil strength and durability.

A *roadway section* consists of a complete pavement system (Fig. 1) including its associated base course, sub-base course, sub-grade, and required system drainage components (Tensar Technical Note, TTN, BR10, 1998).

The *Sub-grade* refers to the *in situ* soils on which the stresses from the overlying roadway will be distributed. The *Sub-base* or *Sub-base course* and the *base* or *base course* materials are stress distributing layer components of a pavement structure.

The *Pavement structure* consists of a relatively thin wearing surface constructed over a base course and a sub-base course, which rests upon an *in situ* sub-grade. The wearing surface is primarily asphalt/concrete. The properties of all of the pavement structure layers are considered in the design of the flexible pavement system (Yoder & Witczak, 1975). They notified that the construction of long lasting, economical flexible pavement structures requires sub-grade materials with good engineering properties. The sub-grade should possess desirable properties to extend the service life of the roadway section and to reduce the required thickness of the flexible pavement structure. These desirable properties include strength, drainage, ease and permanency of compaction, and permanency of strength.

The quality of the sub-grade soil used in pavement applications is classified into 5-types (soft, medium, stiff, very stiff, and hard sub-grade) depending on unconfined compressive strength values (Das, 1994). The quality of the sub-grade soil used in pavement applications is classified into 5-types (very poor, poor to fair, fair, good, and excellent) depending on the CBR values (Bowles, 1992). The sub-grades having CBR–values of 0 – 7% are very poor and poor to fair and the sub-grades having unconfined compressive strength values of (25 – 100 KN/m^2) are soft and medium. These types are considered as unstable sub-grades and need to be stabilized, especially, in terms of pavement applications.

1.2 Review of literature

1.2.1 Lime stabilization

Several investigations were done to evaluate the soil stabilization process using lime [either CaO or $Ca(OH)_2$] (Parkash & Sridhran 1989, Wild et al. 1993, Bell 1996,

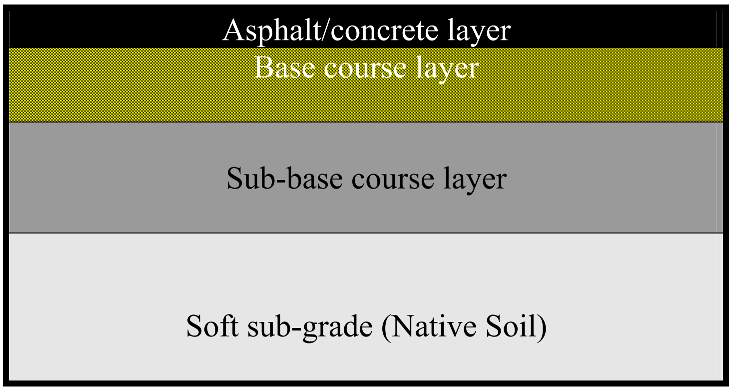


Fig. (1) A typical flexible pavement structure with its four components.

Rajasekan et al. 1997, Rajasekan & Rao 1998, Burkart et al. 1999, Qubain et al 2000, Weber 2001, Yusuf et al. 2001, Ismail 2004, and Ampera & Aydogmus 2005).

Qubain et al. (2000) incorporated the benefits of sub-grade lime stabilization, for the first time, into the design of a major interstate highway pavement in Pennsylvania. The project comprised widening and complete reconstruction of 21 Km of the Pennsylvania turnpike in somerset-county. Field explorations indicated that the sub-grade is fairly homogeneous and consists primarily of medium to stiff clayey soils. To safeguard against potential softening due to rain, lime modification has been traditionally utilized as a construction expedience for highway project with clayey sub-grade. Lime improves the strength of clay by three mechanisms: hydration, flocculation, and cementation. The first and second mechanisms occur almost immediately upon introducing the lime, while the third is a prolonged effect. Qubain et al. (2000) investigated the first and second mechanisms. Laboratory tests were performed to accurately capture the immediate benefits of lime stabilization for design. Both treated and natural clayey samples were subjected to resilient modulus and California bearing ratio testing. To prevent cementation, the lime-treated specimens were not allowed to cure. Nevertheless, they showed significant increase in strength, which, when incorporated into design, reduced the pavement thickness and resulted in substantial savings.

Witt (2002) mentioned (Geotechnik Seminar Weimar 2002) that Weber (2001) investigated the effect of both curing (storage) and degree of compaction on the loss loam stabilized using different additives. He obtained the best results under condition of moisture atmosphere storage. At the water storage condition, the tempering of the stabilized specimens delayed due to the changing of pH-value in the pores water. The reactivity of lime stabilized specimens was continuing under this water storage condition. He noticed that the variation of compaction degree of the stabilized specimens affected on the behavior of the stabilized specimens and the compaction (at the highest densities) led to brittle failure behavior.

Ismail (2004) studied materials and soils derived from the Feuerletten (Keuper) and Amaltheenton (Jura) formations along the new Nuernberg-Ingolstadt railway line (Germany). His work included petrological, mineralogical studies and scanning electron microscop-analysis. Ismail (2004) treated and stabilized these materials related to road construction using lime (10%), cement (10%), and lime/cement (2.5%/7.5%). He determined consistency limits, compaction properties, and shear- and uniaxial-strength. Ismail (2004) concluded that by increasing the optimum moisture content (%) of the treated-materials (soils mixtures), the maximum dry density (g/cm^3) decreased. The cohesion and the friction angle of the improved materials increased for all the treated mixtures. In case of the lime-treated materials, the cohesion decreased by curing time. For Feuerletten materials, uniaxial strength increased strongly using lime and cement together. For Amaltheenton, uniaxial strength increased strongly with cement alone. He also noticed that the loss of weight during freezing and thawing test was low and depended on the material type.

Ampera & Aydogmust (2005) treated Chemnitz clayey soil (A-7-6 Group) [according to American Association of State Highway and Transportation Officials (AASHTO)] using lime (2, 4, and 6%) and cement (3, 6, and 9%). They conducted compaction-, unconfined compressive strength-, and direct shear- tests on untreated and treated specimens. They concluded that the strength of cement-treated soil was generally greater than the strength of lime-treated-soil. They also reported that lime-stabilization is (in general, more tolerant of construction delay than cement-stabilization) more suitable for the clayey soils. The relationships determined from direct shear tests were similar to those determined from unconfined compressive strength tests. Thus, the results of shear strength tests showed a similar trend to that of the unconfined compressive strength tests.

1.2.2 Fly ash stabilization

Various studies were carried out in several countries like USA, Japan, etc to verify the soil stabilization process using fly ash (by-product) either class F or class C and other off-specification types of fly ash (Lee & Fishman 1992, Ferguson 1993, Turner 1997, Sahu 2001, Acosta et al. 2002, Edil et al. 2002, Şenol et al. 2002, and Thomas & White 2003).

Edil et al. (2002) conducted a field evaluation of several alternatives for construction over soft sub-grade soils. The field evaluation was performed along a 1.4 Km segment of Wisconsin state highway 60 and consisted of several test sections. By products such as fly ash, bottom ash, foundry slag, and foundry sand were used. A class C fly ash was used for one test section. Unconfined compression testing showed that 10% fly ash (on the basis of dry weight) was sufficient to provide the strength necessary for the construction on the sub-grade. Data were obtained before and after fly ash placement by testing undisturbed samples in the laboratory and by using a soil stiffness gauge (SSG) and a dynamic cone penetrometer (DCP) in the field. Unconfined compressive strength, soil stiffness, and dynamic cone penetration of the native soil before fly ash placement ranged between 100 - 150 KPa, 4 - 8 MN/m^2, and 30 - 90 mm/blow, respectively. After fly ash addition, the unconfined compressive strength reached as high as 540 KPa, the stiffness ranged from 10 to 18 MN/m^2, and the Dynamic Penetration Index (DPI) was less variable and ranged between 10 and 20 mm/blow. CBR of 32% was reported for the stabilized sub-grade, which is rated as “good” for sub-base highway construction. CBR of the untreated sub-grade was 3%, which is rated as “very poor” according to Bowles, 1992.

Acosta et al. (2002) estimate the self-cementing fly ashes as a sub-grade stabilizer for Wisconsin soils. A laboratory-testing program was conducted to evaluate the mechanical properties of fly ash alone, and also to evaluate how different fly ashes can improve the engineering properties of a range of soft sub-grade soil from different parts of Wisconsin. Seven soils and four fly ashes were considered for the study. Soil samples were prepared with different fly ash contents (i.e., 0, 10, 18, and 30%), and compacted at different soil water contents (optimum water content, 7% wet of optimum water content “approximate natural water content of the soil”, and a very wet conditions “9 to 18% wet of optimum water content”). Three types of tests were performed: California bearing ratio test, resilient modulus test, and unconfined compressive strength test. The soils selected represented poor sub-grade conditions with CBR ranging between 0 and 5 in their natural condition. A substantial increase in the CBR was achieved when soils were mixed with fly ash. Specimens prepared with 18% fly ash content and compacted at the optimum water content show the best improvement, with CBR ranging from 20 to 56. Specimens prepared with 18% fly ash and compacted at 7% wet of optimum water content showed significant improvement compared to the untreated soils, with CBR ranging from 15 to 31 (approximately an average CBR gain of 8 times). On the other hand, less improvement was noticed when the specimens were prepared with 18% fly ash and compacted in very wet condition (CBR ranging from 8 to 15).

Soil-fly ash mixtures prepared with 18% fly ash content and compacted at 7% wet of optimum water content had similar or higher modulus than untreated specimens compacted at optimum water content. Resilient modulus of specimens compacted in significantly wet conditions, in general, had lower module compared to the specimen compacted at optimum water content. The resilient modulus increased with increasing the curing time. The resilient modulus of specimens prepared at 18% fly ash content and compacted at 7% wet of optimum water content was 10 to 40% higher after 28 days of curing, relative to that at 14 days of curing. Unconfined compressive strength of the soil-fly ash mixtures increased with increasing fly ash content. Soil-fly ash specimens prepared with 10 and 18% fly ash content and compacted 7% wet of optimum water content had unconfined compressive strength that were 3 and 4 times higher than the original untreated soil specimen compacted at 7% wet of optimum water content. CBR and resilient modulus data was used for a flexible pavement design. Data developed from stabilized soils showed that a reduction of approximately 40% in the base thickness could be achieved when 18% fly ash is used to stabilize a soft sub-grade.

Şenol et al. (2002) studied the use of self-cementing class C fly ash for the stabilization of soft sub-grade of a city street in cross plains, Wisconsin, USA. Both strength and modulus-based approaches were applied to estimate the optimum mix design and to determine the thickness of the stabilized layer. Stabilized soil samples were prepared by mixing fly ash at three different contents (12, 16, and 20%) with varying water contents. The samples were subjected to unconfined compression test after 7 days of curing to develop water content-strength relationship. The study showed that the engineering properties, such as unconfined compressive strength, CBR, and resilient modulus increase substantially after fly ash stabilization. The stabilization process is construction sensitive and requires strict control of moisture content. The impact of compaction delay that commonly occurs in field construction, was evaluated, one set of the samples was compacted just after mixing with water, while the other set after two hours. The results showed that the strength loss due to compaction delay is significant and, therefore, must be considered in design and construction. CBR and resilient modulus tests were conducted and used to determine the thickness of the stabilized layer in pavement design.

Thomas & White (2003) used self-cementing fly ashes (from eight different fly ash sources) to treat and stabilize five different soil types (ranging from ML to CH) in Iowa for road construction applications. They investigated various geotechnical properties (under different curing-conditions) such as compaction, qu-value, wet/dry and freeze/thaw durability, curing time effect, and others. They reported that Iowa self-cementing fly ashes can be an effective means of stabilizing Iowa soil. Unconfined compressive strength, strength gain, and CBR-value of stabilized soils increased especially with curing time. Soil-fly ash mixtures cured under freezing condition and soaked in water slaked and were unable to be tested for strength. They also noticed that stabilized paleosol exhibited an increase in the freeze/thaw durability when tested according to ASTM C593, but stabilized Turin loess failed in the test.

1.2.3 Lime/fly ash stabilization

Several works were done to treat and stabilize various types of the problematic soils using lime and fly ash together (Nicholson & Kashyap 1993, Nicholson et al. 1994, Indraratna et al. 1995, Virendra & Narendra 1997, Shirazi 1999, Muntuhar & Hantoro 2000, Lav A. & Lav M. 2000, Cokca 2001, Consoli et al. 2001, Nalbantoglu 2001, Nalbantoglu & Tuncer 2001, Yesiller et al. 2001, Nalbantoglu & Gucbilmez 2002, Zhang & Cao 2002, Beeghly 2003, and Parson & Milburn 2003).

Nalbantoglu & Gucbilmez (2002) studied the utilization of an industrial waste in calcareous expansive clay stabilization, where the calcareous expansive soil in Cyprus had caused serious damage to structures. High-quality Soma fly ash admixture has been shown to have a tremendous potential as an economical method for the stabilization of the soil. Fly ash and lime-fly ash admixtures reduce the water absorption capacity and the compressibility of the treated soils. Unlike some of the previously published research, an increase in hydraulic conductivity of the treated soils was obtained with an increase in percent fly ash and curing time. X-ray diffractograms indicate that pozzolanic reactions cause an alteration in the mineralogy of the treated soils, and new mineral formations with more stable silt-sand-like structures are produced. The study showed that, by using cation exchange capacity (CEC) values, with increasing percentage of fly ash and curing time, soils become more granular in nature and show higher hydraulic conductivity values.

Zhang & Cao (2002) conducted an experimental program to study the individual and admixed effects of lime and fly ash on the geotechnical characteristics of expansive soil. Lime and fly ash were added to the expansive soil at 4 - 6% and 40 - 50% by dry weight of soil, respectively. Testing specimens were determined and examined in chemical composition, grain size distribution, consistency limits, compaction, CBR, free swell and swell capacity. The effect of lime and fly ash addition on a reduction of the swelling potential of an expansive soil texture was reported. It was revealed that a change of expansive soil texture takes place when lime and fly ash are mixed with expansive soil. Plastic limit increases by mixing lime and liquid limit decreases by mixing fly ash, and this decreased plasticity index. As the amount of lime and fly ash is increased, there is an apparent reduction of maximum dry density, free swell, and of swelling capacity under 50 KPa pressure and a corresponding increase in the percentage of coarse particles, optimum moisture content, and in the CBR value. They concluded that the expansive soil can be successfully stabilized by lime and fly ash.

Beeghly (2003) evaluated the use of lime together with fly ash in stabilization of soil subgrade (silty and clayey soils) and granular aggregate base course beneath the flexible asphalt layer or rigid concrete layer. He reported that lime alone works well to stabilize clay soils but a combination of lime and fly ash is beneficial for lower plasticity (higher silt content) soils. He noticed that both unconfined compressive strength- and CBR-values of treated stabilized soils (moderate plasticity “PI < 20” and high silt content “i.e. > 50%”) with lime and fly ash together are higher than the values with lime alone. Beeghly (2003) also concluded that the capillary soak of the stabilized specimens led to a loss of unconfined compressive strength (15 - 25%). Finally, lime/fly ash admixtures resulted in cost savings by increment material cost by up to 50% as compared to Portland cement stabilization.

Parson & Milburn (2003) conducted a series of tests to evaluate the stabilization process of seven different soils (CH, CH, CH, CL, CL, ML, and SM) using lime, cement, class C fly ash, and an enzymatic stabilizer. They determined Atterberg limits and unconfined compressive strengths of the stabilized soils before and after carrying out of durability tests (freeze/thaw, wet/dry, and leach testing). They reported that lime- and cement-stabilized soils showed better improvement compared to fly ash-treated soils. In addition, the enzymatic stabilizer did not strongly improve the soils compared to the other stabilizing agents (cement, lime, and fly ash).

1.3 Scope of the present work

Generally, fly ash (a by-product) is currently in use for soil stabilization in some countries like USA, Japan, Scandinavian countries, India, and some other countries and has several recommendations and regulations. In Germany, however, fly ash is not used for soil stabilization and there are no German standards and practical procedures for fly ash soil stabilization. This study is an attempt to utilize the German fly ash (by-product) for soil stabilization using a fly ash from a local electric power plant at Lippendorf, South of Leipzig city, Germany. The present study includes the use of some new methods systematically to investigate and evaluate the lime-, fly ash-, and lime/fly ash-stabilization processes including the ultrasonic p-wave velocity- (non-destructive) and the Calorimetry- method.

The first objective of this study was to determine how the use of fly ash, hydrated lime, and lime/fly ash-admixture could improve the geotechnical properties of three different soft fine-grained soils and to determine systematically the optimum hydrated lime, fly ash and lime/fly ash contents to treat and stabilize these soils using the pH-method. The three studied soils were selected from the Halle-city region: one is inorganic tertiary clay which has high plasticity; the second type of soil is organic silt having relatively low plasticity and high organic content compared to tertiary clay; the third soil is a weathered soil from Muschelkalk formation and has low pH-value and contains gypsum crystals. Why these three different soils were selected? The reason is that these soils are problematic concerning the geotechnical properties and applications and represent three different soil-characters to evaluate the influence of the soil type on the chemical stabilization process with different chemical additives. To achieve the first objective, a laboratory-testing program was conducted where compacted soil-lime, -fly ash, and -lime/fly ash mixtures were prepared with several lime-, fly ash- and lime/fly ash-contents. Subsequently, the mixtures were tested to determine their geotechnical properties, especially, related to road construction. The laboratory program included compaction, consistency, unconfined compressive strength, California bearing ratio, indirect tensile strength, hydraulic conductivity and velocity of ultrasonic P-wave tests, which was performed on natural, untreated compacted, and treated stabilized soils. The second objective of the study was a try to establish a new method (fast and practical) to evaluate the behavior of the treated stabilized soils and the chemical stabilization processes using the measurement of the P-waves ultrasonic velocity as non-destructive method. Steel and concrete are commonly evaluated using ultrasonic testing in civil engineering application, where established procedures and standards are available for ultrasonic evaluation of these materials (Mclntire 1991). Conversely, there are no established procedures and standards available for ultrasonic evaluation of chemical stabilized soils. A very small number of studies (Yesiller et al., 2001) focused on the use of ultrasonic p-wave velocity to evaluate stabilized-fly ash and -soil. Yesiller et al. (2001) reported that “non-destructive test methods” such as ultrasonic testing can provide a fast and simple alternative approach for analyzing chemical stabilized mixtures, and they pointed to the need to develop criteria and guidelines to incorporate the ultrasonic test method into mixture design. This means that the evaluation of stabilized soils using ultrasonic method is a new field and needs in-depth studies and further development. The third aim of this study was to verify the hydraulic conductivity of the treated stabilized soils, since the available data concerning the hydraulic conductivity (K-value) of fly ash and lime/fly ash treated stabilized soils are very limited. Townsend & Kylm (1966), Brandl (1981), Nablantoglu & Tuncer (2001), and Nablantoglu & Gucbilmez (2002) noted an increment of the hydraulic conductivity while others (Terashi et al., 1980 and Locat et al., 1996) reported a decrement of the hydraulic conductivity. This means that hydraulic conductivity studies of fly ash and lime/fly ash treated stabilized soils are widely variable, problematic and need more investigations.

The fourth objective was to evaluate the long-term stability or durability of the fly ash-, and lime/fly ash-stabilized soils under different weathering-circumstances including water soaking, freezing/thawing, and drying/wetting tests. Durability of treated stabilized soils is a problematic geotechnical parameter and previous studies on durability are very restricted and the results are variable and require to further studies. Turner (1997) evaluated the wet-dry and freeze-thaw durability of different low plasticity clayey soils treated with both class F and C fly ash. Although the unconfined compressive strength values of the compacted soil-fly ash mixtures were high and satisfactory, the wet-dry and freeze-thaw durability tests results exhibited a weight loss of more than 14% meaning that the mixtures failed in durability test. The fifth aim of this study was to investigate and evaluate the lime/fly ash ratio for lime/fly ash-stabilization process. Some standards (U.S. army, air force, and navy, 2005) reported that ratios lie between 1 : 3, 1 : 4, and 1 : 5 or about 0.33, 0.25, and 0.20, respectively, where the best lime/fly ash ratio will yield highest strength and best durability results. Other studies (Virendra & Narendra, 1997) obtained the best results (maximum unconfined compressive strength and CBR-values) of alluvial soil stabilized with 15% of lime and fly ash in proportion of 1 : 3 by weight or about 0.33 lime of fly ash weight. A question arises as to whether lime/fly ash ratio is constant or variable based on the type and the chemical composition of both the fly ash and the soil? The sixth objective of this study was to determine the effect of soil type, organic matter, and curing time (7, 28, 56, and 180 days) on the chemical stabilization process. The seventh aim was to estimate the effect of lime-, fly ash-, and lime/fly ash-stabilization process on the microstructure (micro-fabric) of the treated stabilized soils using scanning electron microscope (SEM). The eighth objective was to estimate lime-, fly ash-, and lime/fly ash-soil mixtures and their hydration reactions through the heat-flow calorimetry using a high-sensitive calorimeter developed by Poellmann et al., 1991. Finally, the increasing trend towards electrical power generation through coal combustion has aggravated the problems associated with the disposal of the fly ash "by-product" (Nablantolu, 2001), so that, use of fly ash as a stabilizing agent plays an important environmental and economical role. Economically, use of fly ash as a chemical additive in chemical soil stabilization and for the geotechnical applications is cheaper than Portland cement and other (expensive) chemical stabilizers.

2 Materials and methods

2.1 Lime

2.1.1 Background and composition

Lime can be used either to modify some of the physical properties and thereby improve the quality of soil or to transform the soil into a stabilized mass, which increases its strength and durability. The amount of lime additive will depend upon either the soil to be modified or stabilized. Generally, lime is suitable for clay soils with PI ≥ 20% and > 35% passing the No.200 sieve (0.074 mm). Lime stabilization is applied in road construction to improve sub-base and sub-grades, for railroads and airports construction, for embankments, for soil exchange in unstable slopes, for backfill, for bridge abutments and retaining walls, for canal linings, for improvement of soil beneath foundation slabs, and for lime piles (Anon, 1985 & 1990). Lime stabilization includes the use of burned lime products, quicklime and hydrated lime (oxides and hydroxides, respectively), or lime by-products (codel) (TTN, 1998).

The improvement of the geotechnical properties of the soil and the chemical stabilization process using lime take place through two basic chemical reactions as follow: I) *Short-term reactions* including cation exchange and flocculation, where lime is a strong alkaline base which reacts chemically with clays causing a base exchange. Calcium ions (divalent) displace sodium, potassium, and hydrogen (monovalent) cations and change the electrical charge density around the clay particles. This results in an increase in the interparticle attraction causing flocculation and aggregation with a subsequent decrease in the plasticity of the soils. II) *Long-term reaction* including pozzolanic reaction, where calcium from the lime reacts with the soluble alumina and silica from the clay in the presence of water to produce stable calcium silicate hydrates (CSH), calcium aluminate hydrates (CAH), and calcium alumino-silicate hydrates (CASH) which generate long-term strength gain and improve the geotechnical properties of the soil. These hydrates were observed by many researchers (Diamond et al., 1964; Sloane, 1965; Ormsby & Kinter, 1973; and Choquette et al., 1987). The use of lime for soil stabilization is either in the form of quicklime (CaO) or hydrated lime $Ca(OH)_2$. Agricultural lime or other forms of calcium carbonate, or carbonated lime, will not provide the necessary reactions to improve sub-grade soils mixed with lime.

In the present study, hydrated lime (according to DIN 1060) was used. The chemical composition of the hydrated lime illustrated in table 2.5. *Hydrated lime* is calcium hydroxide, $Ca\ (OH)_2$. It is produced by reacting quicklime (CaO) with sufficient water to form a white powder. This process is referred to as *slaking*.

$$\text{High calcium quicklime + water} \longrightarrow \text{Hydrated lime + Heat}$$

$$CaO + H_2O \longrightarrow Ca(OH)_2 + \text{Heat}$$

Hydrated lime is used in most of the lime stabilization applications. Quicklime represents approximately 10% of the lime used in lime stabilization process. Other forms of lime sometimes used in lime stabilization applications are dehydrated dolomitic lime, monohydrated dolomitic lime, and dolomitic quicklime (TTN, 1998). Calcium oxide (quick lime) may be more effective in some cases, however the quick lime will corrosively attack equipment and may cause severe skin burns to personnel (Muntohar & Hantoro, 2000). The Addition of the hydrated lime $Ca(OH)_2$, *in situ* or in laboratory, is either as slurry formed by the slaking of quicklime, or as dry form (dry powder). In the present study, the addition of the hydrated lime is in a dry form. In general, all lime treated fine-grained soils exhibit decreased plasticity, improved workability and reduced volume change characteristics. However, not all soils exhibit improved strength characteristics. It should be emphasized that the properties of soil-lime mixtures are dependent on many factors such as soil type, lime type, lime percentage, and curing conditions (time, temperature, and moisture) (U.S. Army, Air Force, and Navy, 2005). Table 2.1 shows a general recipe of soils treatment using lime- and cement-stabilization modified after German standard of FGSV, 1997 (Leaflet for soil stabilization and soil improvement; soil stabilization using binding agents, 1997).

Table (2.1) General recipe of lime- and cement-stabilization modified after the German standard (Leaflet for soil stabilization and soil improvement; soil stabilization using binding agents, 1997). X = suitable (X) = conditional suitable - = unsuitable

Soil type			**Binding agent**	
			Quicklime and hydrated lime after DIN (10 60)	Cement after DIN (11 64)
Coarse grain size soil			-	X
Mixed grain size soil			(X)	X
Fine grain size soil	silt	Low plasticity	X	X
		Medium P.	X	(X)
		High P.	X	-
	clay	Low plasticity	X	(X)
		Medium P.	X	-
		High P.	X	-
Organic soil	silt		X	(X)
	clay		X	-

2.2 Fly ash

2.2.1 Background

Fly ash is a by-product (waste material) of burring coal at electric power plants. It is a fine residue composed of unburned particles that solidifies while suspended in exhaust gases. Fly ash is carried off in stack gases from a boiler unit, and is collected by mechanical methods or electrostatic precipitators. Fly ash is composed of fine spherical silt size particles in the range of 0.074 to 0.005 mm (Ferguson, 1993). Fly ash collected using electrostatic precipitators usually has finer particles than fly ash collected using mechanical precipitators. Fly ash is one of the most useful and versatile industrial by-products (Collins & Ciesielski, 1992).

When geotechnical Engineers are faced with problematic soils (such as clayey or expansive soils), the engineering properties of those soils may need to be improved to make them suitable for construction. Waste materials such as fly ash or pozzolanic materials [pozolanic materials “pozzolans” are a source of silica and alumina with high surface area (Choquette el al., 1987)] have been used for soil improvement. Recent investigations reported that fly ash is a potential material to be utilized for soil improvement (Muntohar & Hantoro, 2000).

Fly ash is generated in huge quantities (more than 65 million metric tons per year in the USA) as a by-product of burning coal at electric power plants (Ferguson, 1993). The potential for using fly ash in soil stabilization has increased significantly in many countries (for example in Wisconsin, USA) due to the increased availability and the introduction of new environmental regulation (NR 538, Wisconsin Administrative Code) that encourage the use of fly ash in geotechnical applications since it is environmentally safe (Şenol et al., 2002).

Classification and chemical reactions of fly ash:

Fly ash is classified into two classes: F and C. Class F fly ash (non-self-cementing fly ash) is produced from burning anthracite and bituminous coals and contains small amount of lime (CaO) to produce cementitious products. An activator such as Portland cement or lime must be added. This fly ash (pozzolans) has siliceous and aluminous material, which itself possesses little or no cementitious value but it reacts chemically (in the presence of moisture) with lime at ordinary temperature to form cementitious compounds (Chu et al., 1993).

Class C fly ash (self-cementing fly ash) is produced from lignite and sub-bituminous coals (low-sulfur subbituminous coals), and usually contains significant amount of lime (Cockrell & Leonard, 1970). This type (class C) is self-cementing because it contains a high percent of calcium oxide (CaO) ranging from 20 to 30%.

Formation of cementitious material by the reaction of lime with the pozzolans (Al_2O_3, SiO_2, and Fe_2O_3) in the presence of water is known as *hydration* of fly ash. The hydrated calcium silicate (gel) or calcium aluminate (fibrous), as cementitious material, can join inert materials together. The pozzolanic reactions for soil stabilization are as follow (TRB, 1987):

$CaO + H_2O \Longrightarrow Ca(OH)_2 + \text{Heat}$

$Ca(OH)_2 \Longrightarrow Ca^{++} + 2\,(OH)$

$Ca^{++} + 2\,(OH) + \underset{\text{(silica)}}{SiO_2} \Longrightarrow \underset{\text{(gel)}}{CSH}$ “Calcium silicate hydrate“

$Ca^{++} + 2\,(OH) + \underset{\text{(alumina)}}{Al_2O_3} \Longrightarrow \underset{\text{(fibrous)}}{CAH}$ “Calcium aluminate hydrate”

In case of the class C fly ash, the lime present (in the fly ash) reacts with the siliceous and aluminous materials “pozzolans” (in the fly ash). A similar reaction can occur in class F fly ash, but lime must be added because of the low lime content of the fly ash class F (Şenol et al., 2002).

2.2.2 Source and composition

The fly ash used in the present study is from a local electric power plant at Lippendorf, South of Leipzig city, Saxony, Germany. Physical and chemical properties of the Lippendorf fly ash are summarized in Table 2.2 along with physical properties of class C and class F fly ashes. The fly ash has a powdery texture. It is light grey in colour, which indicates high calcium oxide content (Meyers et al., 1976). The types of coal of this fly ash are lignite and subbituminous.

Table (2.2) Physical properties, chemical composition and classification of fly ashes

Fly ash	Classification after ASTM Standard C 618	Gs g/cm^3	Specific surface area m^2/g	Percent Fines (%)	Moisture content (%)	LOI (%)	Lime CaO (%)	Other oxides ($SiO_2 + Al_2O_3 + Fe_2O_3$) (%)	Sulfur Trioxide (SO_3) (%)
Type 1	Class C**	-	-	-	3	6	24.3*	50	5
Type 2	Class F**	-	-	-	3	6	8.7*	70	5
Lippendorf	Off-spec.	2.8	1.1	81.5	0.11	0.22	38.3	46.00	9.1

Notes:
Gs = Specific gravity
LOI = Loss of ignition
* After Ferguson et al., 1999 ** After ASTM Standard C 618

The chemical composition of fly ash is one of the most important indicators of material quality for various applications. Detailed chemical composition of Lippendorf fly ash is summarized in Table 2.3, along with typical chemical composition of class C and F fly ashes.

Table (2.3) Chemical composition of Lippendorf fly ash and typical chemical composition of both class C and F fly ashes.

Chemical elements	Lippendorf fly ash (%)	Typical class **C** (%)	Typical class **F** (%)
SiO_2	32.20	39.9	54.9
Al_2O_3	11.20	16.7	25.8
Fe_2O_3	2.60	5.8	6.9
CaO	38.3	24.3	8.7
MgO	4.10	4.6	1.8
SO_3	9.10	3.3	0.6
LOI	0.22	6	6

LOI = Loss of ignition

Lippendorf fly ash is classified as off-specification fly ash (ASTM C618). It has high calcium oxide (CaO) content (38.3%) and an organic content of 0.11%. Lippendorf fly ash is off-specification, since the (SiO_2+Al_2O_3+Fe_2O_3) content is below 50% and the sulfur trioxide (SO_3) content exceeds 5%. It is close to class C. The silicon dioxide (SiO_2) content of Lippendorf fly ash (32.2%) is below the typical amount of class C fly ash (39.9%). Al_2O_3 content of Lippendorf fly ash (11.2%) is below the typical amount of class C fly ash (16.7%). Both Fe_2O_3 and MgO contents of Lippendorf fly ash (2.6 and 4.1%, respectively) are close to typical class C fly ash (5.8 and 4.6%, respectively). The sulfur trioxide (SO_3) content (9.1%) is higher compared to typical (SO_3) content of class C and class F ashes (3.3 and 0.6%, respectively).

The mineral composition of Lippendorf fly ash is illustrated in Appendix 1a. The main component is calcium alumino-silicate glass (amorphous). Lippendorf fly ash contains free lime CaO, quartz SiO_2, periclase MgO, anhydrite Ca (SO_4), etc. (see Appendix 1a). It does not contain mineral phases (such as tricalcium aluminate C_3A and tricalcium silicate C_3S) which have fast hydration reactions.

2.2.3 Index- and compaction-properties

The specific gravity and the specific surface area of Lippendorf fly ash is showed in Table 2.2. The specific gravity of Lippendorf fly ash (2.8 g/cm^3) is high relative to the typical values of the fly ashes (specific gravity typically range from 2.11 to 2.71) (Chu & Kao, 1993).

In general, as mentioned before, Fly ash collected using electrostatic precipitators, like Lippendorf fly ash usually has finer particles than fly ash collected using mechanical precipitators. Figure 2.1 illustrates the particle size distribution of Lippendorf fly ash (according to DIN 18 123). Grain size analysis of fly ash was carried out through combination of dry sieving- and sedimentation-analysis. Sedimentation-analysis was conducted by granulometer CILAS 920 using the fine fraction < 400 µm (resulted from the dry sieving) in suspension in an appropriate (Isopropanol) liquid.

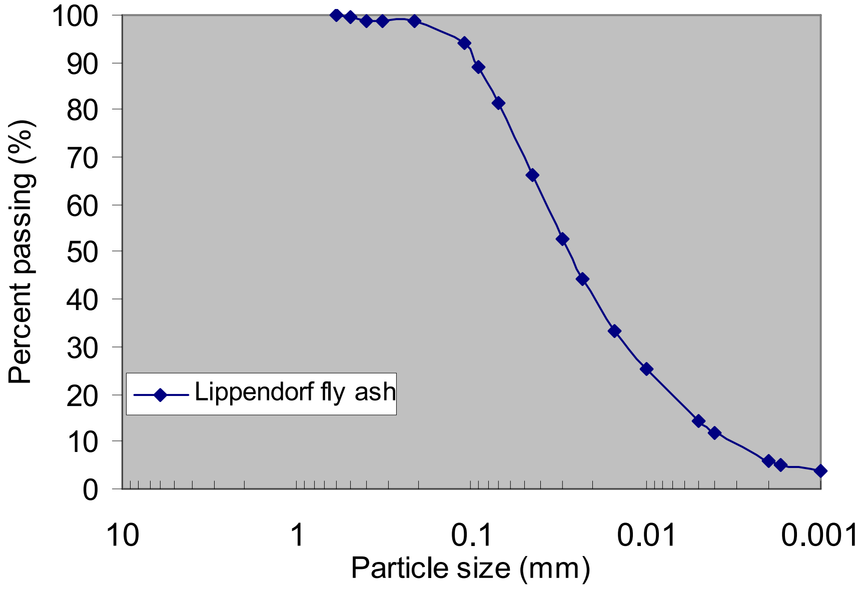


Fig. (2.1) Particle size distribution of Lippendorf fly ash

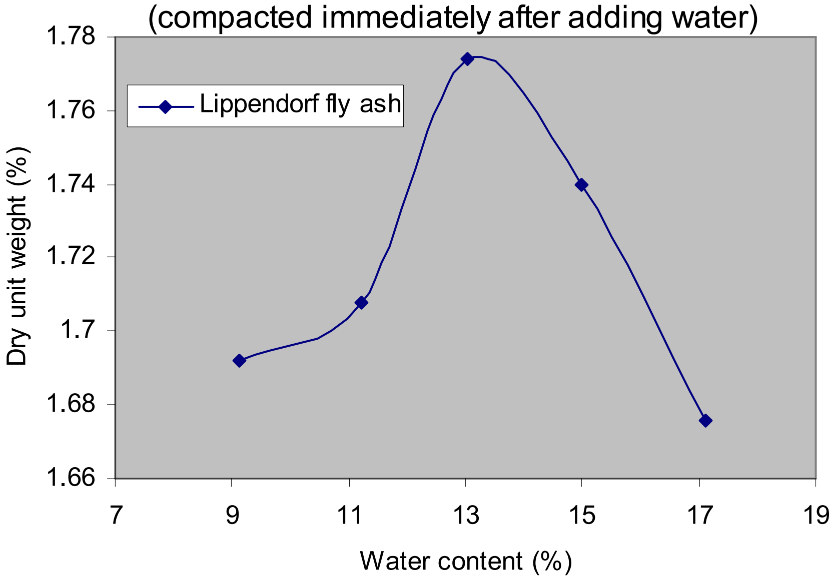


Fig. (2.2) Compaction curve of Lippendorf fly ash (compacted immediately after adding water)

Specific gravity of both the fly ash and the tested soils was measured using Multipycnometer (Quantachrome) with helium gas. Specific surface area of both the fly ash and the tested soils was measured using Micromeritic-Instrument (FlowSorb II 2300) with two mixed gases (30% nitrogen and 70% helium).

Compaction properties of the fly ash using the standard proctor compaction procedure (DIN 18 127) are shown in Figure 2.2. The compaction curve is more bell-shaped curve relative to the typical bell-shaped curves of the fine grained studied soils (Fig. 2.8). The maximum dry unit weight of Lippendorf fly ash is (1.77 g/cm³). Generally, a decrease in the organic content (0.11%) leads to an increase in the maximum dry unit weight. The optimum water content of Lippendorf fly ash is (13.03%).

2.3 Natural fine grained soils

2.3.1 Sources and Geology

Three fine grained soils were considered for the testing geotechnical laboratory program. The locations where the soils were collected are shown in Figure 2.3 (a & b).

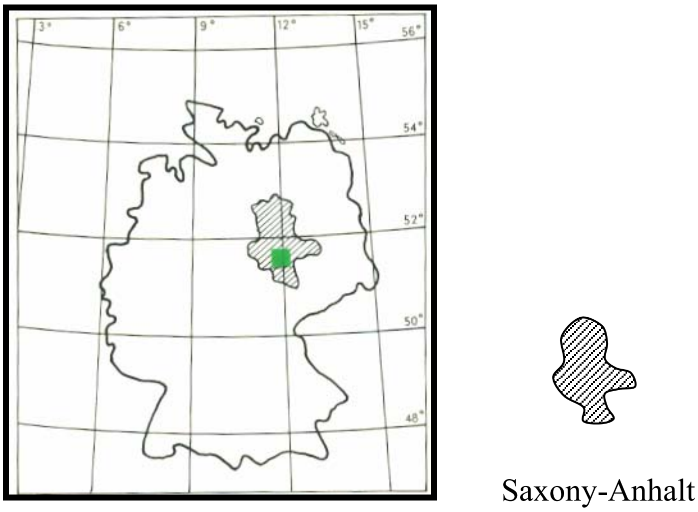


Fig. (2.3, a) Location map of Germany illustrates the studied area 🟩

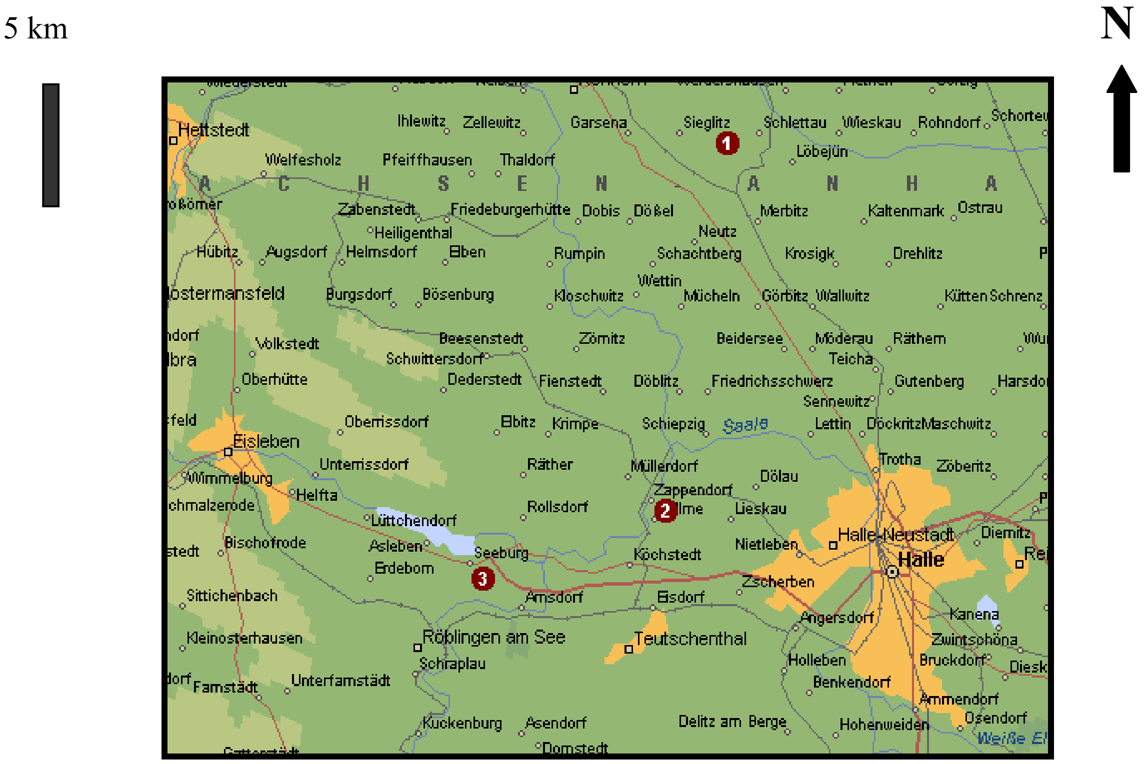


Fig. (2.3, b) Location map of the study specimens after Microsoft Encarta Worldatlas,1998.
1 Tertiary clay 2 Weathered soil 3 Organic silt.

1-Tertiary clay is collected from old sand/gravel quarry area near Sieglitz village (about 20 Km at the NW of Halle city, Saxony-Anhalt, Germany). It is a marine sediment, Lower Oligocene age (Rupel-succession, its thickness about 38 m).

2-Weathered soil of Muschelkalk Formation is collected from old quarry between Zappendorf and Koellme village, (about 10 Km at the NW of Halle city, Saxony-Anhalt, Germany). It is a residual soil, Triassic (Muschelkalk) age.

3-Quaternary organic silt is collected from ehemaliger Salziger See area, about 15 Km at the East of Eisleben city (along the road B80 between Halle and Eisleben city, Saxony-Anhalt, Germany). It is lake sediment, recent age.

2.3.2 Soil index properties

Index- and compaction- properties and classification of the studied soils are summarized in Table 2.4. Index properties include consistency limits (LL, PL, PI = LL-PL, and Ws), consistency index (Ic), percent fines (percentage passing No.200 sieve), specific gravity (Gs), loss of ignition (LOI), and specific surface area (surface area/mass ratio). Shrinkage limit (Ws) was determined only for the natural tertiary clay. Shrinkage limit (Ws) of natural tertiary clay (containing kaolinite, montmorillonite, and halloysite) equals to 12.38%. This indicates that the degree of expansion for natural tertiary clay is medium “marginal” (according to

Gromko, 1974).The consistency index (Ic) according to DIN 18 122-1) is calculated from the following equation: Ic = (LL-w)/(LL-PL) = (LL-w)/PI where w = natural water content Ic-value of tertiary clay, organic silt, and weathered soil is 0.75, 0.83, and 0.97, respectively. According to Fecker and Reik, 1996, the teriary clay is at the boundary between soft and stiff, and both the organic silt and the weathered soil are classified as stiff.

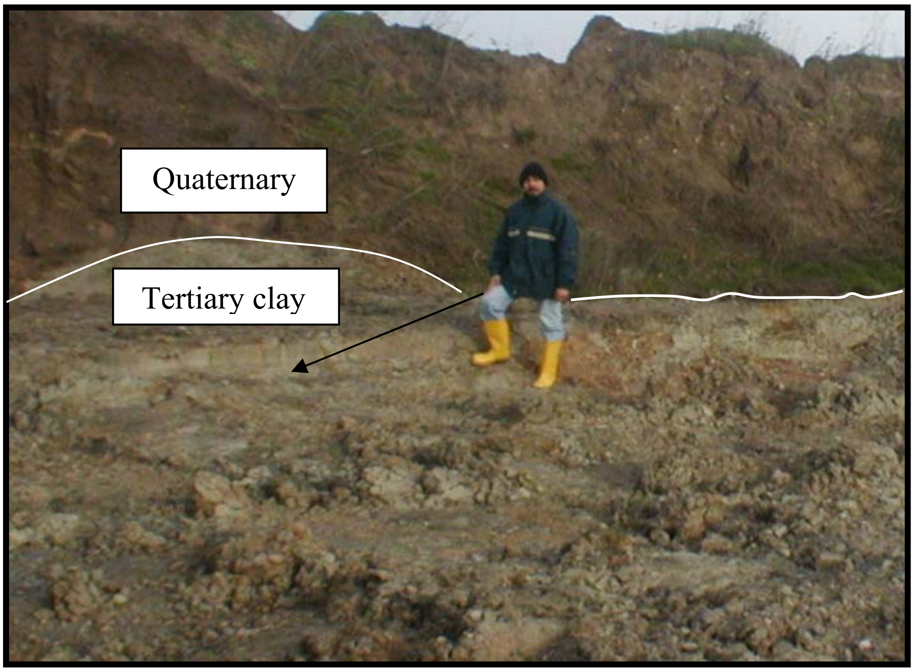


Fig. (2.4) Tertiary clay from old Sand/Gravel quarry area (Lower Oligocene, Rupel-Succession) near Sieglitz village.

Fig. (2.5) Quaternary organic silt from ehemaliger Salziger See-area, at the East of Eisleben city.

Fig. (2.6) Weathered soil of Muschelkalk Formation is collected from old quarry between Zappendorf and Koellme villages (NW of Halle city).

The specific surface area plays a significant role in the reactivity between clay particles and chemical additives. Clay mineral particles have plate like form with high specific surface area (Craig, 1997). According to Hardt, 1985, the specific surface area of montmorillonite, illite, and kaolinite is 800, 100, and 10m²/g, respectively.

According to the unified soil classification system (USCS):

Tertiary clay sample is classified as CH (Inorganic clays of high plasticity).

Organic silt sample is classified as OH (Organic silt of high plasticity).

Weathered soil is classified as MH (Inorganic clayey silt and very fine sand of high plasticity)

According to (DIN 18 196):

Tertiary clay sample is classified as TA (Distinct plasticity clay).

Organic silt sample is classified as OU (Silt with organic matter).

Weathered soil is classified as UA (Distinct plasticity clayey silt).

According to the highway research board classification (H.R.B)/ (AASHTO):

Tertiary clay sample is classified as A-7-6 Group, the general rating as sub-grade is fair to poor.

Organic silt sample is classified as A-7-6 Group, the general rating as sub-grade is fair to poor.

Weathered soil is classified as A-7-5 Group, the general rating as sub-grade is fair to Poor.

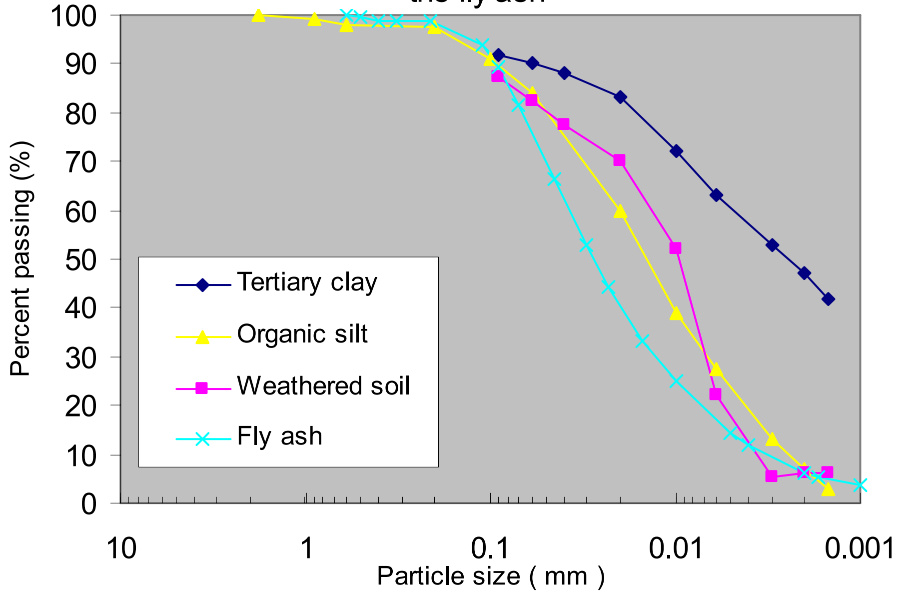
The particle size distribution of the studied soils and fly ash (after DIN 18 123) are presented in Figure 2.7. Tertiary clay is finer than the other soils (percent fines = 91%). Organic silt and weathered soil contain 87 and 85% fines, respectively. All the other index properties of the natural soils are presented in Table 2.4.

Table (2.4) Index properties and classification of the natural soil soils.

Soil Type	LL (%)	PL (%)	PI (%)	Percent Fines (%)	Gs g/cm^3	Specific surface area m²/g	LOI (%)	Classification			Wn (%)	OMC (%)	γd kN/m³	MDD kN/m³
								USCS	AASHTO	DIN 18196				
Clay	61.5	28.6	32.9	91	2.65	28.53	3	CH	A-7-6	TA	36.7	23.8	13.4	14.17
Silt	50.0	29.2	20.8	87	2.55	13.56	6.4	OH	A-7-6	OU	29.8	27.61	14.2	14.27
W.S.	63.7	32.4	31.4	85	2.64	8.45	3.4	MH	A-7-5	UA	37.5	25.74	13.3	14.73

Notes:
LL = Liquid limit
PL = Plastic limit
PI = Plasticity index (PI = LL-PL)
Percent Fines = Percentage passing No.200 sieve (0.074 mm)
Gs = Specific gravity
LOI = Loss of ignition, at 550°C, after (DIN 18 128)
Wn = Natural water content
γd = Natural dry unit weight
MDD = Maximum dry density (Proctor dry density)
OMC = Optimum moisture content (in Proctor test)
W.S = Weathered soil
USCS = Unified soil classification system
AASHTO = American Association of State Highway and Transportation Officials
DIN = (Deutsche Institut fuer Normung) German Institute of standard specification.

Fig. (2.7) Particle size distributions of the studied soils and the fly ash



2.3.3 Chemical and mineralogical analysis

I- Chemical analysis: X-ray fluorescence spectrometry (XRF)

X-ray fluorescence analysis of both natural soils and chemical additives was investigated using XRF-technique (type of the instrument is SRS 3000 Siemens).

Chemical compositions of the three different studied natural soils, fly ash, and the hydrated lime are summarized in Table 2.5.

II- Mineralogical analysis: X-ray powder diffraction (XRD)

X-ray powder diffraction technique with Cu Kα radiation (type of instrument is Siemens D 5000 diffractometer with a generator operating at 40KV, 30mA and with Ni-Filter) was used to determine the mineralogical composition of the natural soils and the chemical additives. Soil mineralogy provides the basis for understanding the basic mechanisms of chemical stabilization. It also helps to identify types of clay minerals in the studied soils in order to determine the ability of the soils to expand. The presence of some clay minerals, like high-activity smecite clays as montmorillonite in the soil is a good indication of the swell potential. These clays cause problems of excessive expansive characteristics, which lead to much damage to the structures built in and on them (Nicholson, et al., 1994). The mineral composition of natural tertiary clay, organic silt, and weathered soil is shown in Appendixes (1b, 2, and 3, respectively) and in Table 2.6.

Table (2.5) Chemical composition of both natural soils and chemical additives (Hydrated lime and Fly ash)

Chemical Elements	Tertiary Clay (%)	Weathered soil (%)	Organic Silt (%)	Hydrated lime (%)	Fly ash (%)
SiO_2	54.9	52.1	41.70	0.503	32.20
Al_2O_3	18.50	21.3	11.80	0.279	11.20
Fe_2O_3	5.73	6.26	4.24	0.225	2.60
$\Sigma =$	79.13	79.66	57.74	1.007	46.00
SiO_2/Al_2O_3	2.97	2.45	3.53	1.802	2.88
CaO	2.41	1.34	18.70	73.4	38.3
MgO	1.86	1.65	1.79	0.556	4.10
Na_2O	0.140	0.179	0.36	0.0321	0.160
K_2O	2.98	3.66	2.60	0.0737	0.267
MnO	0.0261	0.0132	0.105	0.0359	0.113
TiO_2	0.974	1.78	0.711	0.0185	0.99
P_2O_5	0.116	0.0919	0.444	0.0288	0.0758
SO_3	4.23	1.92	4.64	0.191	9.10
LOI	6	6.8	12.8	24.6	0.22

LOI = Loss of ignition, at 1000 °C.

Table (2.6) Mineralogical composition of the studied natural soils.

Soil type	Arrangement of the minerals according to the majority (from primary to secondary components)
Tertiary clay	Quartz, Kaolinite, Montmorillonite, Muscovite, and Halloysite
Organic silt	Calcite, Quartz, Muscovite, and Halloysite
Weathered soil	Quartz, Muscovite, Kaolinite, and Gypsum

2.3.4 Compaction characteristics and geotechnical properties

Compaction characteristics:

Compaction curves corresponding to standard proctor effort (see Fig. 2.11) were determined for each soil following the procedure in (DIN 18 127). Typical bell-shaped compaction curves were obtained (see Fig. 2.8). The maximum dry unit weights and optimum water contents are summarized in Table 2.4. Organic silt has the highest optimum water content (27.61%). It is relatively near its natural water content (29.8%), and its maximum dry unite weight is 17.27 KN/m^3. The curve has narrow bell-shaped form compared to the other two soils. Weathered soil has the highest maximum dry unit weight (14.73 KN/m^3), which reflects the relative larger fraction of coarse particles in the sample. The optimum water content is 25.74%. Tertiary clay has the lowest maximum dry unite weight (14.17 KN/m^3) and its optimum water content is 23.8%.

Geotechnical properties:

Unconfined compression tests (qu-tests) were conducted on each soil according to (DIN 18 136). The qu-values are summarized in Tables 2.7 and 2.8. The qu-value of natural undisturbed (with natural water content and natural unit weight) and of untreated compacted samples (with optimum water content) was measured according to DIN 18 136. Unconfined compressive strength (qu-value) of natural tertiary clay, organic silt, and weathered soil is 40.90, 77.71, and 42.34 KN/m², respectively. These values indicate that tertiary clay, organic silt, and weathered soil are classified as soft, medium, and soft sub-grade, respectively (Das, 1994). After compaction (with optimum water content and without chemical additive) the qu-value of tertiary clay, organic silt, and weathered soil increased to 131.21, 136.91, and 173.25 KN/m², respectively and the three soils are classified as stiff sub-grade (Das, 1994).

California bearing ratio tests (CBR-tests) were conducted on each soil following the methods described in TPBF-StB, part B 7.1. The CBR-values are summarized in Tables 2.7

and 2.8. CBR-value of compacted soils (with natural water content) and of untreated compacted (with optimum water content) samples, using standard proctor effort, was measured. California bearing ratio (CBR-value) of tertiary clay, organic silt, and weathered soil (compacted with natural water content) is 2.1, 2.1, and 2.6%, respectively. These values indicate that tertiary clay, organic silt, and weathered soil are classified as very poor sub-grade (Bowles, 1992). After compaction (with optimum water content and without chemical additive) the CBR-value of tertiary clay, organic silt, and weathered soil increased to 4.6, 3.2, and 5.4%, respectively and the three soils are classified as poor to fair sub-grade (Bowles, 1992).

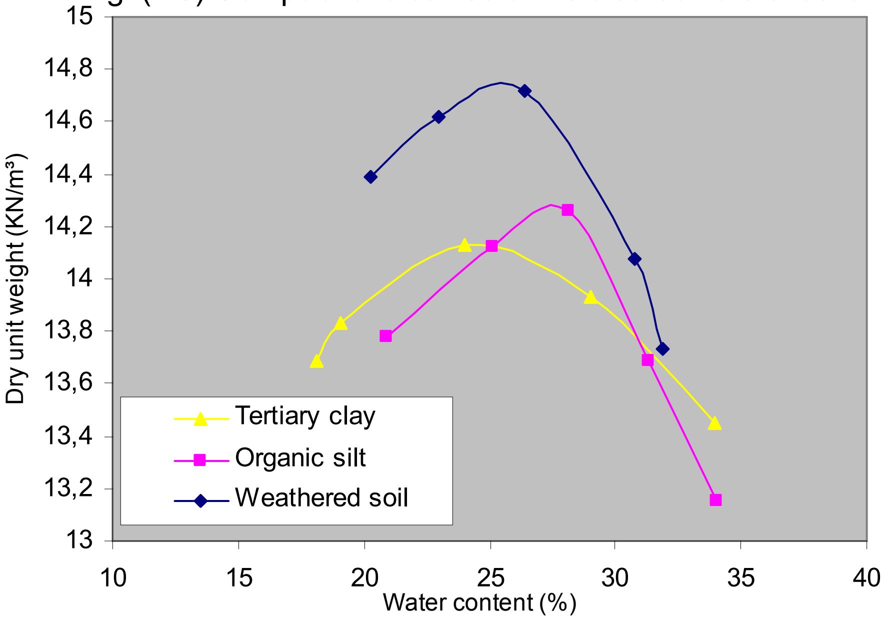


Fig. (2.8) Compactions curves of the studied natural soils.

Soil type	MDD (KN/m³)	OMC (%)	qu-value KN/m^2	CBR-value %	K-value m/sec	Vp-value m/sec
Tertiary clay	14.17	23.8	40.90	2.1	1.9 E-11	643
Weathered soil	14.73	25.74	42.34	2.6	3.2 E-11	700
Organic silt	14.27	27.61	77.71	2.1	5.5 E-07	424

Table (2.7) illustrated compaction characteristics, qu-, CBR-, k-, and p-waves velocity (Vp)-values of the studied natural soils.

Soil type	qu-value kN/m²		Quality after qu (Das, 1994)		CBR-value (%)		Quality after CBR (Bowles, 1992)	
	Natural soil	Untreated compacted	Natural soil	Untreated compacted	Natural soil	Untreated compacted	Natural soil	Untreated compacted
Tertiary Clay	40.90	131.21	soft	stiff	2.1	4.6	Very poor	Poor to fair
Weathered Soil	42.34	173.25	soft	stiff	2.6	5.4	Very poor	Poor to fair
Organic Silt	77.71	136.91	medium	stiff	2.1	3.2	Very poor	Poor to fair

Table (2.8) Description of the quality of natural and untreated compacted soils after Das, 1994 and Bowles, 1992.

2.4 Test procedures

2.4.1 Unconfined compressive strength test

Unconfined compressive strength tests were conducted according to DIN 18 136. A photograph of a soil specimen subjected to unconfined compression is shown in Figure 2.10. Unconfined compressive strength for natural soils, for untreated compacted, and for treated stabilized specimens is determined by using computerized triaxial instrument without application of the cell pressure (σ_3 = zero). The dimensions of the tested specimens (for natural specimens) are 120 mm height and 95 mm diameter and the dimensions of untreated compacted and treated stabilized specimens are 120 mm height and 100 mm diameter.

The maximal vertical strain according to DIN 18 136 is equal to 20% from the maximal height of the tested specimen, so that, the maximal vertical strain = 20 / 100 * 120 mm = 24 mm.

Unconfined compressive strength (qu–value) of the tested specimens is either at the failure of the specimen or at the maximal vertical strain (ε) equal to 20% of the original height of the soil specimen (DIN 18 136). The speed of deformation (strain rate), according to DIN 18 136, is at least equal to 1% of the maximal height of the tested sample = 1/100 * 120 mm = 1.2 mm/min. In the present study, the strain rate for both undisturbed natural soils, untreated compacted, and for treated stabilized specimens was equal to 0.2% of the maximal height of the tested specimens. The strain rate is 120 mm * 0.2 /100 = 0.24 mm/min, according to DIN 18 136 for the cemented and stabilized specimens.

Fig. (2.9) Temperature-humidity chamber.

Fig. (2.10) Computerized triaxial cell to measure the unconfined compressive strength (σ_3 = zero).

After compaction the specimens were extruded, sealed in polyethylene paper, and stored in ≥ 98% relative humidity at 40°C ± 2 for 7 days curing (for soil-lime mixtures) in computerized temperature-humidity chamber (Fig. 2.9). For soil-fly ash and soil-lime/fly ash mixtures, the specimens are stored in ≥ 98% relative humidity and at 25°C ± 2 for 7 days curing. After 7 days curing period, all treated stabilized specimens were tested in unconfined compression at strain rate of 0.24 mm/min.

The general relationship between unconfined compressive strength and the quality of the sub-grade soils used in pavement applications (Das, 1994) is as follow:

Qu-values	Quality of sub-grade
25 – 50 kpa	soft sub-grade
50 – 100 kpa	medium sub-grade
100 – 200 kpa	stiff sub-grade
200 – 380 kpa	very stiff sub-grade
› 380 kpa	hard sub-grade

2.4.2 CBR test

CBR test was conducted according to TPBF-StB part B 7.1, 1988 for natural soils compacted with natural water content, for untreated compacted specimens with optimum water content, and for all treated stabilized specimens at the optimum water contents using standard proctor effort (Fig. 2.11) and computerized CBR-instrument (Fig. 2.12). The dimensions of the tested specimens are 125 mm height (H) and 150 mm diameter (Ø).

The test was conducted with annular surcharge mass of 5 Kg. A natural soil specimen (tertiary clay) undergoing a CBR-test is shown in Figure 2.12.

$$CBR = P/Ps * 100 \ (\%)$$

Where: P is plunger-load in N/mm² for tested soil.

Ps is plunger-load in N/mm² for standard soil (see Fig. 2.13).

Fig. (2.11) Standard proctor instruments.

Fig. (2.12) Computerized CBR-instrument, CBR-test conducted on tertiary clay specimen.

For soil-lime mixtures, the specimens were left in the mold, sealed using polyethylene paper, and left to cure for 7 days at 40°C ± 2 and ≥ 98% relative humidity prior to testing. For soil-fly ash and soil-lime/fly ash mixtures, the specimens were left in the mold, sealed using polyethylene paper, and cured for 7 days (at 25°C ± 2 and ≥ 98% relative humidity) prior to testing.

The general relationship between CBR-values and the quality of the sub-grade soils used in pavement applications (Bowles, 1992) is as follow:

CBR - values	Quality of sub-grade
0 – 3 %	very poor sub-grade
3 – 7 %	poor to fair sub-grade
7 – 20 %	fair sub-grade
20 – 50 %	good sub-grade
› 50 %	excellent sub-grade

The sub-grades having (0 – 7%) CBR–values are very poor and poor to fair. They are considered as unstable sub-grades and need to be stabilized, especially, in terms of pavement applications.

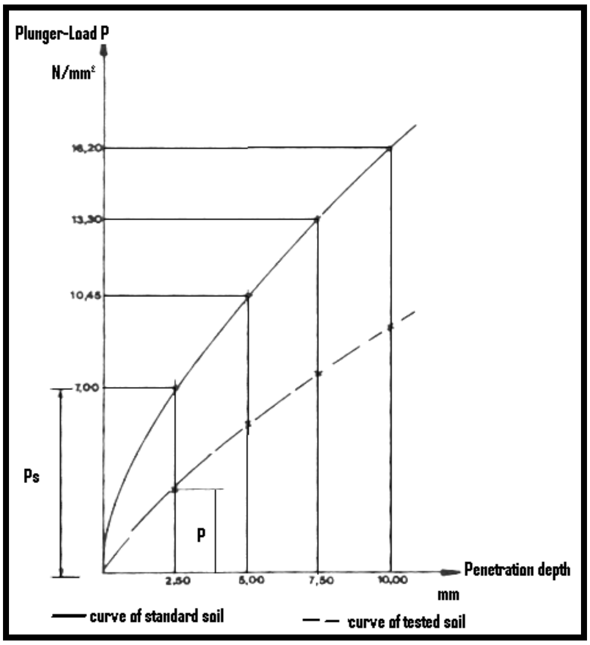


Fig. (2.13) CBR curves after TPBF-StB, part 7.1, 1988.

2.4.3 Indirect tensile strength test

Indirect tensile strength tests were conducted loosely based on the International Society for Rock Mechanics (ISRM), for all the treated stabilized specimens at the optimum water contents using manual tensile strength-instrument with two horizontal steel-plates (with felt-streaks) instead of the standard steel-plates as in Brazilian test. The dimensions of each plate are 12 cm long and 8 cm width (Fig. 2.14). The dimensions of the tested stabilized specimens are 100 mm thickness “height” (H) & 100 mm diameter (Ø), where H/Ø ratio is equal to 1.

Tensile strength (σt) of the specimens was calculated according to ISRM by the following formula:

$\sigma t = 2P / \pi * D * L \quad = 0.636 \, P / D * L$

Where: σt = tensile strength (MPa)

P = Load at failure (N)

D =Diameter of the tested specimen (mm)

L = Thickness “height” measured at the center (mm) (see Fig. 2.16).

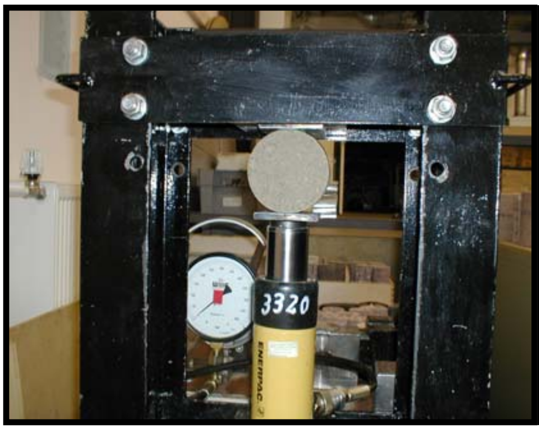


Fig. (2.14) Indirect tensile strength instrument.

Fig. (2.15) Triaxial cell to measure K-value.

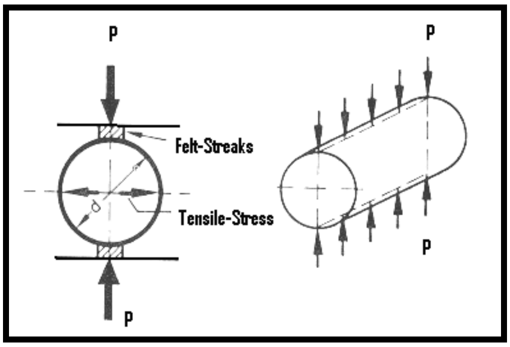


Fig. (2.16) illustrated the principles of tensile measurement after Maidl, 1988.

2.4.4 Hydraulic conductivity test

Hydraulic conductivity tests were conducted following the procedure in Laboratory test of DIN 18 130-1. A photograph of a triaxial cell, in which the K-value of the studied specimens was measured, is shown in Figure 2.15. K-value for natural soils, for untreated compacted specimens, and for treated stabilized specimens is determined by using triaxial cell with constant hydraulic gradient equal to 50 and with cell pressure, in-pressure, and out-pressure equal to 5, 2.6, and 2 bars, respectively. The dimensions of the tested specimens (for natural soil specimens) are 120 mm height and 95 mm diameter and the dimensions of untreated compacted and treated stabilized specimens are 120 mm height and 100 mm diameter. The analysis of the measured values (volume of water, time, and pressure) of the test and the calculation of K-value were conducted using GGU-software program.

2.4.5 Ultrasonic P-waves velocity test

P-wave velocities were measured using ultrasonic measurement instrument, USME-C. The measurement system consisted of two P-wave transducers and a pulser-receiver. One of the P-wave transducer is for transmitting waveforms, 64 KHz frequency, the other is for receiving waveforms, 40 - 700 KHz frequency. The P-wave velocity of the samples was determined using the through-transmission inspection method with the transmitting transducer placed on one end of the sample and the receiving transducer placed on the opposite end of the sample (Fig. 2.17).

This arrangement is typically used for highly attenuating materials such as concrete. The velocity of the p-waves was obtained as the quotient of the travel pass “x” (the height of the specimens) and the travel time of the P-waves “*ta”* (P-waves velocity “Vp” = x/*ta*) (Yesiller et al., 2001). The height of the specimens is 0.12 m. The travel time was obtained from ultrasonic measurements (with resolution of 0.1 µs). It is defined as the first arrival time of the waves at the receiving transducer. A typical waveform obtained in the tests is presented in Figure 2.18. The first arrival time of the P-wave is indicated on the waveform.

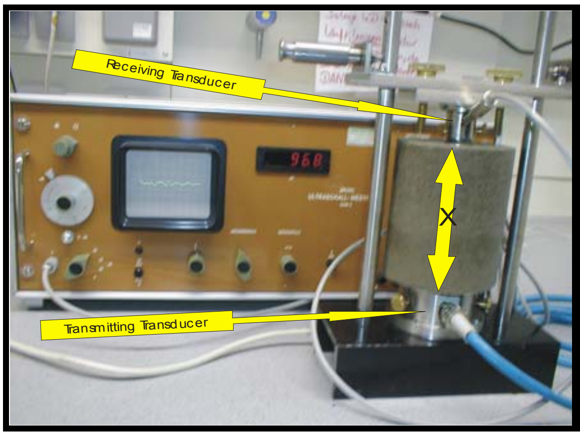


Fig. (2.17) Ultrasonic instrument.

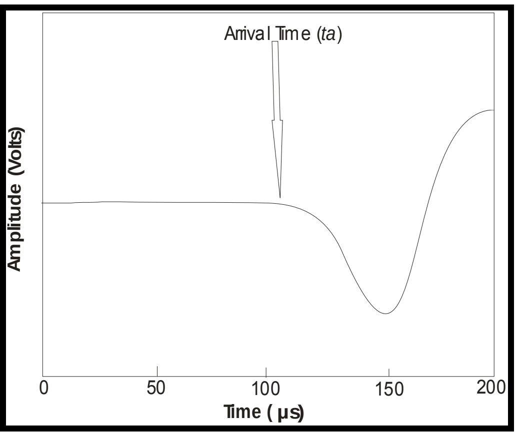


Fig. (2.18) Typical waveform.

2.5 Procedures of the stabilization process in the laboratory

2.5.1 Procedures of lime-stabilization process

1- Preparation of the soil sample; soil sample was dried in the air and then it was put into the oven at 50°C for 24 hours. The dried soil was crushed in crushing-machine.

2- Determination of the reactivity of the soil for lime stabilization; the reaction of a soil-lime or a soil-cement mixture is important for stabilization and design methodology. It should be based on an increase in the unconfined compression strength test data (Materials and Tests Division, Geotechnical Section, Indiana, 2002). To determine the reactivity of the soils for lime-stabilization, the following steps were followed:

A- Mixing at least 5% lime by dry weight of the natural untreated soil.

B- Carrying out of a standard proctor test to determine both optimum water content and maximum dry density for the lime-soil mixture.

C- Construction of a compacted sample of the lime-soil mixture at optimum water content and maximum dry density.

D- Curing of the compacted sample for 48 hours at 50°C in a constant temperature chamber and at humidity $\geq 98\%$.

E- Measurement of an unconfined compressive strength (qu-value) for the treated lime-stabilized compacted sample by using triaxial instrument with σ_3 = zero.

In case of the reactive soil (with lime): the strength gain of the treated sample must be at least 50 Psi = 350 KN/m^2 greater than the strength of the natural untreated soil (Materials and Tests Division, Geotechnical Section, Indiana, 2002).

In the present study, according to the results of reactivity tests of the three tested soils mixed with 5% hydrated lime, the unconfined compression strength of tertiary clay, organic silt and weathered soil is 1050.96 KN/m^2 (reactive), 249.68 KN/m^2 (not reactive), and 256.10 KN/m^2 (not reactive), respectively.

3- Determination of the optimum lime content for lime stabilization using *Eades and Grim pH-test, 1966.*

First: measurement of the pH-value for natural soil and lime samples separately.

Second: addition of sufficient amount of lime to soils to produce a pH of 12.4 or equal to a pH-value of lime itself. A graph is plotted between pH-value (at Y-Axis) versus lime percentage (at X-Axis). Optimum lime content should be determined corresponding to maximum pH-value of lime-soil mixture.

Procedures:

A- Representative samples of air-dried, minus No.40 sieve (0.42 mm), were dried in oven (60 °C). About 20 gm for each sample is weighed to the nearest 0.1 gm and poured into 150-ml (or larger) plastic bottle with screw top.

B- It is advisable to set up eight bottles with lime percentages of 1, 2, 3, 4, 5, 6, 7, and 8. This will ensure, in most cases, that the percentage of lime required can be determined in one hour. Weigh the lime to the nearest 0.01 gm and add it to the soil. Shake to mix soil and dry lime.

C- Add 100 ml of CO_2 – free distilled water to the bottles.

D- Shake the soil-lime and water until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.

E- Shake the bottles for 30 seconds every 10 minutes.

After one hour, transfer part of the slurry to a plastic beaker and measure the pH-value.

F- Record the pH-value for each of the lime-soil mixtures. The lowest percent lime that gives a pH-value of 12.40 is the percent required to stabilize the soil. If the pH-value does not go beyond 12.30 and 2 percents lime give the same reading, the lowest percent which gives a pH-value of 12.30 is that required to stabilize the soil. If the highest pH-value is 12.30 and only 1 percent lime gives a pH-value of 12.30, additional test bottles should be started with larger percentages of lime.

In the present study, optimum lime content of tertiary clay, organic silt and weathered soil is 4.5, 3, and 5%, respectively (see Appendix 6 and Fig. 2.19 in Appendix 7).

4- Preparation of the treated lime-stabilized compacted samples with optimum lime content at maximum dry density and optimum water content.

A- After the optimum lime content has been estimated, a standard proctor test (according to DIN 18 127 and TPBF-StB, part B 11.5, 1991) for the lime-soil mixture with optimum lime content was conducted to determine the maximum dry density and the optimum water content and to plot the water content-dry density curve.

B- Then, construction of compacted sample of optimum lime-soil mixture through homogenously mixing of both optimum lime content and the dried soil (2 minutes). This is followed by addition of the optimum water content on the dry mixture and mixing homogenously (2 minutes). The mixture should be allowed to cure no less than 1 hour and no more than 2 hours in a sealed container, followed by remixing (2 minutes) before compaction.

C- Construction of two other samples with increasing lime content (+ 2 and + 4% above the optimum lime content) to study the effect of an increase in the percentage of lime content (above the optimum lime content) on the geotechnical properties. After the compaction, the specimens were prepared to measure the geotechnical properties. The samples should be wrapped securely with polyethylene paper, laid in a plastic bag to prevent moisture loss, and cured in a constant temperature and humidity chamber (at 40 °C ± 2°C and at humidity ≥ 98 %) to 7 days. Finally, the geotechnical parameters of the different stabilized lime-soil mixtures were measured.

2.5.2 Procedures of fly ash-stabilization process

1- Preparation of soil sample, soil sample was dried in the air then it was put into oven at 50 °C for 24 hours. The dried soil was crushed in crushing-machine.

2- Determination of the optimum fly ash content for fly ash stabilization using the basis of *Eades and Grim* pH-test, 1966. The determination of the fly ash content using the same procedures taken for lime stabilization, except that, five bottles, instead of eight in lime stabilization, were prepared with fly ash percentages of 8, 12, 16, 20, and 25.

In the present study, optimum fly ash content of tertiary clay, organic silt and weathered soil is 16, 20, and 35%, respectively (see Appendix 6 and Fig. 2.20 in Appendix 8).

3- Mixtures were prepared with optimum fly ash content and with other fly ash contents, under and above the optimum fly ash content, as follows: 8, 12, 16, 20, and 25% on dry weight basis with the soil to determine the effect of a decrease and an increase in the fly ash contents compared to the optimum fly ash content.

4- Carrying out of the standard proctor test (DIN 18 127) to determine both the maximum dry density and the optimum water content for each mixture. Compaction of the samples should be carried out after mixing with water (2-hours delay) to simulate the typical duration between mixing and compaction that occurs in the field (Şenol et al., 2002).

5- Construction of compacted samples of the all fly ash-soil mixtures at their maximum dry densities and their optimum water contents, through homogeneous mixing of fly ash contents with the dried soils. This followed by addition of the optimum water content on the dry mixture and homogenously mixing (2 minutes). The mixture must be cured for 2-hours, where the mixture should be compacted after 2-hours delay from the mixing with water.

6- After the compaction, each specimen should be wrapped with polyethylene paper, laid in plastic bags, and allowed to cure for 7 days in a humidity-temperature chamber (at ≥ 98% humidity and at temperature 25 °C ± 2). The optimum fly ash-soil mixture is cured for 28, 56, and 180 days in a humidity-temperature chamber (at the same conditions of humidity and temperature) to determine the influence of the curing time factor on the geotechnical properties and on the process of fly ash-stabilization.

2.5.3 Procedures of lime/fly ash-stabilization process

1- Preparation of soil sample; as the procedure 1 in the fly ash-stabilization process.

2- Determination of the optimum lime/fly ash content for lime/fly ash-stabilization using *Eades and Grim* pH-test, 1966. The method of lime/fly ash content determination is similar to the lime content method, except five bottles were used instead of eight in lime content method. The bottles were prepared at small lime percent (optimum lime content minus 2% for both the tertiary clay and the weathered soil and optimum lime content minus 1% for the organic silt) with different fly ash contents as follows: 8, 12, 16, 20, and 25%.

In the present study, optimum lime/fly ash content of tertiary clay, organic silt and weathered soils according to pH-test is (2.5%L/8%F), (2%L/12%F), and (3%L/20%F), respectively (see Appendix 6 and Fig. 2.21 in Appendix 9).

3- Other mixtures (at optimum fly ash content with different lime percentages as follow: optimum lime-2%, optimum lime, and optimum lime+2% in the case of both tertiary clay and weathered soil and optimum lime-1%, optimum lime, and optimum lime+2% in the case of organic silt) were prepared to estimate the effect of the increase in the lime content and the lime/fly ash ratio on the lime/fly ash-stabilization process.

4- Conduction of a standard proctor test (DIN 18 127) to determine both the maximum dry density and the optimum water content for each mixture. The mixture should be allowed to cure no less than 1 hour and no more than 2 hours in a sealed container, followed by remixing (2 minutes) before compaction.

5- Construction of compacted samples, for each mixture, at the maximum dry density and the optimum water content.

6- After the compaction, each sample should be wrapped securely to prevent moisture loss and cured in a constant temperature-humidity chamber (at 25°C ± 2 and at relative humidity ≥ 98%) for 7 days. Some of the lime/fly ash mixtures were cured for 28, 56, and 180 days to estimate the influence of curing time factor on the geotechnical properties and on the lime/fly ash-stabilization process. The lime/fly ash-soil mixtures should remain securely wrapped until testing.

Figure (2.22) illustrates a flowchart including the present geotechnical laboratory program to evaluate lime-, fly ash-, and lime/fly ash-stabilization process of the studied soils.

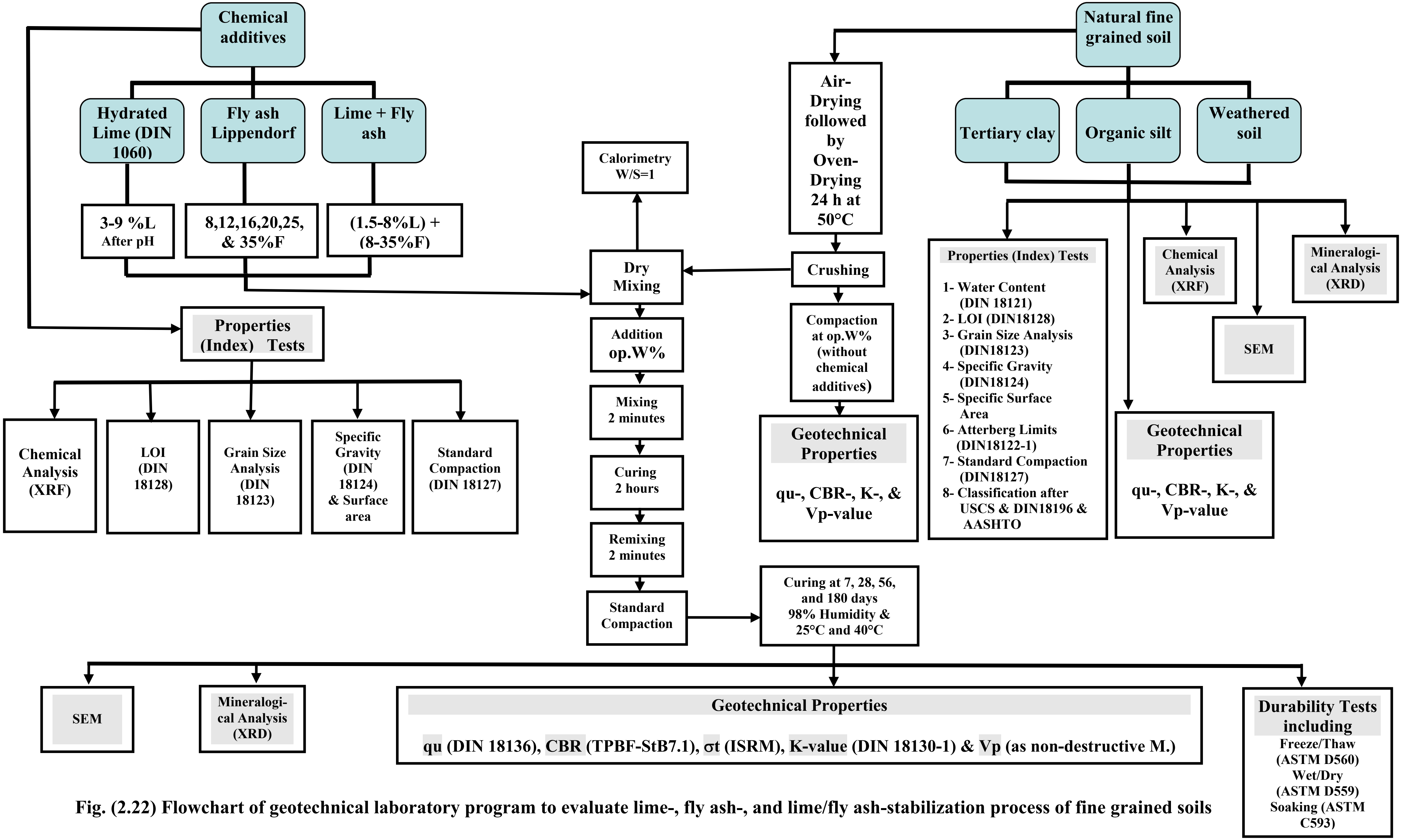


Fig. (2.22) Flowchart of geotechnical laboratory program to evaluate lime-, fly ash-, and lime/fly ash-stabilization process of fine grained soils

3 Results: Plasticity, compaction, and unconfined compressive strength (qu)

3.1 Plasticity

Atterberg limits (Plastic limit “PL”, Liquid limit “LL”, and Plasticity index “PI” = LL-PL) play an important role in soil identification and classification. These parameters indicate to some of the geotechnical problems such as swell potential and workability. One of the important and principle aims of this study was to evaluate the changes of liquid, plastic limits, and plasticity index with addition of lime, fly ash, and lime/fly ash together to the three studied soils. To achieve this objective, Atterberg limits test (including PL, LL, and PI) was conducted on both natural soils and different lime-, fly ash-, and lime/fly ash-soil mixtures, for the three studied soils according to consistency test of DIN 18 122-1.

PL and LL of the different soil-lime, -fly ash, -lime/fly ash mixtures were determined after 1 day curing according to DIN 18 122-1 and TPBF-StB, part B 11.5, 1991. The results of the tests were calculated using GGU-software program. Figure 3.1 (a, b, & c) illustrates both liquid limit (LL), plastic limit (LL), and plasticity index (PI) of tertiary clay, organic silt, and weathered soil, respectively with different chemical additives. In general, the figures show that the addition of lime, fly ash, and lime/fly ash together, in case of the three studied soils, led to an increase in both the liquid limit and the plastic limit. The increase of the plastic limit is greater than that of the liquid limit. This resulted in a reduction of the plasticity index [PI = LL-PL] (see Appendixes 10, 11, & 12).

3.2 Compaction

The geotechnical properties of soil (such as swell potential, compressive strength, CBR, permeability, and compressibility etc) are dependent on the moisture and density at which the soil is compacted. Generally, a high level of compaction of soil enhances the geotechnical parameters of the soil, so that achieving the desired degree of relative compaction necessary to meet specified or desired properties of soil is very important (Nicholson et al., 1994). The aim of the proctor test (moisture-density test) was to determine the optimum moisture contents of both untreated compacted and treated stabilized soil-mixtures. Standard proctor test was carried out according to DIN 18 127. The test-results were calculated using GGU-software program. Figure 3.2 (a, b, & c) illustrates the moisture-density relationship of tertiary clay, organic silt, and weathered soil, respectively.

The curves show the physical changes that occur (after 2-hr delay) during lime, fly ash, and lime/fly ash treatment. In general, the addition of lime, fly ash, and lime/fly ash together, for the three studied soils, led to an increase in the optimum moisture content and to a decrease in the maximum dry density. The bell-shaped compaction curves of the three studied soils were converted to flattened-shaped compaction curves (see Appendixes 10, 11, & 12).

In the case of tertiary clay, the maximum dry density due to fly ash addition decreased with continuous increase in fly ash content. In the case of organic silt and weathered soil, the maximum dry density decreased with the fly ash addition, and with continuous increase in fly ash content, it increased relatively. This may be due to the different mineralogical composition, where tertiary clay contains montmorolonite which reacts fast with the chemical additives (lime & fly ash) in comparison to kaolinite and halloysite in both weathered soil and organic silt, respectively (Kézdi, 1979). In the case of tertiary clay, the addition of lime and fly ash together led to a more decrease of the maximum dry density compared to the addition of lime and fly ash separately. In the case of both weathered soil and organic silt, the addition of lime and fly ash together resulted in a more decrease of the maximum dry density compared to the addition of fly ash alone.

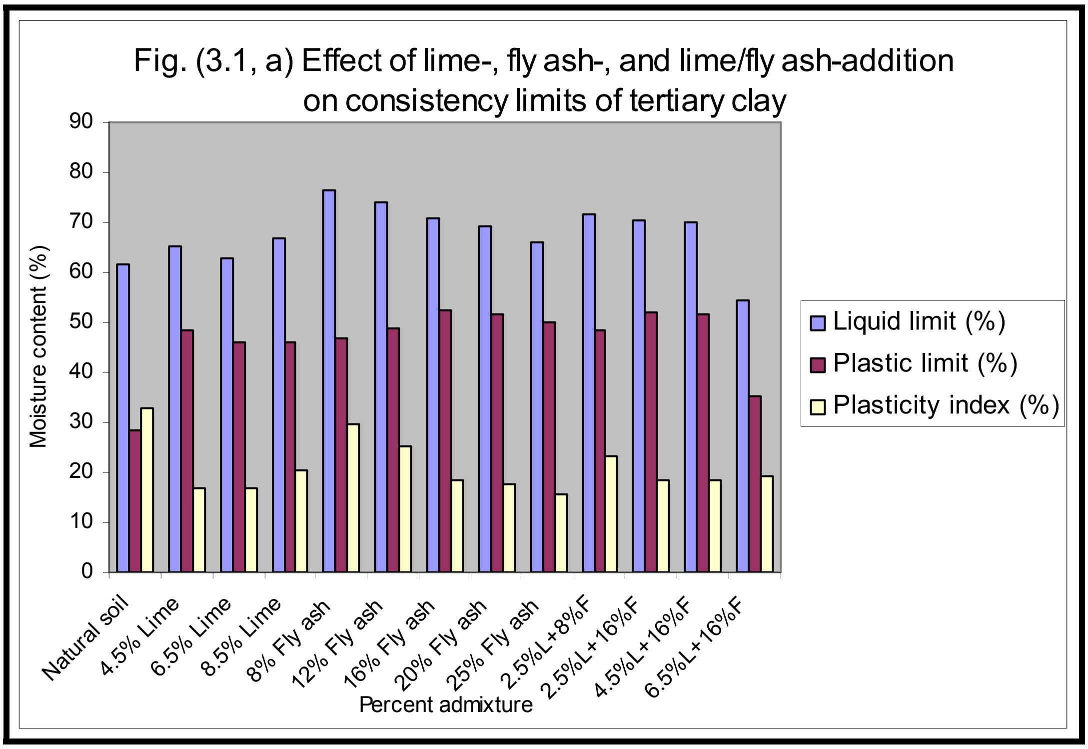


Fig. (3.1, a) Effect of lime-, fly ash-, and lime/fly ash-addition on consistency limits of tertiary clay

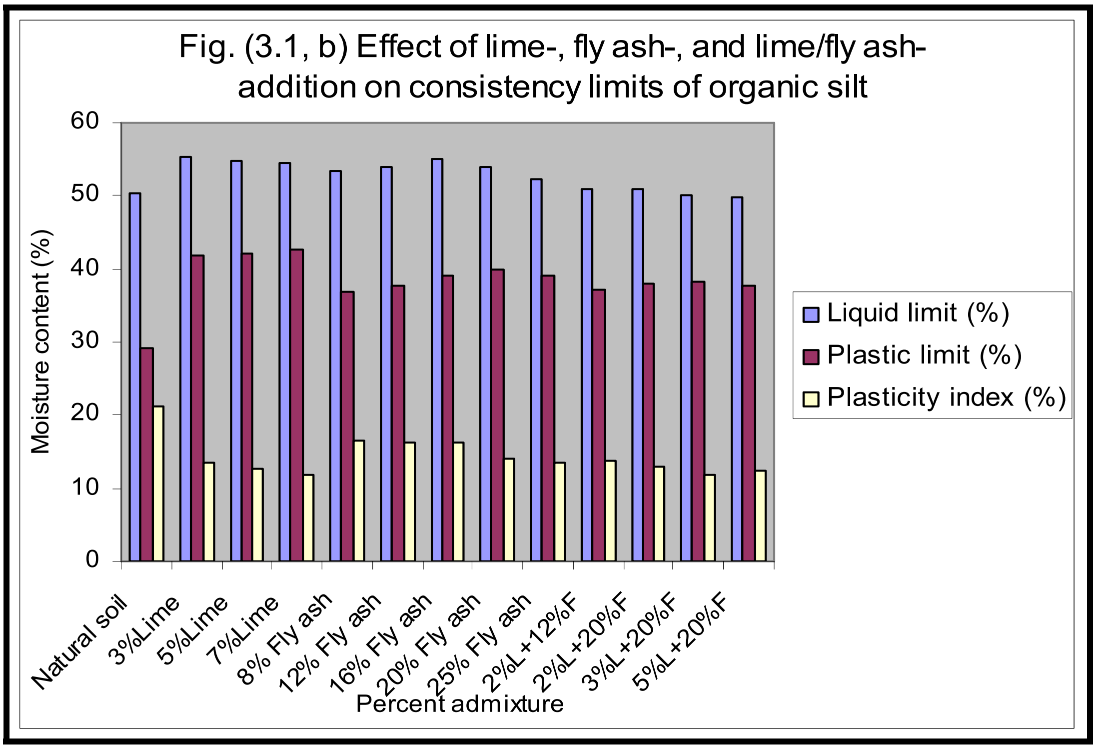


Fig. (3.1, b) Effect of lime-, fly ash-, and lime/fly ash-addition on consistency limits of organic silt

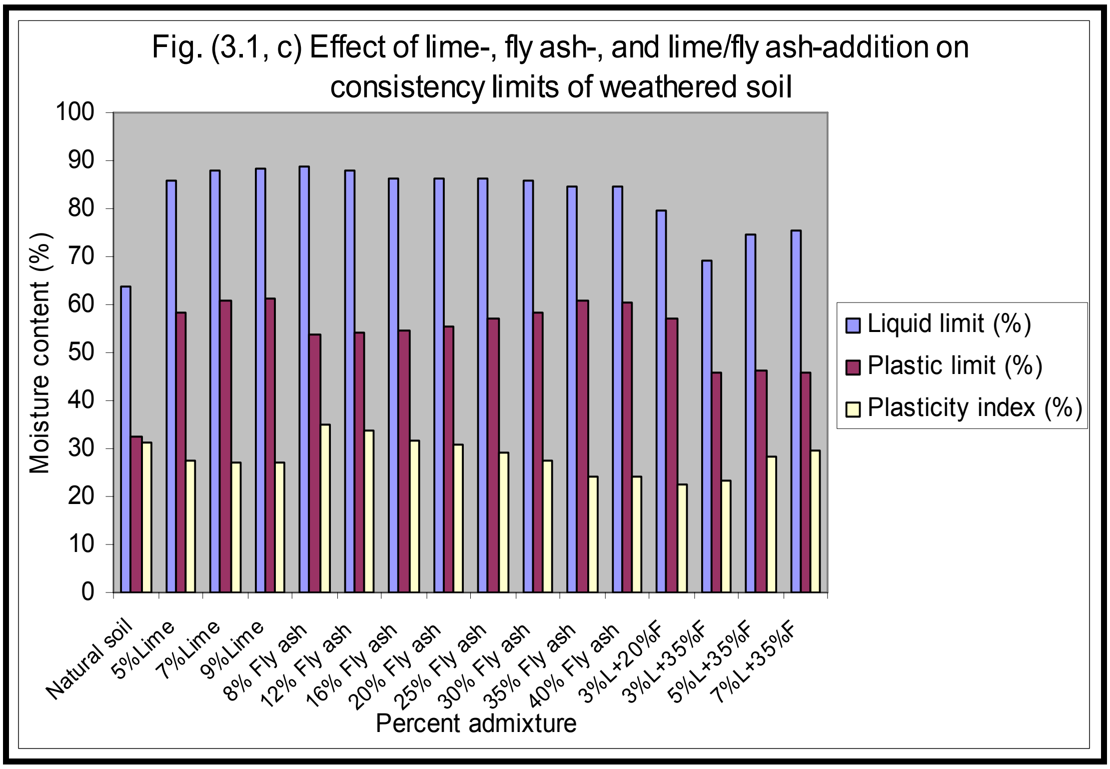


Fig. (3.1, c) Effect of lime-, fly ash-, and lime/fly ash-addition on consistency limits of weathered soil

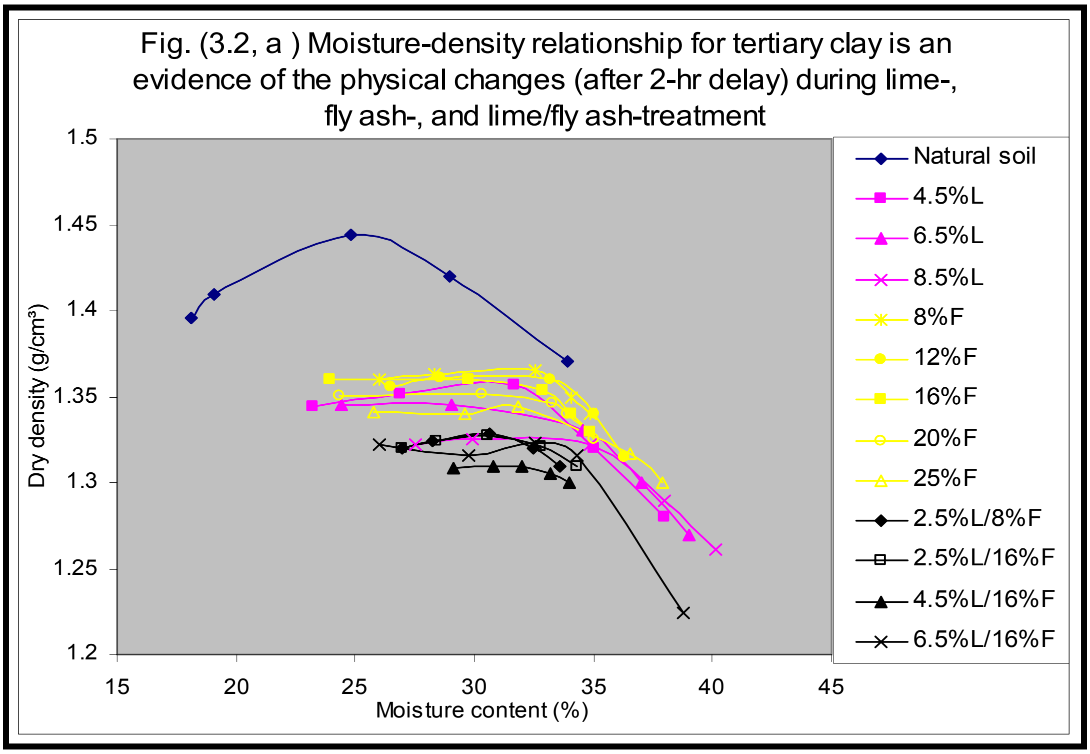


Fig. (3.2, a) Moisture-density relationship for tertiary clay is an evidence of the physical changes (after 2-hr delay) during lime-, fly ash-, and lime/fly ash-treatment

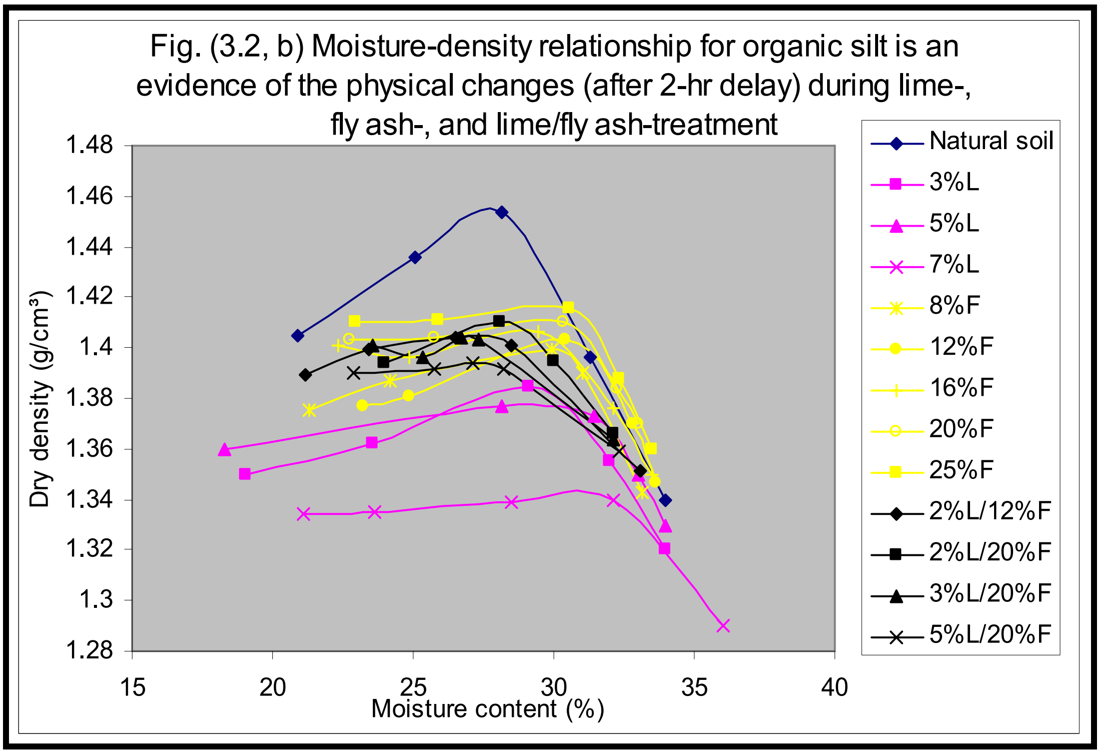


Fig. (3.2, b) Moisture-density relationship for organic silt is an evidence of the physical changes (after 2-hr delay) during lime-, fly ash-, and lime/fly ash-treatment

3.3 Unconfined compressive strength of untreated compacted soils

Compressive strength of a soil is a significant factor to estimate the design criteria for the use as a pavement and construction material. The lime- and fly ash-stabilization of soil, generally, leads to increase in the strength of the soil. Therefore, lime and fly ash become cost-effective and efficient material for use in road construction, embankment, and earth fills.

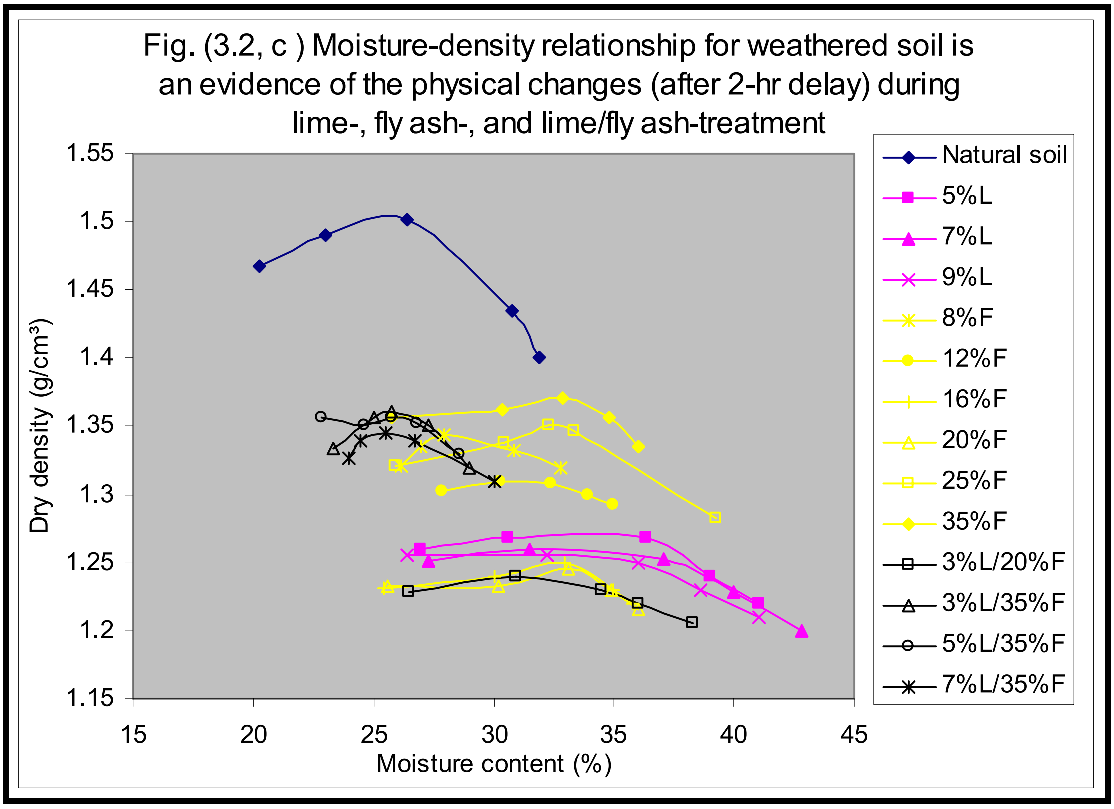


Fig. (3.2, c) Moisture-density relationship for weathered soil is an evidence of the physical changes (after 2-hr delay) during lime-, fly ash-, and lime/fly ash-treatment

The strength gain of lime- and fly ash-stabilized soil is primarily caused by the formation of various calcium silicate hydrates and calcium aluminate hydrates. The exact products formed, however, depend on the type of clay mineralogy and the reaction conditions including temperature, moisture, and curing conditions (Nicholson et al., 1994).

Unconfined compression tests were conducted to characterize the strength of soils and their mixtures with lime, fly ash and lime/fly ash. The test procedures and the preparation of the specimens were performed according to the procedures in chapter 2.4.1 and 2.5, respectively.

Unconfined compressive strengths of the three studied soils, compacted at optimum water content and without chemical additives, are described in chapter 2.3.4 and given in Tables 2.8, 3.1, 3.2, and 3.3. Unconfined compressive strengths for tertiary clay, organic silt, and weathered soil were measured after carrying out of the ultrasonic p-wave velocity (Vp) test. All the specimens were prepared using a standard proctor test. Untreated compacted soil specimens (without chemical additives) of tertiary clay, organic silt, and weathered soil (compacted at optimum water content) had unconfined compressive strengths of 131.21, 136.91, and 173.25 KN/m^2, respectively. Unconfined compressive strengths of untreated compacted soils need to be interpreted in the context of the general relationship between the unconfined compressive strength and the consistency (quality) of the soils used in pavement applications according to Das, 1994 (see chapter 2.4.1). Unconfined compressive strength ranging from 100 to 200 Kpa (KN/m^2) is considered as a stiff consistency. This means that the three tested untreated compacted soils are stiff consistency due to compaction process at the optimum water content and without chemical additives (lime & fly ash).

3.4 Unconfined compressive strength of treated stabilized soils

Unconfined compressive strength of different soil-lime, soil-fly ash and soil-lime/fly ash mixtures prepared at optimum water content are given in Tables 3.1, 3.2, and 3.3. The unconfined compressive strength was measured after 7 days curing and after carrying out of the p-wave velocity (Vp) test for the three studied stabilized soils.

Soil-lime mixtures of the three tested soils were prepared at the optimum lime content, 2%, and 4% above the optimum lime content (compacted at optimum water content) and cured for 7 days (see chapter 2.5.1). The values of unconfined compressive strength are illustrated in Tables 3.1, 3.2, & 3.3 and in Figures 3.3 a & 3.4.

Soil-fly ash mixtures were prepared at 8, 12, 16, 20, and 25% fly ash for both the tertiary clay and the organic silt and at 8, 12, 16, 20, 25, and 35% fly ash for the weathered soil. All the mixtures were compacted at the optimum water content, two hours after mixing with water to simulate the construction delay that typically occurs in the field before sub-grade compaction due to construction operations, and cured for 7 days (see chapter 2.5.2).

Soil-fly ash mixtures of tertiary clay, organic silt and weathered soil were prepared (at optimum water contents) at the optimum fly ash content 16, 20, and 35%, respectively. Subsequently, they were cured for 7, 28, 56, and 180 days to evaluate the effect of long-term curing on the unconfined compressive strength and on the fly ash-stabilization process. The values of unconfined compressive strength are shown in Tables 3.1, 3.2, & 3.3 and in Figures 3.3 b & 3.4.

Soil-lime/fly ash mixtures were prepared (at optimum water contents) at the optimum lime/fly ash contents, according to pH-test, as follows: for tertiary clay (2.5%L/8%F), for organic silt (2%L/12%F), and for weathered soil (3%L/20%F). All the mixtures were cured for 7 days. Other mixtures were prepared at the optimum fly ash content with different lime percentages to evaluate the influence of the increase in the lime and the lime/fly ash ratio on the lime/fly ash-stabilization process (see chapter 2.5.3).

Table (3.1) Unconfined compressive strength (qu-value), qu-gain, California bearing ratio (CBR-value), and CBR-gain of untreated compacted- and treated stabilized-tertiary clay with several blending.

Admixture mixing		Curing time	qu-value	qu-gain	CBR-value	CBR-gain
L (%)	FA (%)	Days	kN/m^2	-	(%)	-
0	0	-	131.21	1	4.6	1
4.5*	0	7	1034.00	7.88	60	13.00
6.5	0	7	1147.80	8.75	61.8	13.43
8.5	0	7	1221.70	9.31	62.3	13.54
0	8	7	366.88	2.80	21.9	4.76
0	12	7	594.90	4.53	36.6	7.96
0	16*	7	820.38	6.25	39.4	8.57
		28	1459.27	11.12	73.8	16.04
		56	1660.64	12.66	77.0	16.74
		180	1791.37	13.65	94.9	20.63
0	20	7	1064.97	8.12	61.1	13.28
0	25	7	950.00	7.24	51.6	11.22
2.5*	8*	7	905.59	6.90	46.8	10.17
1.5	16	7	1424.42	10.86	59.7	12.98
2.5	16	7	1476.28	11.25	87.2	18.96
		28	2256.57	17.20	109	23.70
		56	3081.50	23.49	168.3	36.59
		180	3505.23	26.71	234.2	50.91
4.5	16	7	1363.75	10.39	81.8	17.78
6.5	16	7	1159.56	8.84	79.5	17.28

Notes:
L = Lime content
FA = Fly ash content
CBR = California bearing ratio
qu = Unconfined compressive strength
CBR-gain = CBR of treated stabilized soil/CBR of untreated compacted soil
qu-gain = qu of treated stabilized soil/qu of untreated compacted soil
* Optimum content according to pH-test

Soil-lime/fly ash mixtures of tertiary clay (2.5%L/16%F), organic silt (2%L/20%F), and weathered soil (3%L/35%F) were prepared at the optimum fly ash content with small percentages of lime (see chapter 2.5.3) and at the optimum water contents. The mixtures were cured for 7, 28, 56, and 180 days to estimate the influence of curing time on the unconfined compressive strength and on the lime/fly ash-stabilization process. The values of unconfined compressive strength are illustrated in Tables 3.1, 3.2, & 3.3 and in Figures 3.3 c, 3.4 & 3.5.

Table (3.2) Unconfined compressive strength (qu-value), qu-gain, California bearing ratio (CBR-value), and CBR-gain of untreated compacted- and treated stabilized-organic silt with several blending.

Admixture mixing		Curing time	qu-value	qu-gain	CBR-value	CBR-gain
L (%)	FA (%)	Days	kN/m²	-	(%)	-
0	0	-	136.91	1	3.2	1
3*	0	7	439.49	3.21	16.8	5.25
5	0	7	634.40	4.63	23.4	7.31
7	0	7	628.03	4.59	23.4	7.31
0	8	7	322.29	2.35	14.7	4.59
0	12	7	532.48	3.89	28.7	8.97
0	16	7	620.38	4.53	37	11.56
0	20*	7	685.35	5.01	45.8	14.31
		28	1331.26	9.72	58.2	18.19
		56	1385.55	10.12	94.4	29.50
		180	1738.98	12.70	105.0	32.81
0	25	7	964.33	7.04	47.8	14.94
2*	12 *	7	729.65	5.33	35.2	11.00
2	20	7	866.43	6.33	38.4	12.00
		28	1348.78	9.85	61.1	19.09
		56	1648.44	12.04	80.4	25.13
		180	2148.35	15.69	103.1	32.22
3	20	7	871.88	6.37	40.2	12.56
5	20	7	796.11	5.82	27.4	8.56

Table (3.3) Unconfined compressive strength (qu-value), qu-gain, California bearing ratio (CBR-value), and CBR-gain of untreated compacted- and treated stabilized-weathered soil with several blending.

Admixture mixing		Curing time	qu-value	qu-gain	CBR-value	CBR-gain
L (%)	FA (%)	Days	kN/m²	-	(%)	-
0	0	-	173.25	1	5.4	1
5*	0	7	309.55	1.79	17.1	3.17
7	0	7	329.90	1.90	17.6	3.26
9	0	7	298.09	1.72	16.8	3.11
0	8	7	294.27	1.70	15.3	2.83
0	12	7	301.54	1.74	17.5	3.24
0	16	7	335.50	1.94	22.8	4.22
0	20	7	389.81	2.25	26.4	4.89
0	25	7	526.12	3.04	43.5	8.06
0	35*	7	938.85	5.42	50.2	9.30
		28	1151.07	6.64	51.0	9.44
		56	1184.21	6.84	62.9	11.65
		180	1412.79	8.16	95.3	17.65
3*	20*	7	713.39	4.12	44.5	8.24
3	35	7	1327.30	7.66	79.2	14.67
		28	1619.81	9.35	84.4	15.63
		56	1751.11	10.11	103.3	19.13
		180	2297.16	13.26	115.0	21.30
5	35	7	1421.54	8.21	80.2	14.85
7	35	7	1416.49	8.18	76.6	14.19
8	35	7	1376.68	7.95	72.4	13.41

3.4.1 General effect of lime-, fly ash-, and lime/fly ash-stabilization process

The general effect of lime-, fly ash-, and lime/fly ash-stabilization process on the three studied soils is illustrated in Figure 3.3 a, b, and c, respectively.

The addition of optimum lime content led to an increase in the unconfined compressive strength for the three different studied soils. Lime-tertiary clay mixtures have the highest unconfined compressive strength values and lime-weathered soil mixtures have the lowest values. The reactivity of tertiary clay with lime is strong and the unconfined compressive strength increased with increasing lime content (2 and 4% above the optimum lime content). The reactivity of both organic silt and weathered soil with lime is weak according to reactivity test (see chapter 2.5.1). The lime-organic silt mixtures have unconfined compressive strength values relatively higher than the values of lime-weathered soil mixtures. The unconfined compressive strengths of both lime-organic silt and -weathered soil mixtures increased with increase in the lime content (2% above the optimum lime content) and decreased slightly with continuous increasing the lime content (4% above the optimum lime content).

The ratio of the unconfined compressive strength of the lime-, fly ash-, and lime/fly ash-stabilized soil to that of the untreated compacted soil is known as *strength gain factor* (see Tables 3.1, 3.2, & 3.3 and Fig. 3.4). The strength gain factor, due to optimum lime content, of tertiary clay, organic silt, and weathered soil is 7.88, 3.21, and 1.79 (time), respectively. The final consistency (quality) of the mixtures is hard, hard, and very stiff, respectively (see Table 3.4).

The addition of fly ash contents resulted in an increase in the unconfined compressive strength for the three tested soils. Fly ash-tertiary clay mixtures have the highest unconfined compressive strength values and fly ash-weathered soil mixtures have the lowest values, at the same fly ash contents. The unconfined compressive strength values increased with continuous increasing the fly ash content (from 8 to 20% fly ash). Above 20% fly ash, for example 25%, the unconfined compressive strength value of the fly ash-tertiary clay mixture decreased.

The qu-values of both fly ash-organic silt and -weathered soil mixtures are lower than the values of fly ash-tertiary clay mixtures, at the same fly ash contents. Fly ash-organic silt mixtures have unconfined compressive strength values relatively higher than the values of fly ash-weathered soil mixtures, at the same fly ash contents. The unconfined compressive strength values for both fly ash-organic silt and -weathered soil mixtures increased with increasing the fly ash content.

The strength gain factor of optimum fly ash-tertiary clay, optimum fly ash-organic silt, and optimum fly ash-weathered soil mixtures is 6.25, 5.01, and 5.42, respectively (see Fig. 3.4).

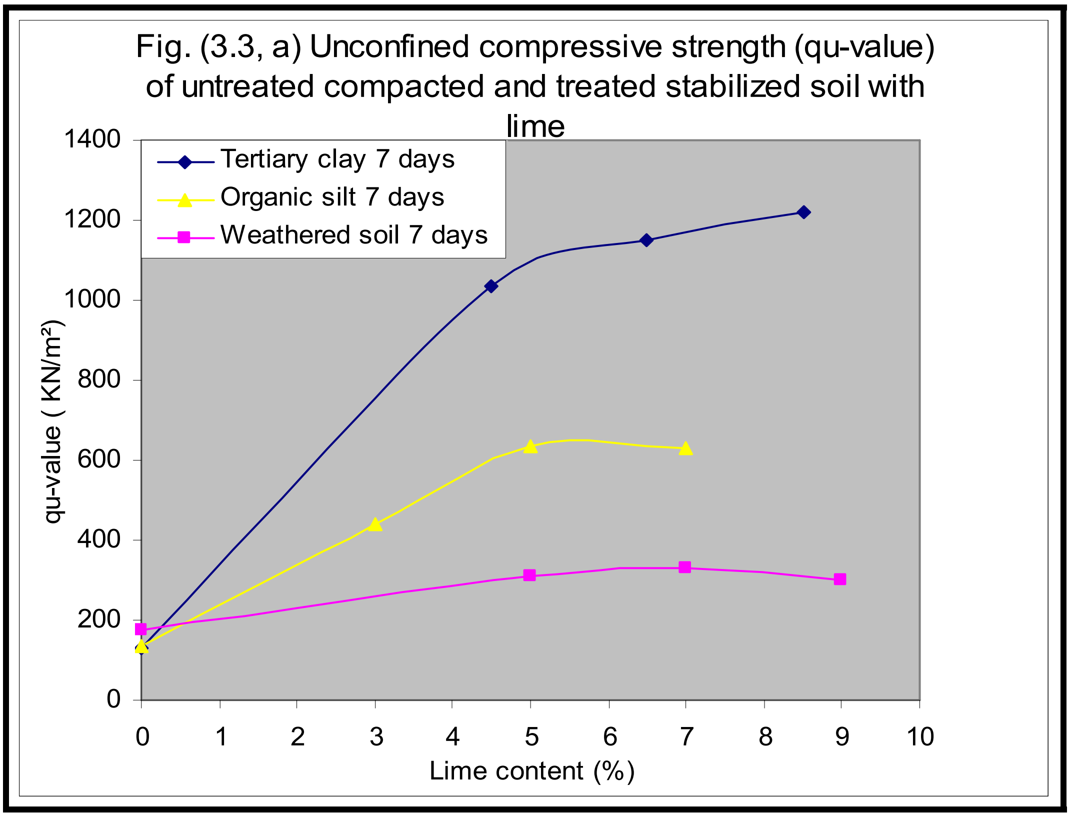


Fig. (3.3, a) Unconfined compressive strength (qu-value) of untreated compacted and treated stabilized soil with lime

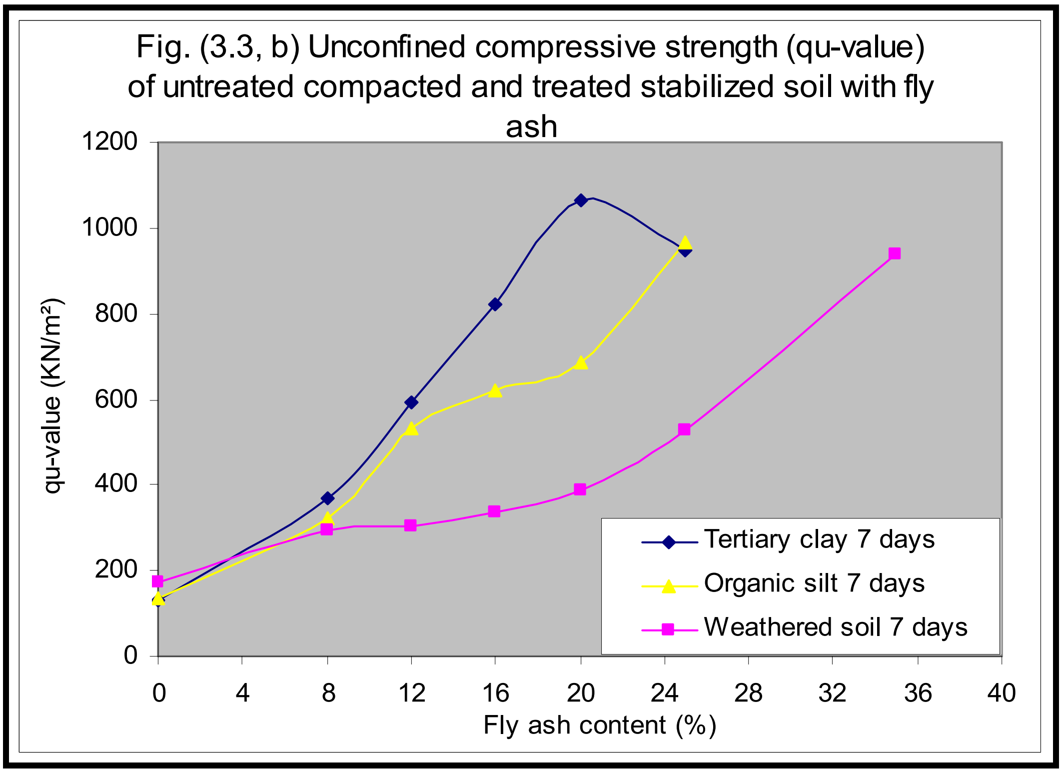


Fig. (3.3, b) Unconfined compressive strength (qu-value) of untreated compacted and treated stabilized soil with fly ash

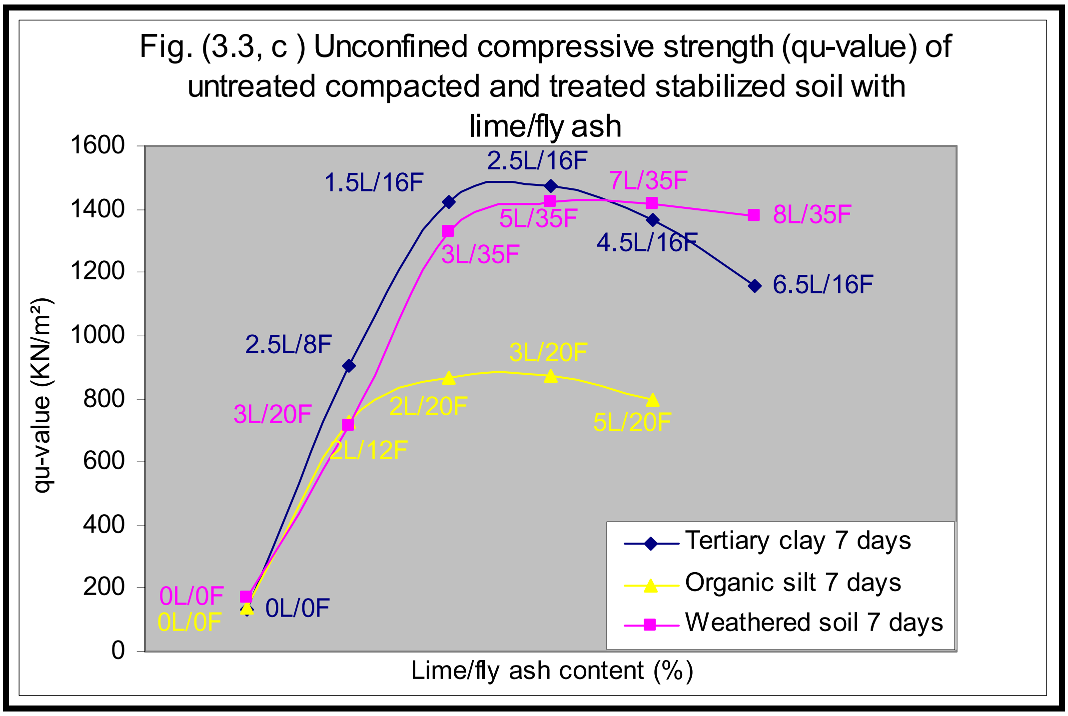


Fig. (3.3, c) Unconfined compressive strength (qu-value) of untreated compacted and treated stabilized soil with lime/fly ash

The strength gain factor of weathered soil is relatively higher than the gain factor of organic silt, this may be, since the optimum fly ash content of weathered soil (35% fly ash) is greater than the optimum fly ash of organic silt (20% fly ash). The final consistency (quality) of the three mixtures is hard (see Table 3.4).

The addition of lime and fly ash together led to an increase in the unconfined compressive strength values for the three studied soils strongly compared to the addition of lime and fly ash separately. Lime/fly ash-tertiary clay mixtures have unconfined compressive strength values higher than the values of both lime/fly ash-weathered soil and lime/fly ash-organic silt mixtures. The reactivity of tertiary clay with lime and fly ash together is stronger than the reactivity of both weathered soil and organic silt. The strength gain factors (due to addition of the optimum lime/fly ash content, according to the pH-test) of tertiary clay, organic silt, and weathered soil are 6.90, 5.33, and 4.12, respectively. The strength gain factor (due to addition of the optimum fly ash with small percent of lime) of tertiary clay (2.5%L/16%F), organic silt (2%L/20%F), and weathered soil (3%L/35%F) is 11.25, 6.33, and 7.66, respectively (see Fig. 3.4). The final consistency (quality) of the three mixtures is hard (see Table 3.4). The reaction of the three studied soils with lime and fly ash together is stronger than the reaction with lime and fly ash separately. The unconfined compressive strength values of the three studied soils increased with increasing the lime/fly ash ratio. The optimum lime/fly ash ratio of tertiary clay, organic silt, and weathered soil is 0.16, 0.15, and 0.14, respectively (see Fig. 3.5). Above the ratio of 0.16 in the case of tertiary clay and 0.15 in the case of organic silt (about 1 lime: 6

fly ash by weight) and above the ratio of 0.14 in the case of weathered soil (about 1 lime: 7 fly ash by weight), qu-value of the mixtures decreased.

Table (3.4) Description of the quality of both untreated compacted- and treated stabilized- soils (according to Das, 1994 & Bowles, 1992).

Soil type	Type and percent of chemical additives	Curing time (days)	qu-value (kN/m²)	Quality after qu-value, Das (1994)	CBR-value (%)	Quality after CBR-value, Bowles (1992)
Organic silt	0%	-	136.91	stiff	3.2	poor to fair
	3% Lime	7	439.49	hard	16.8	good
	20% Fly ash	7	685.35	hard	45.8	good
		28	1331.26	hard	58.2	excellent
		56	1385.55	hard	94.4	excellent
		180	1420.12	hard	105.0	excellent
	2% L+12% F	7	729.65	hard	35.2	good
	2% L+20% F	7	866.43	hard	40.2	good
		28	1348.78	hard	61.1	excellent
		56	1648.44	hard	80.4	excellent
		180	2148.35	hard	103.1	excellent
Tertiary clay	0%	-	131.21	stiff	4.6	poor to fair
	4.5% Lime	7	1034	hard	60	excellent
	16% Fly ash	7	820.38	hard	39.4	good
		28	1459.27	hard	73.8	excellent
		56	1660.64	hard	77.0	excellent
		180	1791.37	hard	94.9	excellent
	2.5% L+8% F	7	905.59	hard	46.8	good
	2.5% L+16% F	7	1476.28	hard	87.2	excellent
		28	2256.57	hard	109.4	excellent
		56	3081.49	hard	168.3	excellent
		180	3505.23	hard	234.2	excellent
Weathered soil	0%	-	173.25	stiff	5.4	poor to fair
	5% Lime	7	309.55	very stiff	17.1	fair
	35% Fly ash	7	938.85	hard	50.2	excellent
		28	1151.07	hard	51.0	excellent
		56	1184.21	hard	62.9	excellent
		180	1412.79	hard	95.3	excellent
	3% L+ 20% F	7	713.39	hard	44.5	good
	3% L+ 35% F	7	1327.30	hard	79.2	excellent
		28	1619.81	hard	84.4	excellent
		56	1751.11	hard	103.3	excellent
		180	2297.16	hard	115.0	excellent

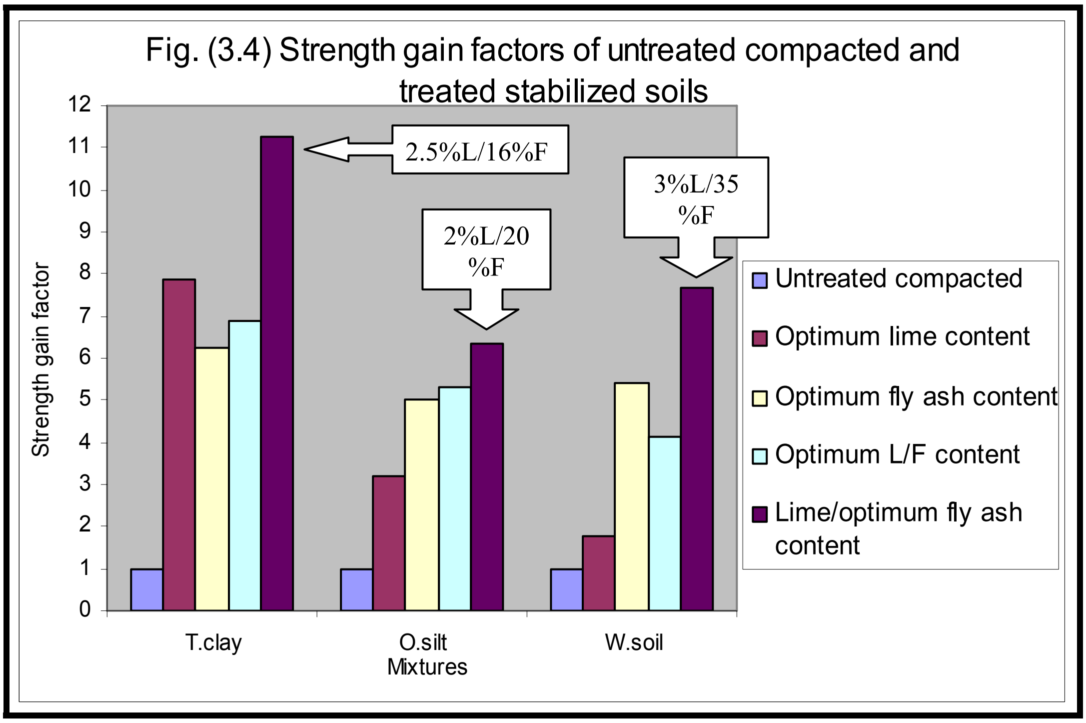


Fig. (3.4) Strength gain factors of untreated compacted and treated stabilized soils

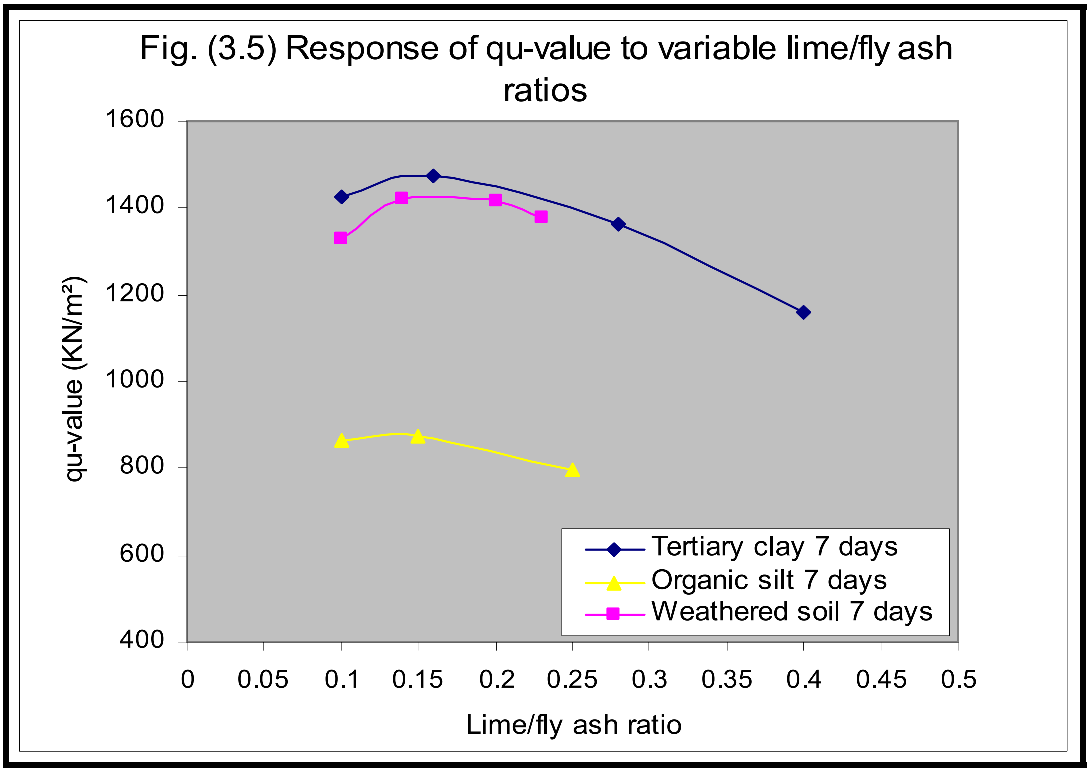


Fig. (3.5) Response of qu-value to variable lime/fly ash ratios

3.4.2 Effect of curing time

Soil-fly ash and soil-lime/fly ash mixtures were prepared, for the three different soils, and cured for periods of 7, 28, 56, and 180 days to estimate how curing time affects unconfined compressive strength of the stabilized soils. Unconfined compressive strength tests were performed on the specimens after carrying out of the p-wave velocity (Vp) test.

All the specimens were prepared at the optimum water content. The effect of long-term curing on the unconfined compressive strength of soil-fly ash and soil-lime/fly ash mixtures is shown in Figure 3.6. The unconfined compressive strength values of both soil-fly ash and soil-lime/fly ash mixtures increased with long-term curing. Unconfined compressive strength values of soil-lime/fly ash mixtures were strongly affected by the long-term curing compared to the strength values of soil-fly ash mixtures. Unconfined compressive strength values of tertiary clay-optimum fly ash mixtures increased strongly with curing time compared to unconfined compressive strength values of both weathered soil- and organic silt-optimum fly ash mixtures. Unconfined compressive strength values of weathered soil-optimum fly ash mixtures increased slightly with curing time. Unconfined compressive strength value of tertiary clay-lime/fly ash mixtures increased dramatically with long-term curing compared to both weathered soil- and organic silt-lime/fly ash mixtures (see Fig. 3.6 and Tables 3.1, 3.2, & 3.3).

3.4.3 Stress-strain behavior

Figure 3.8 (a, b, & c) shows stress-strain curves, at confining pressure σ_3 = zero, for the three studied soils at different soil-chemical additives with different curing time (7, 28, and 180 days). All the specimens were compacted at optimum water content. It was observed that the overall soil behavior was significantly influenced by both lime- and fly ash-addition. Unconfined compressive strength, elasticity modulus at the first loading (E_{secant}), failure axial strain (ε_f), and brittleness changed as a consequence of either the separate or the joined effects of lime and fly ash contents. By comparing the curves, for all the three tested soils, showed that the natural soils have ductile behavior, where the unconfined compressive strength equal to stress-value at 20% strain (according to DIN 18 136).

The untreated compacted soils have relatively similar behavior, which means that the compaction process for the natural soils at optimum water content and without chemical additives, does not affect the stress-stain behavior to large amplitude. Peak strength and elasticity modulus increased slightly, while axial strain at the failure decreased in case of the tertiary clay and the weathered soil and remained quite similar in case of the organic silt (see Table 3.5 and Fig. 3.7).

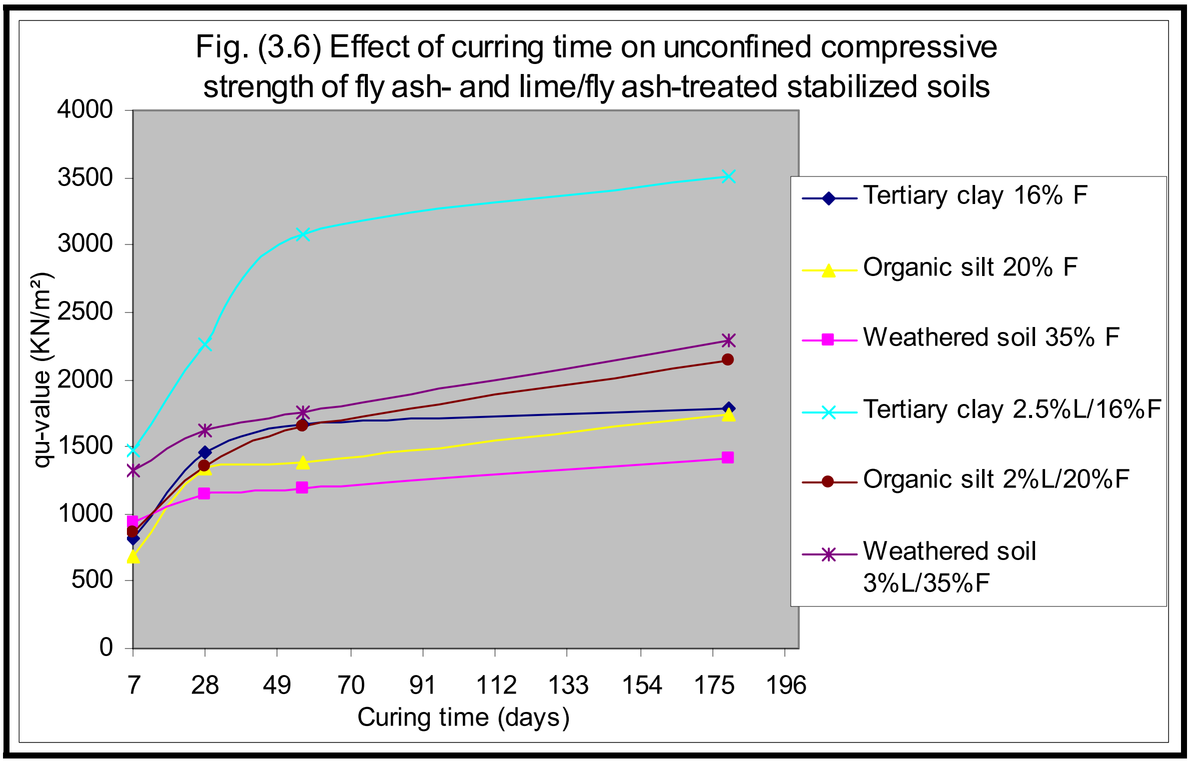
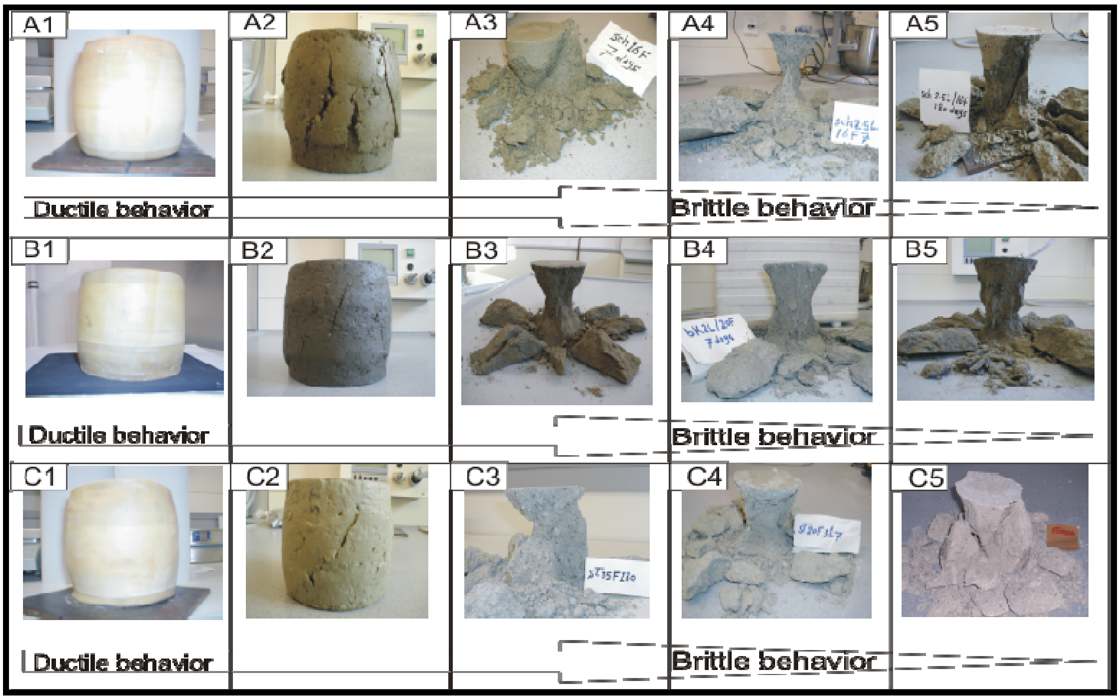


Fig. (3.6) Effect of curring time on unconfined compressive strength of fly ash- and lime/fly ash-treated stabilized soils

Fig. (3.7) Photos of specimens after unconfined compressive strength illustrated the development of the mechanical behavior from ductile to brittle due to the stabilization process and the curing time.



A1= Natural clay, A2= Untreated compacted clay, A3= Clay+16%ash 7 days, A4= Clay+2.5%lime+16%ash 7 days, A5= Clay+2.5%lime+16%ash 180 days.
B1= Natural organic silt, B2= Untreated compacted organic silt, B3= Organic silt+20%ash 28 days, B4= Organic silt+2%lime+20%ash 7 days, B5= Organic silt+2%lime+20% ash 180 days.
C1= Natural weathered soil, C2= Untreated compacted weathered soil, C3= Weathered soil+35%ash 180 days, C4= Weathered soil+3%lime+20%ash 7 days, C5= Weathered soil+3%lime+35%ash 180 days.

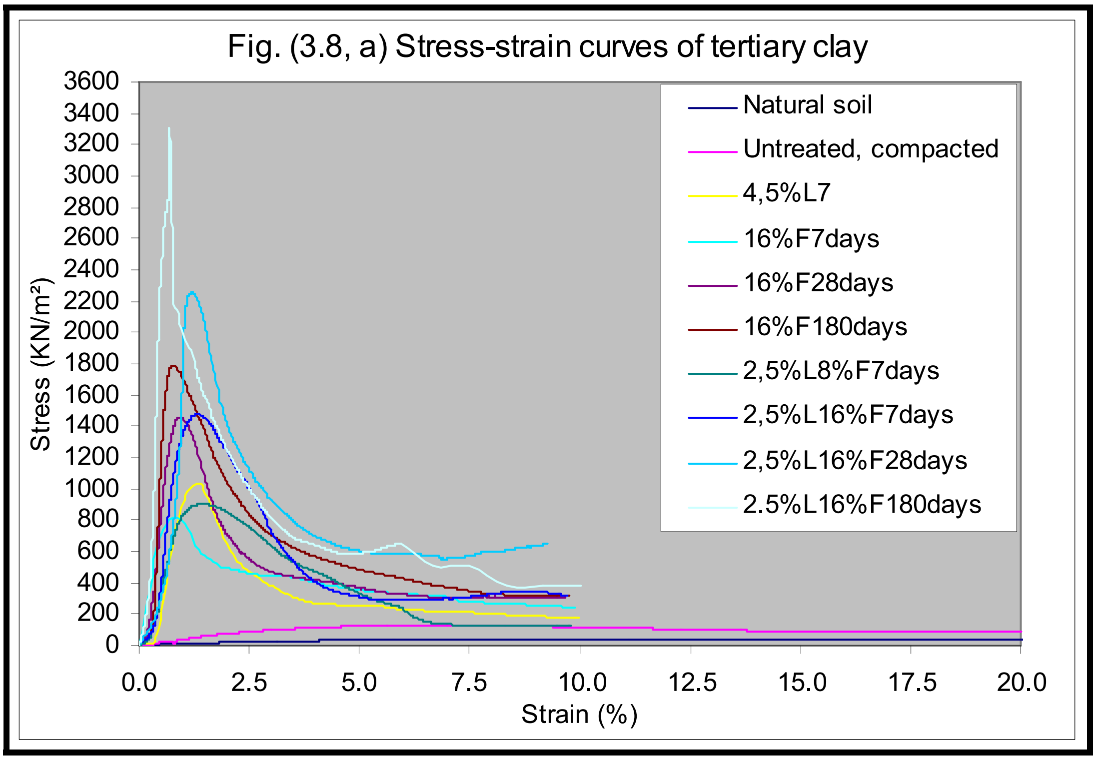


Fig. (3.8, a) Stress-strain curves of tertiary clay

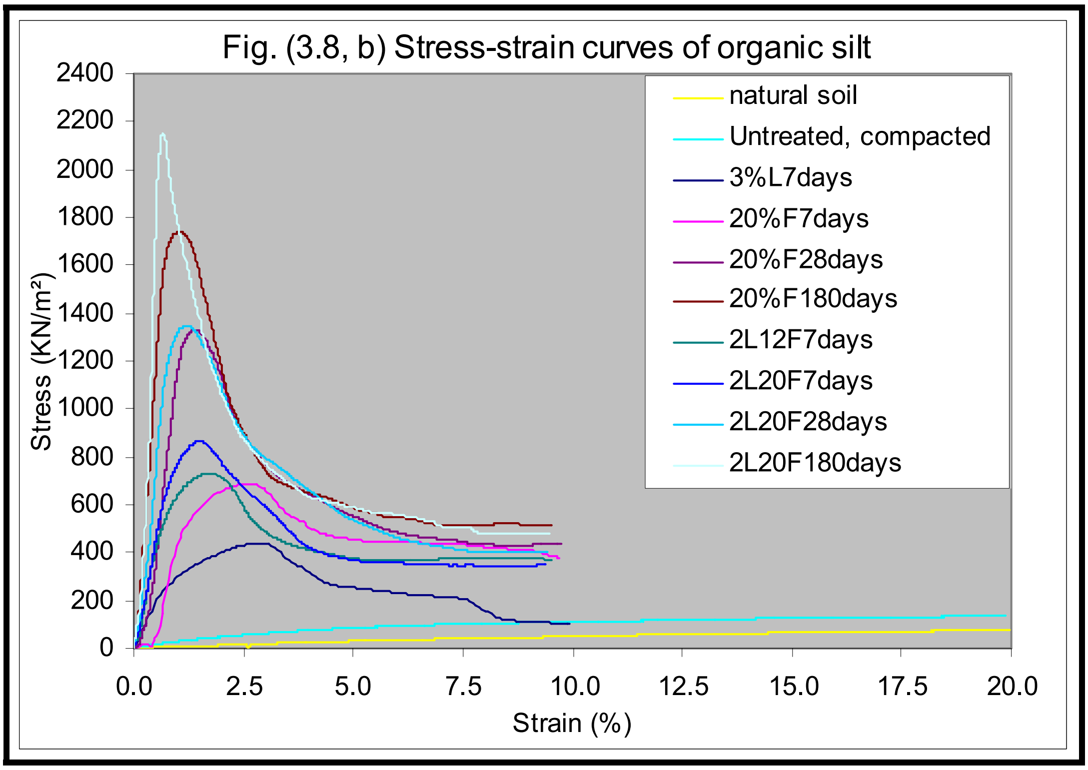


Fig. (3.8, b) Stress-strain curves of organic silt

The addition of both optimum lime content and optimum fly ash content, separately, led to an increment of the E_{secant} and a decrement of the failure axial strain (ε_f) for the three studied soils. The increase in E_{secant} and the reduction of the failure axial strain (ε_f) were relatively high with the addition of optimum fly ash content compared to the addition of optimum lime content for the three studied soils.

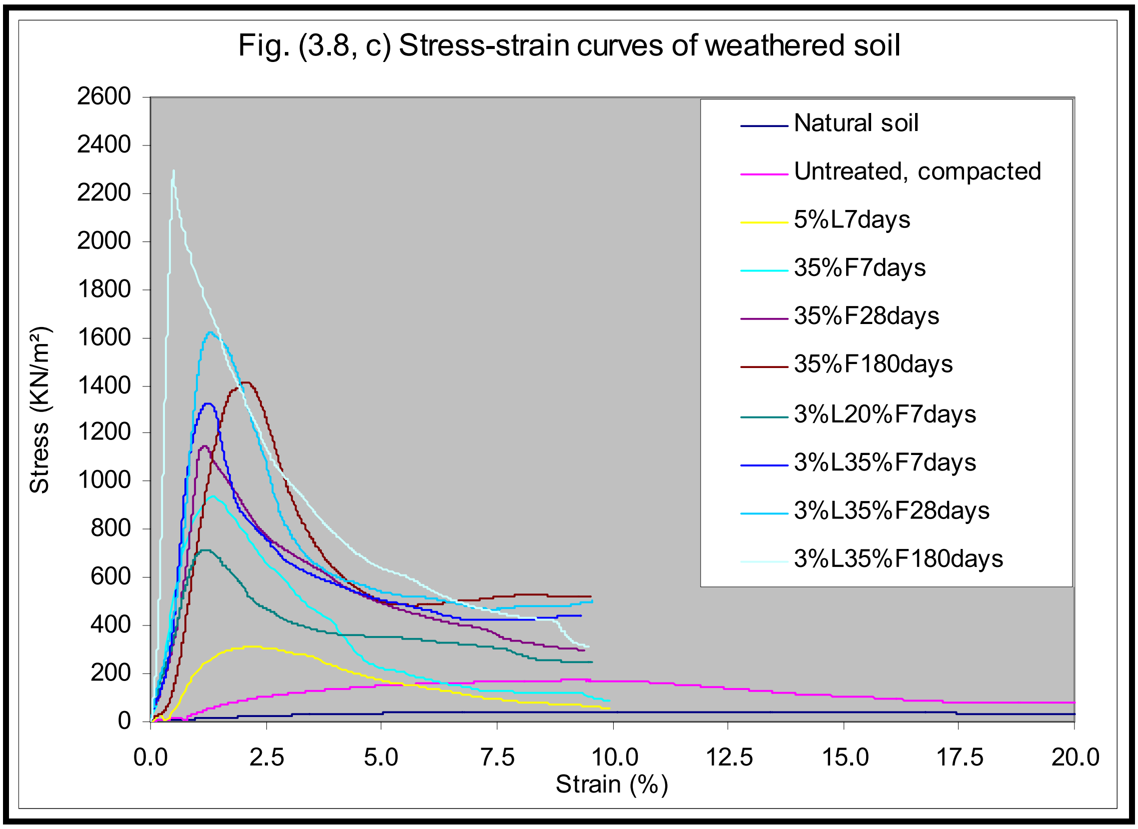


Fig. (3.8, c) Stress-strain curves of weathered soil

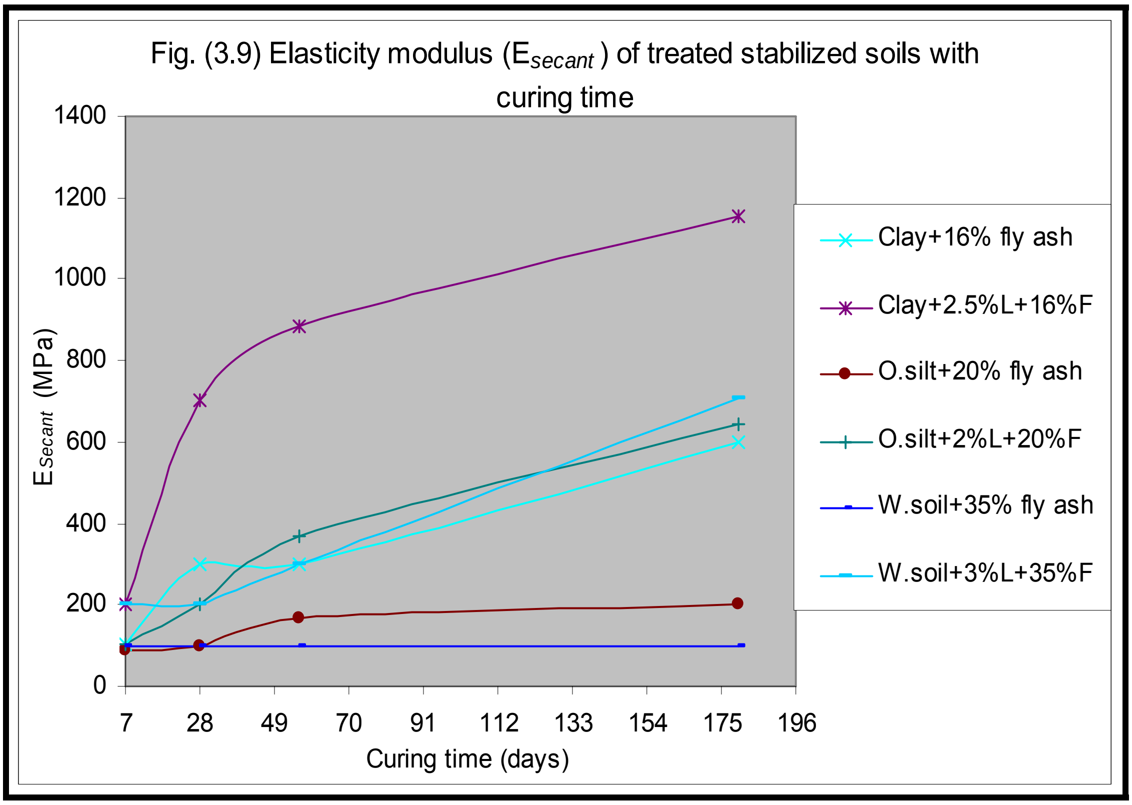


Fig. (3.9) Elasticity modulus (E_{secant}) of treated stabilized soils with curing time

In the case of soil-fly ash mixtures, E_{secant} increased and the failure axial strain (ε_f) decreased through the long-term curing in case of the three tested soils (with the exception of the weathered soil-fly ash mixture, where the general direction of the development of these two parameters (E_{secant} and ε_f) with the long-term curing is relatively constant). In the case of soil-lime/fly ash mixtures, unconfined compressive strength qu-value and E_{secant} increased and the failure axial strain (ε_f) decreased through the long-term curing for the three different soils especially in the case of lime/fly ash-stabilized tertiary clay, where the change was dramatic. Finally, the soil-lime/fly ash mixtures showed an extremely brittle behavior especially through the long-term curing in comparison to the soil-fly ash mixtures which showed relatively smaller change (see Table 3.5, Appendix 13, and Fig. 3.9 & 3.10).

Table (3.5) illustrates unconfined compressive strengths (qu-values), failure axial strains (ε_f), and elasticity modulus (E_{Secant}) of untreated compacted- and treated stabilized-soils at different mixtures and curing time.

Soil type	Mixtures	qu-value (KN/m^2)	ε_f (%)	E_{secant} (MPa)
Tertiary clay	Untreated compacted	131.21	6.10	5
	4.5% lime 7 days	1034.00	1.37	101
	16% fly ash 7 days	820.38	1.20	101
	16% fly ash 28 days	1459.27	0.93	300
	16% fly ash 56 days	1660.64	0.92	300
	16% fly ash 180 days	1791.37	0.78	600
	2.5% lime/ 8% fly ash7	905.59	1.46	100
	2.5% L/16% fly ash 7	1476.28	1.33	201
	2.5% L/16% fly ash 28	2256.57	1.19	701
	2.5% L/16% fly ash 56	3081.49	0.84	882
	2.5% L/16%fly ash180	3505.23	0.70	1157
Organic silt	Untreated compacted	136.91	20	2
	3% lime 7 days	439.49	3	43
	20% fly ash 7 days	685.35	2.51	90
	20% fly ash 28 days	1331.26	1.40	101
	20% fly ash 56 days	1385.55	1.33	169
	20% fly ash 180 days	1420.12	1.00	201
	2% lime/12% fly ash 7	729.65	1.72	100
	2% L/20% fly ash 7	866.43	1.52	101
	2% L/20% fly ash 28	1348.78	1.20	200
	2% L/20% fly ash 56	1648.44	0.91	370
	2% L/20% fly ash 180	2148.35	0.67	642
Weathered soil	Untreated compacted	173.25	8.91	7
	5% lime 7 days	309.55	2.08	33
	35% fly ash 7 days	938.85	1.33	100
	35% fly ash 28 days	1151.07	1.19	100
	35% fly ash 56 days	1184.21	1.38	98
	35% fly ash 180 days	1412.79	1.20	101
	3% lime/20% fly ash 7	713.39	1.20	72
	3% L/35% fly ash 7	1327.30	1.27	200
	3% L/35% fly ash 28	1619.81	1.22	201
	3% L/35% fly ash 56	1751.11	1.38	299
	3% L/35% fly ash 180	2297.16	0.48	709

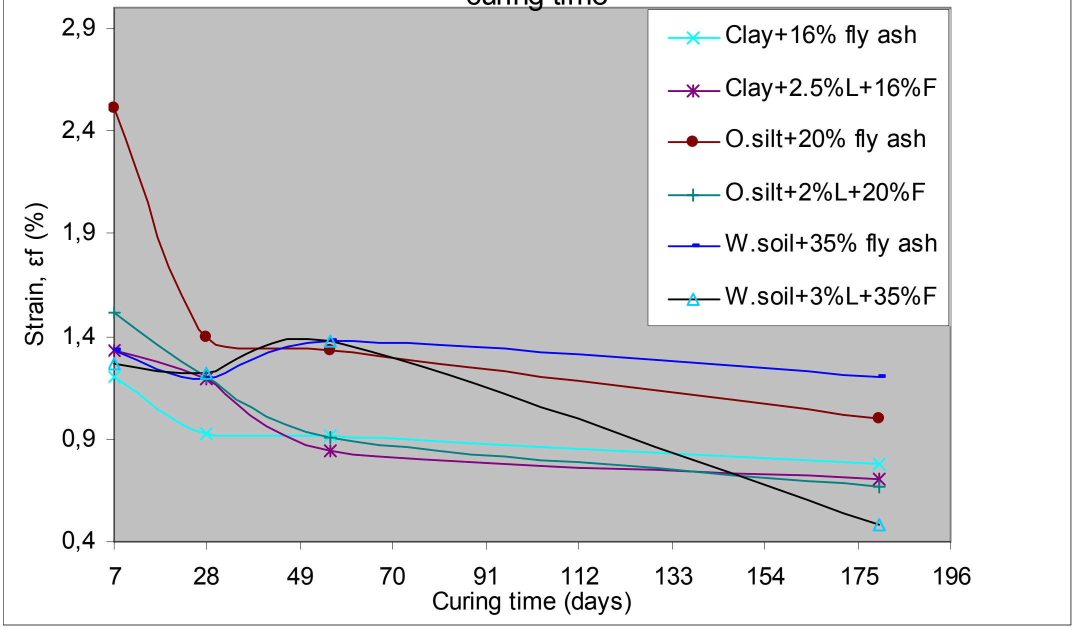


Fig. (3.10) Failure axial strain (εf) of treated stabilized soils with curing time

3.5 Durability

The stabilized soils must be subjected to the worst possible field conditions especially in the rainy season, where the soil becomes completely water-saturated, to estimate the strength loss due to the water saturation (soaking). Thus, durability (water-soaking) tests were conducted (loosely based on ASTM C593) on soil-fly ash mixtures of tertiary clay (16%F), organic silt (20%F), and of weathered soil (35%F) and on soil-lime/fly ash mixtures of tertiary clay (2.5% lime+16% fly ash), organic silt (2% lime+20% fly ash), and of weathered soil (3% lime+35% fly ash). The mixtures were compacted after standard proctor test and cured to 7 days (25°C temperature & 98% humidity). After the curing, one set of the samples was soaked in water bath for one hour and the corresponding sample set was vacuum saturated for one hour.

Unconfined compression test (according to DIN 18 138, see chapter 2.4.1) was performed, after soaking, on the two sample sets. The results of the test are shown in Table 3.6 and Figure 3.11. Both fly ash- and lime/fly ash-organic silt mixtures had the lowest strength loss (strength loss = qu-soaked/ qu-unsoaked * 100). Fly ash- and lime/fly ash-weathered soil mixtures had the highest strength loss.

Soil-fly ash mixtures of tertiary clay (16%F), organic silt (20%F), and weathered soil (35%F) and soil-lime/fly ash mixtures of tertiary clay (2.5% lime+16% fly ash), organic silt (2% lime+20% fly ash), and of weathered soil (3% lime+35% fly ash) were prepared after standard compaction test and cured to 7 days (25°C temperature & 98% humidity). They were subjected to wetting/drying-, and freezing/thawing-durability tests according to ASTM D559 and D560, respectively including 12 cycles of wetting/drying and 12 cycles of freezing/thawing. The procedures of the tests are similar to the procedures of the standards test methods for compacted cement mixtures except wire brushing after each cycle is omitted (U.S. Army, Air force, and Navy, 2005). Each test consists of twelve two-day cycles of wetting/drying or freezing/thawing, which means that each test requires 24 days to complete. Stabilized soil mixtures, that satisfy strength requirements, are required to pass these tests to prove their ability to withstand environmental conditions (Baghdadi el al., 1995). The results of the durability test were expressed in terms of weight loss at the end of the 12 cycles. The weigh loss criteria and the durability test results of the stabilized soils after 12 cycles are shown in Tables 3.7 and 3.8, respectively. In the case of freezing/thawing durability test, all the mixtures passed successively. In the case of wetting/drying durability test, three mixtures passed successively (tertiary clay-lime/fly ash, organic silt-fly ash, and -lime/fly ash mixtures) while the other three mixtures failed (see Tables 3.7 & 3.8 and Appendixes 14 & 15).

Table (3.6) Unconfined compressive strength, strength gain, and strength loss of the treated stabilized soils under different conditions

qu-value at different conditions / Samples	Unconfined compressive strength after 7 days curing (under 25°C temperature & 98 % humidity)							
	Unsoaked		Plain soaked loosely based on (ASTM C593)			Vacuum saturated soaked loosely based on (ASTM C593)		
	qu-value (KN/m^2)	Strength gain	qu-value (KN/m^2)	Strength gain	Strength loss (%)	qu-value (KN/m^2)	Strength gain	Strength loss (%)
Clay 16% F	820.38	6.25	646.45	4.93	21.20	643.27	4.90	21.59
Clay 2.5% L/ 16%F	1476.28	11.25	706.75	5.39	52.13	824.64	6.28	44.14
Silt 20% F	685.35	5.01	615.15	4.49	10.24	615.89	4.50	10.13
Silt 2% L/ 20% F	866.43	6.33	654.66	4.78	24.44	668.34	4.88	22.86
W.soil 35% F	938.85	5.42	472.30	2.73	49.69	456.01	2.63	51.43
W.soil 3% L/ 35% F	1327.30	7.66	601.50	3.47	54.68	735.28	4.24	44.60

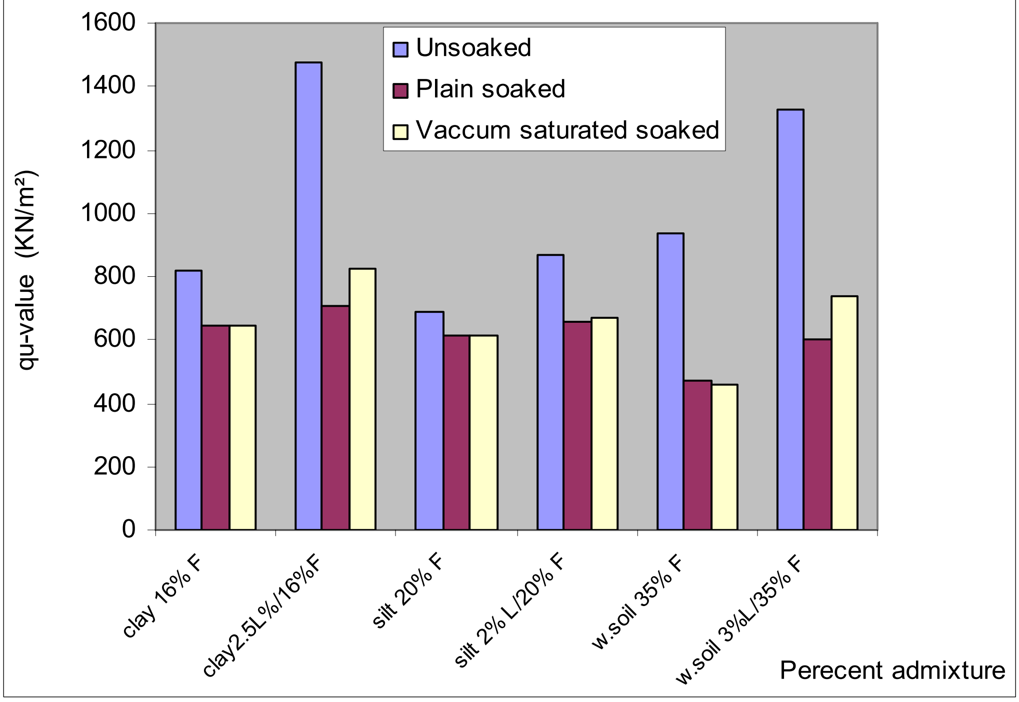
Table (3.7) Durability test weight loss criteria (Durability requirements) according to (ASTM D559 and D560).

Type of soil stabilized	Maximum allowable weight loss after 12 wet-dry or freeze-thaw cycles percent of initial specimen weight
Granular, PI < 10	11
Granular, PI > 10	8
Silt	8
Clay	6

Table (3.8) Durability test results.

Specimens	Numbers of specimens	Durability test results	
		Wet-Dry	Freeze-Thaw
T.clay+16% fly ash 7days	2	fail	pass
T.clay+2.5% lime+16% fly ash7	2	pass	pass
O.silt+20% fly ash 7days	2	pass	pass
O.silt+2% lime+20% fly ash 7	2	pass	pass
W. soil+ 35% fly ash 7days	2	fail	pass
W. soil+ 3% lime+35%fly ash 7	2	fail	pass

Fig. (3.11) Unconfined compressive strength of stabilized soils after 7 days under different conditions



3.6 Conclusions

* The addition of lime, fly ash, and lime/fly ash to the three tested soils led to a reduction of the plasticity index.

* The addition of lime, fly ash, and lime/fly ash to the studied soils resulted in an increase in the optimum moisture content and a decrease in the maximum dry density. The moisture-density curves of the stabilized soils had typical flattened form.

* In the case of lime-stabilization process, the optimum lime content of the tertiary clay, organic silt, and weathered soil is 4.5, 3, and 5%, respectively. The tertiary clay is strongly

reactive with lime and the unconfined compressive strength increased continuously with an increase in the lime content. Both the weathered soil and the organic silt failed in the reactivity test. The unconfined compressive strength increased at the optimum lime content and 2% above the optimum compared to the strength of the untreated compacted specimens. The continuous increase in the lime (4% above the optimum lime content) led to a decrease in the unconfined compressive strength.

* In the case of fly ash-stabilization process, the optimum fly ash content (according to pH-method) of the tertiary clay, organic silt, and weathered soil is 16, 20, and 35%, respectively. The unconfined compressive strength increased with increasing the fly ash content for both the organic silt and weathered soil. In case of the tertiary clay, the unconfined compressive strength increased with increasing fly ash content (from 8 to 20%) and decreased with continuous increase in fly ash content (above 20% fly ash content). Fly ash-organic silt mixtures have lower qu- and strength gain-values compared to fly ash-inorganic tertiary clay mixtures. These values are satisfactory and higher than qu-values of fly ash-weathered soil mixtures.

* In the case of lime/fly ash-stabilization process, the optimum lime/fly ash content (according to pH-method) of tertiary clay, organic silt, and weathered soil is (2.5%L+8%F), (2%L+12%F), and (3%L+20%F), respectively. The addition of lime and fly ash together increased the unconfined compressive strength- and the strength gain-values of the three studied soils strongly compared to the addition of lime and fly ash separately. Lime/fly ash-tertiary clay mixtures have qu- and strength gain-values higher than the values of both lime/fly ash-organic silt and –weathered soil mixtures.

In the present study, the strength gain increased for all the three tested soils with an increase in both the lime and the fly ash contents, but this increase has upper limit at the optimum lime/fly ash ratio. The optimum ratio of tertiary clay and organic silt is 0.16 and 0.15, respectively (about 1 lime: 6 fly ash by weight) and the ratio of weathered soil is 0.14 (about 1 lime: 7 fly ash by weight).

* For the three studied stabilized soils, the elasticity modulus (E_{secant}) increased and the failure axial strain (ε_f) decreased as a consequence of either the separate or the joined effects of lime and fly ash contents. The E_{secant} increased and the failure axial strain (ε_f) decreased dramatically with the addition of both lime and fly ash together, especially in the case of tertiary clay. The mechanical behavior of the three studied soils was changed from ductile to brittle. This development was relatively weak for the acidic weathered soil.

The development of the mechanical behavior from ductile to brittle of the three stabilized soils was dramatic through the long-term curing, especially in the case of tertiary clay.

* In the case of durability (water-soaking) test, both fly ash- and lime/fly ash-organic silt mixtures had the lowest strength loss. Fly ash- and lime/fly ash-weathered soil mixtures had the highest strength loss.

All the tested stabilized mixtures passed successively in the freeze-thaw durability test. Three mixtures (lime/fly ash-tertiary clay and fly ash- and lime/fly ash-organic silt mixtures) passed successively in the wet-dry durability test and the other three mixtures (fly ash-tertiary clay and fly ash-and lime/fly ash-weathered soil mixtures) failed. Lime/fly ash-tertiary clay mixtures are more durable than fly ash-tertiary clay mixtures. Both fly ash- and lime/fly ash-organic silt mixtures are durable. In the case of wet-dry durability test, both fly ash-and lime/fly ash-weathered soil mixtures failed.

4 Results: California Bearing Ratio (CBR)

CBR-value is used as an index of soil strength and bearing capacity. This value is broadly used and applied in design of the base and the sub-base material for pavement. Lime- and fly ash–stabilized soils are often used for the construction of these pavement layers and also for embankments. CBR-value is a familiar indicator test used to evaluate the strength of soils for these applications (Nicholson et al., 1994). CBR-test was conducted to characterize the strength and the bearing capacity of the three studied soils and their mixtures with lime, fly ash, and lime/fly ash. The test procedures and the preparation of the specimens were achieved according to the procedures in chapter 2.4.2 and 2.5, respectively.

4.1 CBR of untreated compacted soils

CBR-values of the three tested soils, compacted at optimum water content, are given in Tables 2.8, 3.1, 3.2, and 3.3 (see chapter 2.3.4 and 3). All the specimens were prepared using a standard proctor test. Untreated compacted soil specimens (without chemical additives) of tertiary clay, organic silt, and weathered soil, compacted at optimum water content, had CBR-values of 4.6, 3.2, and 5.4%, respectively. CBR-values of untreated compacted soils need to be interpreted in the context of the general relationship between the CBR-values and the consistency (quality) of the soils used in pavement applications (Bowles, 1992). CBR-values ranging from 3 to 7% are considered as a poor to fair consistency. This means that the untreated compacted-tertiary clay, -weathered soil, and -organic silt belong to poor to fair consistency resulting from the compaction process at the optimum water content and without chemical additives "lime & fly ash" (see chapter 2.4.2 and Table 3.4).

4.2 CBR of treated stabilized soils

CBR-values of soil-lime, soil-fly ash, and soil-lime/fly ash mixtures prepared at optimum water content are given in Tables 3.1, 3.2, and 3.3 (see Appendixes 16, 17, & 18). CBR-value was measured after 7 days curing for the three studied soils.

Soil-lime mixtures of the three tested soils were prepared at the optimum lime content, 2%, and 4% above the optimum lime content (compacted at optimum water content) and cured for 7 days (see chapter 2.5.1). CBR-values are shown in Tables 3.1, 3.2, & 3.3 and in Figures 4.1 a & 4.2.

Soil-fly ash mixtures were prepared at 8, 12, 16, 20, and 25% fly ash for both the tertiary clay and the organic silt and at 8, 12, 16, 20, 25, and 35% fly ash for the weathered soil.

All the mixtures were compacted at optimum water content (two hours delay after the mixing) and cured for 7 days (see chapter 2.5.2).

Soil-fly ash mixtures of tertiary clay, organic silt, and weathered soil were prepared at the optimum fly ash content of 16, 20, and 35%, respectively (at optimum water content) and cured for 7, 28, 56, and 180 days to estimate the influence of curing time on the CBR-value and on the fly ash-stabilization process. The values of CBR are illustrated in Tables 3.1, 3.2, & 3.3 and in Figures 4.1 b & 4.2.

Soil-lime/fly ash mixtures were prepared at the optimum lime/fly ash contents, after pH-test, of (2.5%L/8%F), (2%L/12%F), and (3%L/20%F) for tertiary clay, organic silt, and weathered soil, respectively. Other mixtures were prepared at the optimum fly ash content with different lime percentages to evaluate the effect of the increase in the lime and the lime/fly ash ratio on the lime/fly ash-stabilization process (see chapter 2.5.3). All the mixtures were compacted at the optimum water content and cured for 7 days.

Soil-lime/fly ash mixtures of tertiary clay (2.5%L/16%F), organic silt (2%L/20%F), and weathered soil (3%L/35%F) were prepared at the optimum fly ash contents with small percentage of lime and at optimum water content (see chapter 2.5.3). The mixtures were cured for 7, 28, 56, and 180 days to evaluate the effect of the long-term curing on the CBR-value and on the lime/fly ash-stabilization process. CBR-values are illustrated in Tables 3.1, 3.2, & 3.3 and in Figures 4.1 c, 4.2, 4.3, & 4.4 (see Appendixes 16, 17, & 18).

4.2.1 General effect of lime-, fly ash- and lime/fly ash-stabilization process

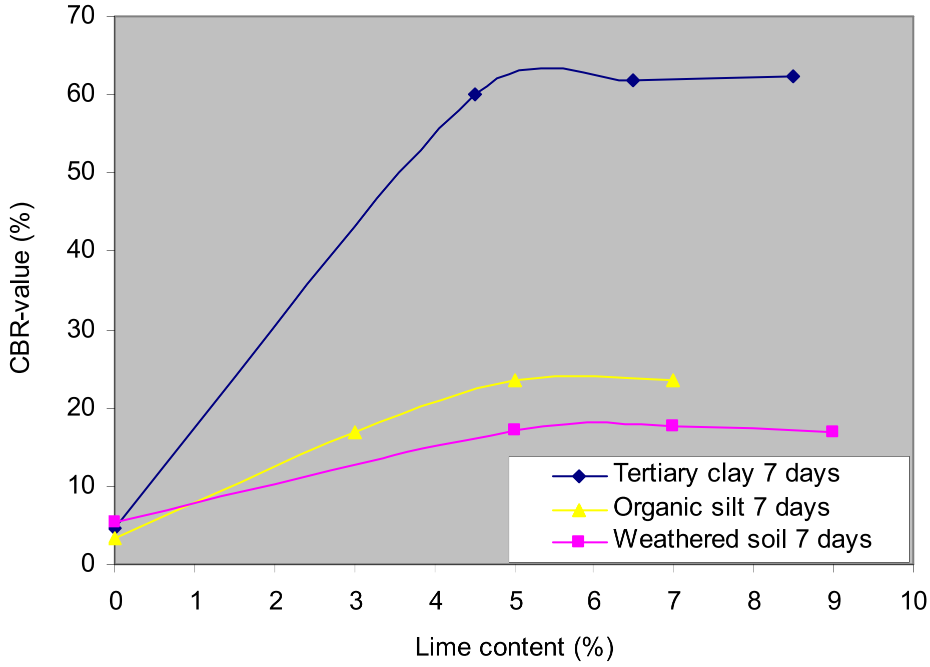
The general effect of lime-, fly ash- and lime/fly ash-stabilization process on the three studied soils is illustrated in Figure 4.1 a, b, and c, respectively. The addition of the optimum lime content led to an increase in the CBR-values for the three tested soils. The lime-tertiary clay mixtures have the highest CBR-values, whereas the lime-weathered soil mixtures have the lowest values. The reactivity of the tertiary clay with lime is stronger than the reactivity of both the weathered soil and the organic silt. The CBR-values of lime-tertiary clay mixtures increased slightly with increasing lime content (2 and 4% above the optimum lime content). The reactivity of both the organic silt and the weathered soil with lime is weak, according to the reactivity test (see chapter 2.5.1). The lime-organic silt mixtures have CBR-values relatively higher than the CBR-values of lime-weathered soil mixtures. CBR-values of both lime stabilized-organic silt and -weathered soil increased with increasing lime content (2% above the optimum lime content) and decreased slightly with continual increase in the lime content (4% above the optimum lime content) in case of the weathered soil and were not changed in case of the organic silt (see Fig. 4.1 a).

The ratio of the CBR-value of the lime-, fly ash-, and lime/fly ash-stabilized soil to that of the untreated compacted soil is known as the *CBR-gain factor*. The CBR-gain factors (due to the addition of the optimum lime content) of the tertiary clay, the organic silt, and the weathered soil are 13.1, 5.25, and 3.17, respectively. The final consistency (quality) of the mixtures is excellent, good, and fair, respectively (see Table 3.4 and Fig. 4.2).

The addition of fly ash contents resulted in an increase in the CBR-values for the three studied soils. Fly ash-tertiary clay mixtures have the highest CBR-values and fly ash-weathered soil mixtures have the lowest values, at the same fly ash contents.

The CBR-values of tertiary clay-fly ash mixtures increased with increasing the fly ash content until 20%. Above 20% fly ash (for example 25%), the CBR-value decreased. Similar behavior was obtained at measurement of the unconfined compressive strength (see chapter 3.4.1).

Fig. (4.1, a) California bearing ratio (CBR-value, laboratory) of untreated compacted- and treated stabilized-soils with lime



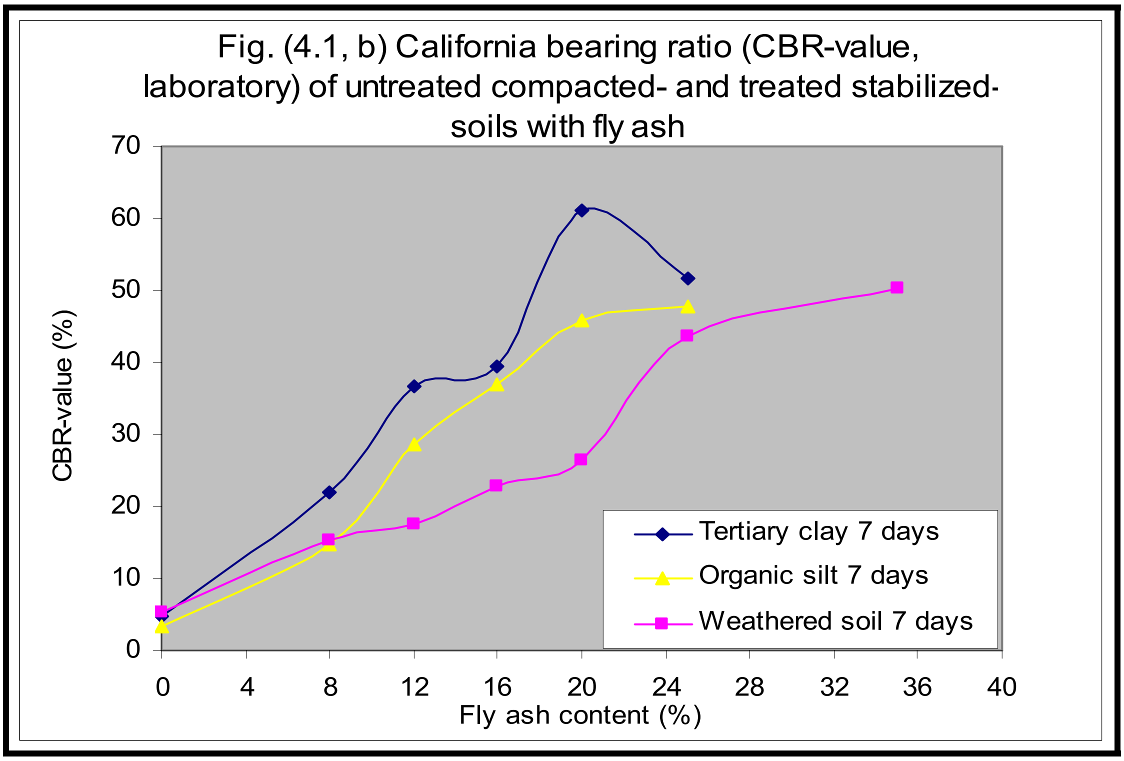


Fig. (4.1, b) California bearing ratio (CBR-value, laboratory) of untreated compacted- and treated stabilized-soils with fly ash

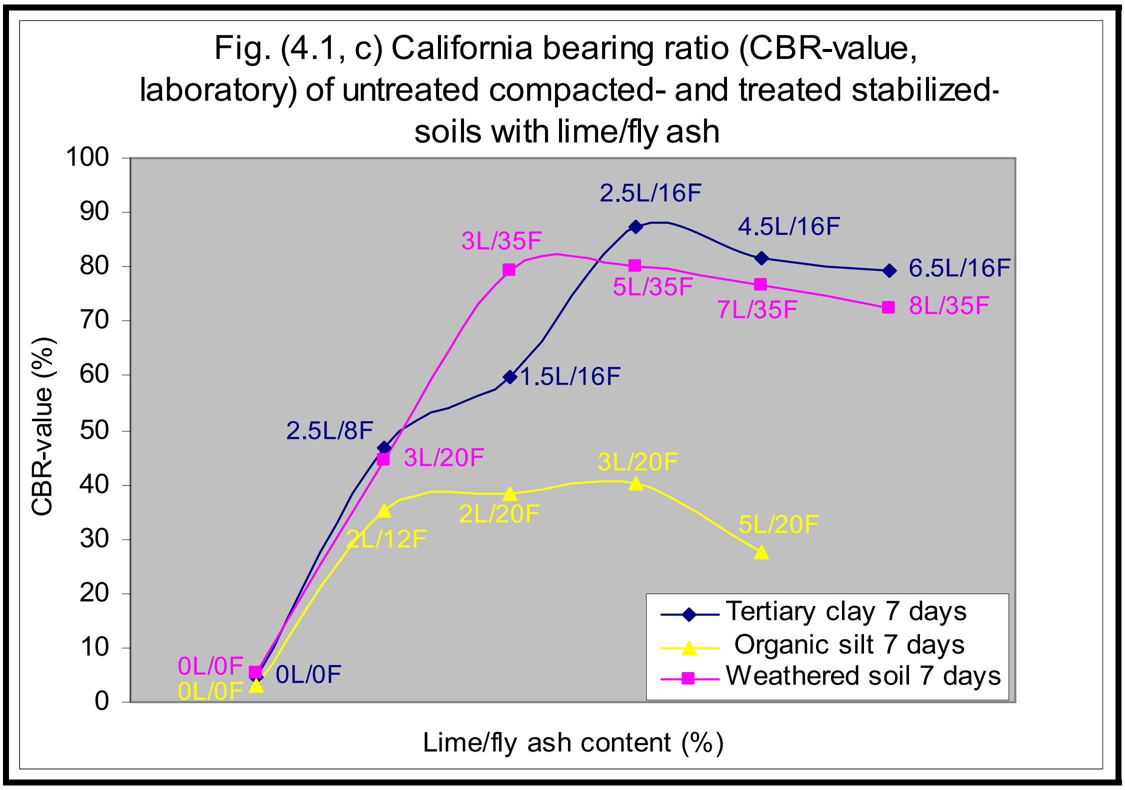


Fig. (4.1, c) California bearing ratio (CBR-value, laboratory) of untreated compacted- and treated stabilized-soils with lime/fly ash

The CBR-values of both fly ash-organic silt and -weathered soil mixtures are relatively lower than the CBR-values of fly ash-tertiary clay mixtures, at the same fly ash contents. Fly ash-organic silt mixtures have the CBR-values relatively higher than the CBR-values of fly ash-weathered soil mixtures, at the same fly ash contents. CBR-values for both fly ash-organic silt and -weathered soil mixtures increased with increasing the fly ash content.

CBR-gain factors of fly ash-organic silt mixtures are slightly higher than the CBR-gain factors of fly ash-tertiary clay mixtures, at the same fly ash contents. CBR-gain factors of fly ash-weathered soil mixtures are lower than the factors of both fly ash-organic silt and – tertiary clay mixtures, at the same fly ash contents. CBR-gain factors (due to the addition of the optimum fly ash content) of tertiary clay, organic silt, and weathered soil are 8.57, 14.31, and 9.44, respectively (see Fig. 4.2).

CBR-gain factor of both the organic silt and the weathered soil is higher than the factor of tertiary clay. The final consistency (quality) of optimum fly ash-tertiary clay, -organic silt, and -weathered soil mixtures is good, good, and excellent, respectively (see Table 3.4).

The addition of lime and fly ash together led to a dramatic increase in the CBR-values for the three studied soils compared to the addition of lime and fly ash separately. The CBR-values of tertiary clay-lime/fly ash mixtures are higher than both weathered soil- and organic silt-lime/fly ash mixtures. CBR-gain factors (due to the addition of the optimum lime/fly ash content after pH-method) of tertiary clay, organic silt, and weathered soil are 10.17, 11.00, and 8.24, respectively. CBR-gain factor (due to the addition of the optimum fly ash with small percentage of lime) of tertiary clay (2.5%L/16%F), organic silt (2%L/20%F), and weathered soil (3%L/35%F) is 18.96, 12.56, and 14.67, respectively (see Fig. 4.2). The final consistency of tertiary clay, organic silt, and weathered soil–lime/fly ash mixtures is excellent, good, and excellent, respectively (see Table 3.4). CBR-values increased with increasing lime/fly ash ratio. The optimum lime/fly ash ratio of tertiary clay, organic silt, and weathered soil is 0.16, 0.15, and 0.14, respectively. Above the ratios of 0.16 and 0.15 for both the tertiary clay and the organic silt, respectively (about 1 lime: 6 fly ash by weight) and above the ratio 0.14 (about 1 lime: 7 fly ash by weight) for the weathered soil, the CBR-values of the soils-lime/fly ash mixtures decreased (see Fig. 4.3).

This means that the increment of lime percentage above 16, 15, and 14% of the fly ash weight for the tertiary clay, the organic silt, and the weathered soil, respectively resulted in a decrement of the CBR-values of the soil-lime/fly ash mixtures.

4.2.2 Effect of curing time

Soil-fly ash and soil-lime/fly ash mixtures, for the three different tested soils, were prepared and cured for periods of 7, 28, 56, and 180 days to evaluate the effect of long-term curing on the CBR-value of the treated stabilized soils. All the specimens were prepared at the optimum water content.

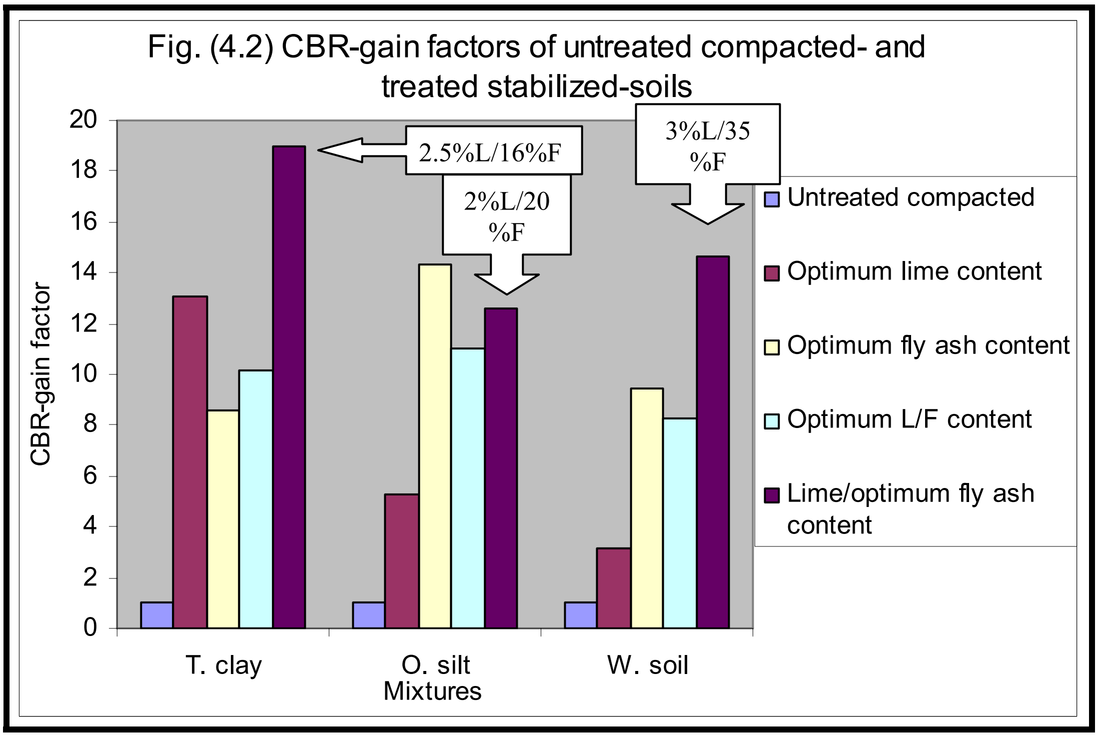


Fig. (4.2) CBR-gain factors of untreated compacted- and treated stabilized-soils

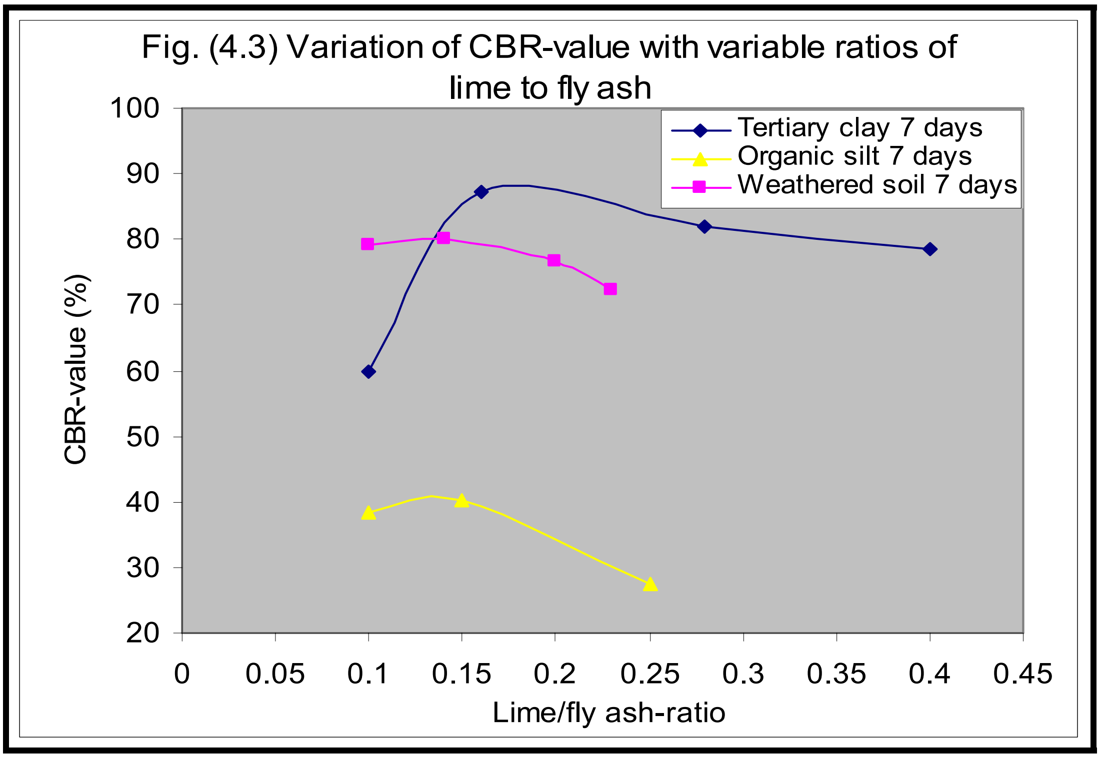


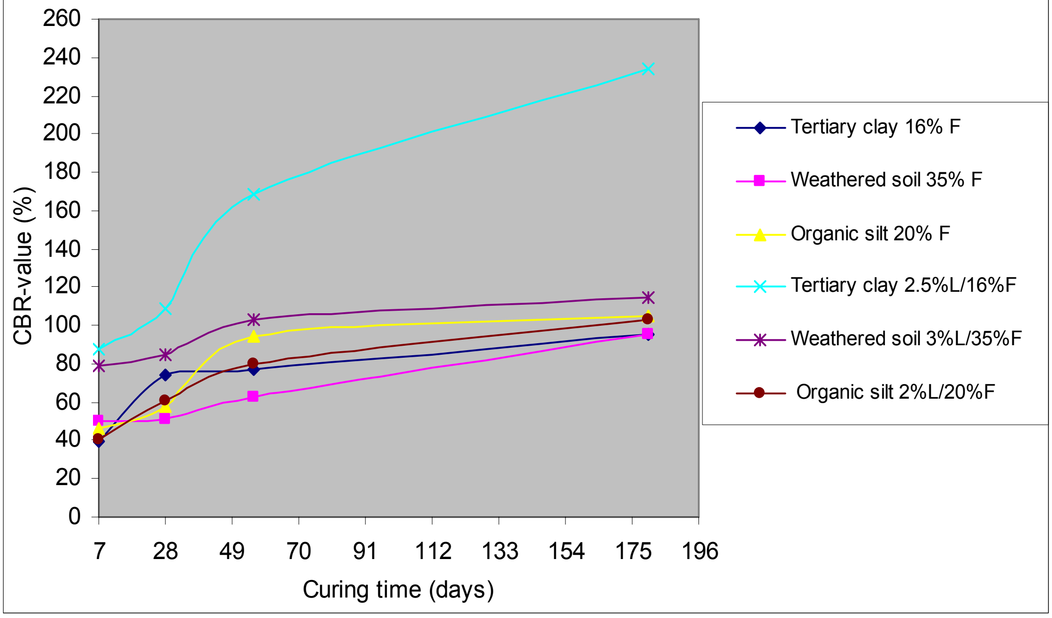
Fig. (4.3) Variation of CBR-value with variable ratios of lime to fly ash

The effect of curing time on CBR-value of soil-fly ash and soil-lime/fly ash mixtures is illustrated in Figure 4.4. The CBR-values of both soil-fly ash and soil-lime/fly ash mixtures increased with curing time in case of the three tested soils.

CBR-values of tertiary clay- and weathered soil-lime/fly ash mixtures were strongly affected by the long-term curing compared to tertiary clay- and weathered soil-optimum fly ash mixtures. Organic silt-optimum fly ash mixtures were strongly affected by curing time compared to organic silt-lime/fly ash mixtures. In general, the CBR-values of fly ash- and lime/fly ash-stabilized weathered soil were slightly affected by the long-term curing in comparison to CBR-values of both stabilized-tertiary clay and –organic silt.

CBR-values of tertiary clay-lime/fly ash mixture increased dramatically with the long-term curing compared to weathered soil- and organic silt-lime/fly ash mixtures (see Fig. 4.4 and Tables 3.1, 3.2, & 3.3).

Fig. (4.4) Effect of curring time on the CBR-value of fly ash- and lime/fly ash-stabilized soils



4.3 Conclusions

* In the case of lime-stabilization process, the addition of optimum lime content (according to pH-method) resulted in an increment of the CBR-values of the three studied soils. Tertiary clay has the highest CBR-values, whereas weathered soil has the lowest CBR-values. CBR-value of lime-stabilized tertiary clay increased with an increase in the lime content continually (2 and 4% above the optimum lime content). Another behavior of both the organic silt and the weathered soil was obtained, in which case the CBR-values increased at 2% above the optimum lime content. CBR-values, at 4% above the optimum, decreased slightly in the case of weathered soil and were not changed in the case of organic silt. CBR-gain factors (due to the addition of the optimum lime content) of tertiary clay, organic silt, and weathered soil are 13.1, 5.25, and 3.17, respectively. The final consistency (quality) of the mixtures is excellent, good, and fair, respectively.

* In the case of fly ash-stabilization process, the addition of fly ash led to an increase in the CBR-values for the three tested soils. CBR-values of both the organic silt and the weathered

soil increased with an increase in the fly ash content. CBR-values of tertiary clay increased with the increase in fly ash content from 8 to 20%. Above 20% fly ash (for example 25%), the CBR-values decreased. Similar behavior was observed at qu-measurement. The CBR-gain factors of fly ash-organic silt mixtures are slightly higher than the CBR-gain factors of fly ash-tertiary clay mixtures, at the same fly ash contents.

* In the case of lime/fly ash-stabilization process, the addition of lime and fly ash together increased the CBR-values of the three tested soils strongly compared to the addition of lime and fly ash separately. Lime/fly ash-tertiary clay mixtures have CBR-gain factors higher than lime/fly ash-organic silt and –weathered soil mixtures. The optimum ratio of lime to fly ash is 0.16 in case of the tertiary clay and 0.15 in case of the organic silt (about 1 lime : 6 fly ash) and is 0.14 (about 1 lime : 7 fly ash) in case of the weathered soil.

* CBR-values and CBR-gain factors of both the fly ash- and the lime/fly ash-mixtures, for the three tested soils, were improved and increased through the long-term curing. The influence of curing time was strong on the lime/fly ash-stabilization process compared to the fly ash-stabilization process, especially in case of the tertiary clay where the increase of CBR-value and CBR-gain factors was dramatic. The effect of curing time on the fly ash- and lime/fly ash-stabilization process in the case of weathered soil was weaker than the effect in the case of both tertiary clay and organic silt.

5 Results: Indirect tensile strength (σt)

Tensile strength is a very important geotechnical parameter to predict the cracking behavior of pavements, earth dams, and earth structures using stabilized soils (Baghdadi et al., 1995). Baghdadi et al. (1995) investigated the split tensile and the unconfined compressive strength of the dune sand specimens treated with 50 and 100% cement kiln dust, (CKD by-product) and cured for seven days at 10, 25, and 40°C temperature levels. They reported that both the tensile and the compressive strengths increased with an increase in the curing temperatures and the ratio of split tensile strength to unconfined compressive strength varied from 0.035 for specimens cured at 10°C to 0.13 for specimens cured at 40°C. Thompson (1966) reported similar results where the tensile strength to unconfined compressive strength ratio was approximately 0.13 for lime stabilized soils. Consoli et al. (2001) studied both unconfined compressive and tensile strengths for the stabilized soil (derived from weathered sandstone) treated with carbide lime and fly ash. They reported that both tensile strength and unconfined compressive strength increased through curing time (7, 28, 90, and 180 days) and the ratio of tensile to compressive strength increased with long-term curing but with different rates.

In the present study, split tensile strength test (Indirect tensile test) was conducted to characterize the tensile strength and the cracking behavior of the stabilized soils treated with lime, fly ash, and lime/fly ash. The test was carried out on cylindrical specimens (100 mm height * 100 mm diameter). Indirect tensile test procedures and the preparation of the specimens were performed according to the procedures in chapter 2.4.3 and 2.5, respectively.

5.1 Indirect tensile strength of treated stabilized soils

Tensile strength-values (σt-values) of soil-lime, soil-fly ash, and soil-lime/fly ash mixtures prepared at optimum water content are given in Tables 5.1, 5.2, and 5.3, respectively (see Appendixes 19, 20, & 21). Split tensile strength was measured after 7 days curing and after conduction of the p-wave velocity (Vp) test for the three different studied soils.

Soil-lime mixtures of the three studied soils were prepared at the optimum lime content, 2, and 4% above the optimum lime content. All the mixtures were compacted at optimum water content and cured for 7 days (see chapter 2.5.1). The values of tensile strength are illustrated in Tables 5.1, 5.2, & 5.3 and in Figure 5.1 a.

Soil-fly ash mixtures were prepared at 8, 12, 16, 20, and 25% fly ash for both the tertiary clay and the organic silt and at 8, 12, 16, 20, 25, and 35% fly ash for the weathered soil.

The mixtures were compacted at optimum water content (two hours delay after the mixing) and cured for 7 days (see chapter 2.5.2).

Soil-fly ash mixtures of tertiary clay, organic silt, and weathered soil were prepared at the optimum fly ash content (according to the pH-test) of 16, 20, and 35%, respectively (at optimum water content). The mixtures were cured for 7, 28, 56, and 180 days to judge the effect of the long-term curing on the tensile strength, on the cracking behavior, and on the fly ash-stabilization process. The values of tensile strength (σt-values) are shown in Tables 5.1, 5.2, & 5.3 and in Figure 5.1 b.

Table (5.1) Tensile strength, unconfined compressive strength, and tensile/compressive strength ratio of treated stabilized tertiary clay with several blending.

Admixture mixing		Curing time	qu-value	σt-value	σt/qu-ratio
L (%)	FA (%)	Days	kN/m^2	kN/m^2	-
4.5*	**0**	**7**	**1034.00**	**109.25**	**0.106**
6.5	**0**	**7**	**1147.80**	**160.25**	**0.140**
8.5	**0**	**7**	**1221.70**	**183.60**	**0.150**
0	**8**	**7**	**366.88**	**43.70**	**0.119**
0	**12**	**7**	**594.90**	**72.90**	**0.123**
0	**16***	**7**	**820.38**	**102.00**	**0.124**
		28	**1459.27**	**185.80**	**0.127**
		56	**1660.64**	**211.26**	**0.127**
		180	**1791.37**	**276.82**	**0.155**
0	**20**	**7**	**1064.97**	**153.00**	**0.144**
0	**25**	**7**	**950.00**	**152.95**	**0.161**
2.5*	**8***	**7**	**905.59**	**123.85**	**0.137**
1.5	**16**	**7**	**1424.42**	**131.15**	**0.092**
2.5	**16**	**7**	**1476.28**	**142.06**	**0.096**
		28	**2256.57**	**240.4**	**0.107**
		56	**3081.50**	**335.10**	**0.109**
		180	**3505.23**	**400.66**	**0.114**
4.5	**16**	**7**	**1363.75**	**138.41**	**0.102**
6.5	**16**	**7**	**1159.56**	**127.49**	**0.110**

Notes:
L = Lime content
FA = Fly ash content
qu = Unconfined compressive strength
σt = Tensile strength
* Optimum content according to pH-test

Soil-lime/fly ash mixtures were prepared at the optimum lime/fly ash contents (after the pH-test) of tertiary clay (2.5%L/8%F), organic silt (2%L/12%F), and weathered soil (3%L/20%F). Other mixtures were prepared at the optimum fly ash content with different lime percentages to judge the influence of the increment of the lime and the lime/fly ash ratio on the tensile strength values and on the lime/fly-ash stabilization process (see chapter 2.5.3). The mixtures were compacted at the optimum water content and cured for 7 days.

Table (5.2) Tensile strength, unconfined compressive strength, and tensile/compressive strength ratio of treated stabilized organic silt with several blending.

Admixture mixing		Curing time	qu-value	σt-value	σt/qu-ratio
L (%)	FA (%)	Days	kN/m²	kN/m²	-
3*	0	7	439.49	87.40	0.199
5	0	7	634.40	109.30	0.172
7	0	7	628.03	101.95	0.162
0	8	7	322.29	37.90	0.118
0	12	7	532.48	58.30	0.110
0	16	7	620.38	72.85	0.117
0	20*	7	685.35	76.49	0.112
		28	1331.26	153.00	0.115
		56	1385.55	167.55	0.121
		180	1738.98	254.97	0.147
0	25	7	964.33	109.27	0.113
2*	12*	7	729.65	65.55	0.090
2	20	7	866.43	80.13	0.092
		28	1348.78	152.98	0.113
		56	1648.44	189.41	0.115
		180	2148.35	258.61	0.120
3	20	7	871.88	83.78	0.096
5	20	7	796.11	72.90	0.092

Table (5.3) Tensile strength, unconfined compressive strength, and tensile/compressive strength ratio of treated stabilized weathered soil with several blending.

Admixture mixing		Curing time	qu-value	σt-value	σt/qu-ratio
L (%)	FA (%)	Days	kN/m²	kN/m²	-
5*	0	7	309.55	47.35	0.153
7	0	7	329.90	50.95	0.154
9	0	7	298.09	40.05	0.134
0	8	7	294.27	36.40	0.124
0	12	7	301.54	40.04	0.133
0	16	7	335.50	50.95	0.152
0	20	7	389.81	69.20	0.178
0	25	7	526.12	72.90	0.138
0	35*	7	938.85	110.75	0.118
		28	1151.07	138.41	0.120
		56	1184.21	152.98	0.129
		180	1412.79	182.12	0.129
3*	20*	7	713.39	109.30	0.153
3	35	7	1327.30	116.56	0.088
		28	1619.81	145.70	0.090
		56	1751.11	160.27	0.092
		180	2297.16	236.80	0.103
5	35	7	1421.54	182.10	0.128
7	35	7	1416.49	174.80	0.123
8	35	7	1376.68	149.34	0.109

Soil-lime/fly ash mixtures (prepared at the optimum fly ash content with small percent of lime) of tertiary clay (2.5%L/16%F), organic silt (2%L/20%F), and weathered soil (3%L/35%F) were prepared at the optimum water content and cured for 7, 28, 56, and 180 days to estimate the effect of long-term curing on the tensile strength, on the cracking behavior, and on the lime/fly ash-stabilization process. The values of tensile strength are illustrated in Tables 5.1, 5.2, & 5.3 and in Figures 5.1 c, 5.2, 5.3, & 5.4.

5.1.1 General effect of lime-, fly ash-, and lime/fly ash-stabilization process

The general influence of lime-, fly ash-, and lime/fly ash-stabilization on the three different studied soils is illustrated in Figures 5.1 a, b, and c, respectively. The addition of the optimum lime content resulted in an increment of the tensile strengths for the three tested soils. Lime-tertiary clay mixtures have the highest tensile strength values and lime-weathered soil mixtures have the lowest values. The reactivity of tertiary clay with lime is stronger than the reactivity of both organic silt and weathered soil. In case of the tertiary clay, the σt-values increased with an increase in the lime content (2 and 4% above the optimum lime content). The reactivity of both the organic silt and the weathered soil with lime are weak, according to the reactivity test (see chapter 2.5.1). Organic silt-lime mixtures have tensile strength values relatively higher than the values of weathered soil-lime mixtures. The tensile strengths for both the organic silt- and the weathered soil-lime mixtures increased with an increase in the lime content (2% above the optimum lime content) and decreased with continual increase in the lime content, 4% above the optimum lime content (see Fig. 5.1 a). The ratio of tensile strength to unconfined compressive strength of lime stabilized-tertiary clay, -organic silt, and -weathered soil (at the optimum lime content) is 0.106, 0.199, and 0.153, respectively.

The addition of fly ash contents to the three tested soils led to an increase in the tensile strengths. Fly ash-tertiary clay mixtures have the highest tensile strength values and fly ash-weathered soil mixtures have the lowest values, at the same fly ash contents. The tensile strength values increased with increasing fly ash content from 8 to 20%. Above the 20% fly ash, for example 25%, the tensile strength values decreased. Similar behavior was observed at the measurement of unconfined compressive strength and California bearing ratio (see chapter 3.4.1 and 4.2.1).

Tensile strength values of both the fly ash-organic silt and the -weathered soil mixtures are lower than the σt-values of fly ash-tertiary clay mixtures, at the same fly ash contents. Fly ash-stabilized organic silt has tensile strength values relatively higher than fly ash-stabilized weathered soil, at the same fly ash contents. The tensile strength values of both the organic silt and the weathered soil increased with increasing the fly ash content (see Fig. 5.1 b). The ratio of tensile strength to unconfined compressive strength of fly ash stabilized-tertiary clay, -organic silt, and -weathered soil (at the optimum fly ash content) is 0.124, 0.112, and 0.118, respectively.

The addition of lime and fly ash together to the three tested soils led to a strong increase in the tensile strength values compared to the addition of lime and fly ash separately. Tertiary clay-optimum lime/fly ash mixtures have tensile strength values higher than the values of both the weathered soil- and the organic silt-optimum lime/fly ash mixtures. The ratio of tensile strength to unconfined compressive strength of optimum lime/fly ash stabilized-tertiary clay, -organic silt, and -weathered soil is 0.137, 0.090, and 0.153, respectively.

The addition of optimum fly ash with different percentages of lime contributed to increasing tensile strength values of the stabilized soils. Lime/fly ash-stabilized weathered soil has tensile strength values higher than the tensile strength values of both the tertiary clay and the organic silt at the optimum lime/fly ash ratio. Organic silt-lime/fly ash mixtures have the lowest tensile strength values compared to the other two soils-lime/fly ash mixtures. With increasing lime percentage, the tensile strength values have further increased until the ratio of lime to fly ash equals to 0.16 in case of the tertiary clay, 0.15 in case of the organic silt (about1 lime: 6 fly ash by weight) and 0.14 in case of the weathered soil (about 1lime: 7 fly ash by weight). Above these ratios, the tensile strengths decreased (see Fig. 5.1 c & 5.2). Similar behavior was noticed at both qu- and CBR-values (see chapter 3.4.1 and 4.2.1).

5.1.2 Effect of curing time

Soil-fly ash and soil-lime/fly ash mixtures for the three studied soils were prepared and cured for the periods of 7, 28, 56, and 180 days to estimate how the long-term curing affects the tensile strength and the cracking behavior of the stabilized soils. The tensile strength test was performed on the specimens after carrying out of the Vp-velocity test. All the mixtures were prepared at the optimum water content. The effect of curing time on the tensile strength of soil-fly ash and soil-lime/fly ash mixtures is shown in Figure 5.4. The tensile strength values of both soil-fly ash and soil-lime/fly ash mixtures, in case of the three tested soils, increased with the long-term curing. The curing time has a stronger effect on the tensile strength values of soil-lime/fly ash mixtures compared to soil-fly ash mixtures. The tensile strength values of tertiary clay-optimum fly ash mixture were strongly affected by the lone-term curing compared to the values of both the weathered soil- and the organic silt-optimum fly ash mixtures.

Fig. (5.1, a) Effect of lime content on tensile strength

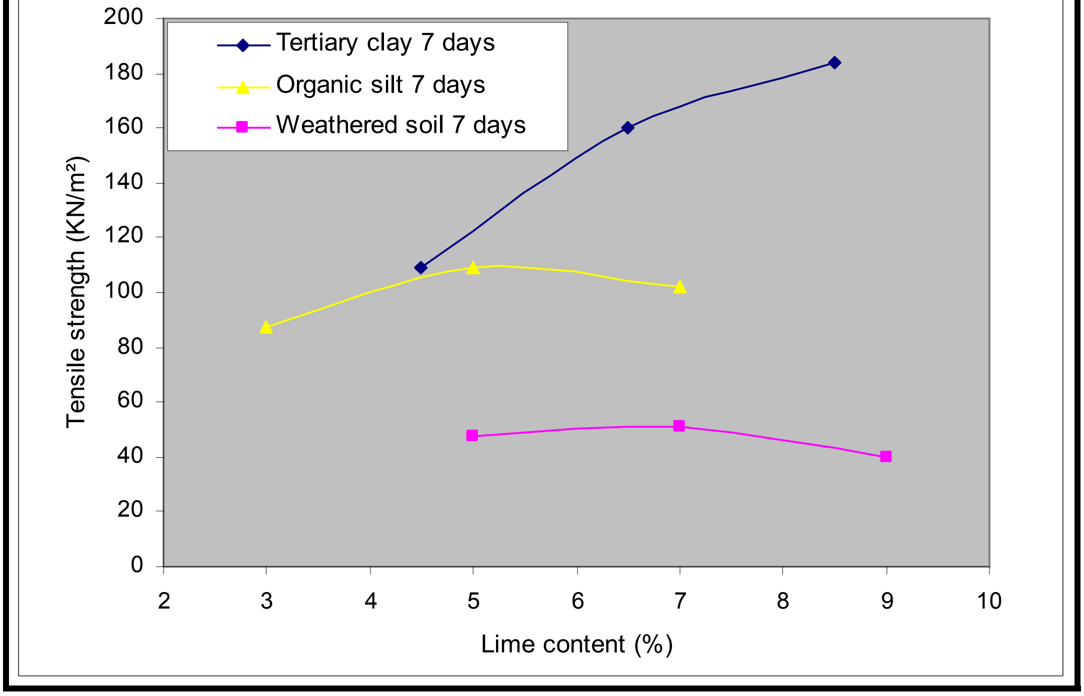


Fig. (5.1, b) Effect of fly ash content on tensile strength

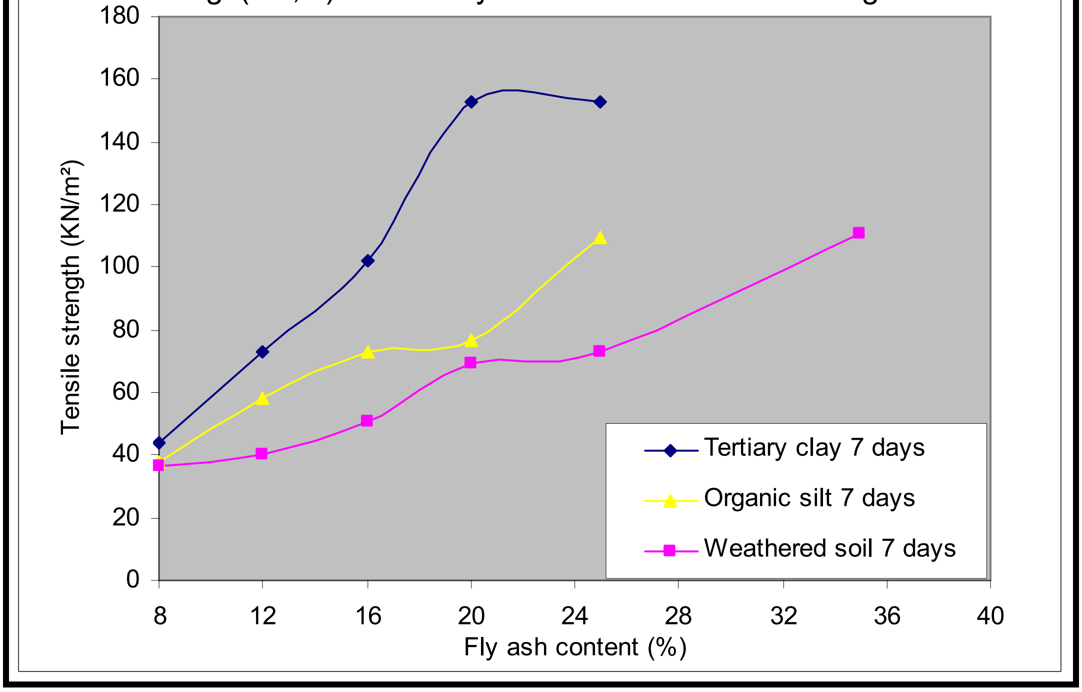


Fig. (5.1, c) Effect of lime/fly ash content on tensile strength

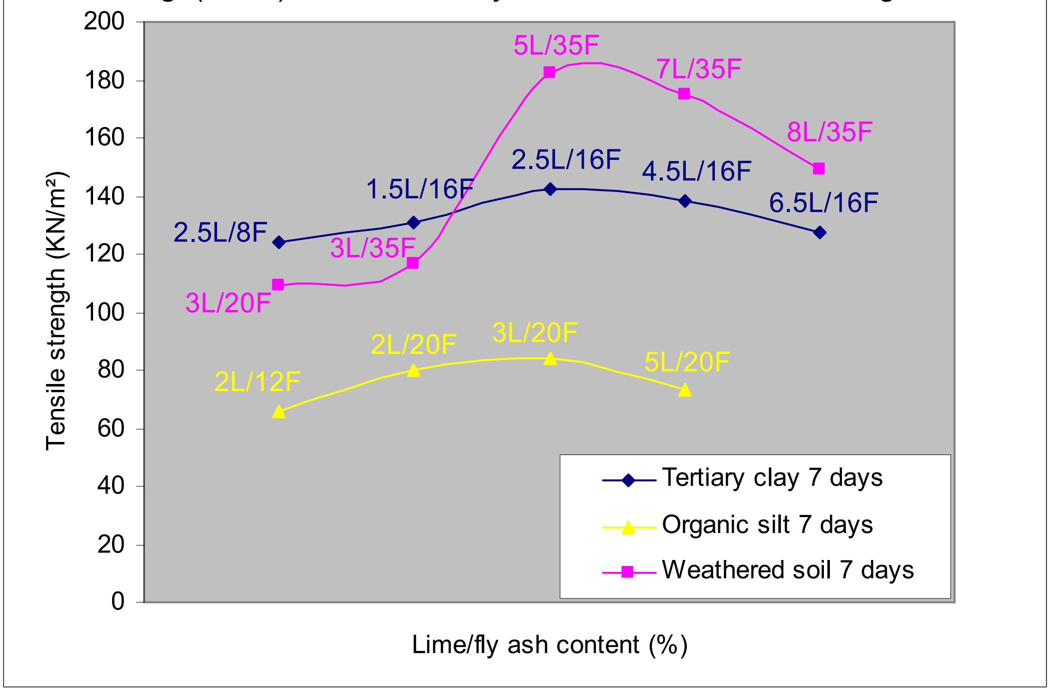
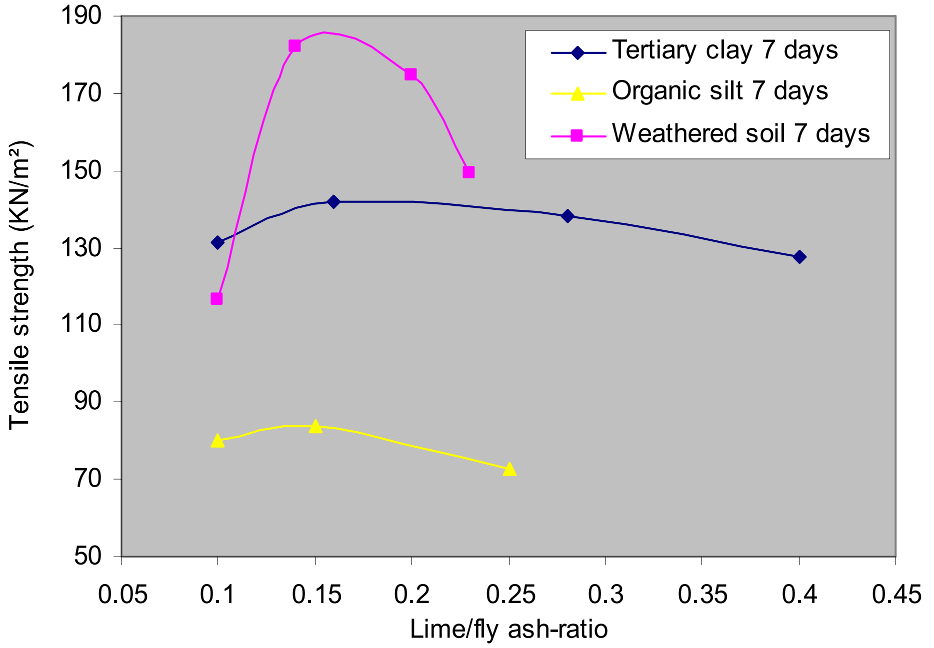


Fig. (5.2) Response of tensile strength to variable lime/fly ash-ratios



The curing time has the lowest effect on the tensile strength values of weathered soil-optimum fly ash mixtures. The tensile strength values of tertiary clay-lime/fly ash mixtures were dramatically increased with curing time compared to both the weathered soil- and the organic silt-lime/fly ash mixtures (see Fig. 5.4 and Tables 5.1, 5.2, & 5.3).

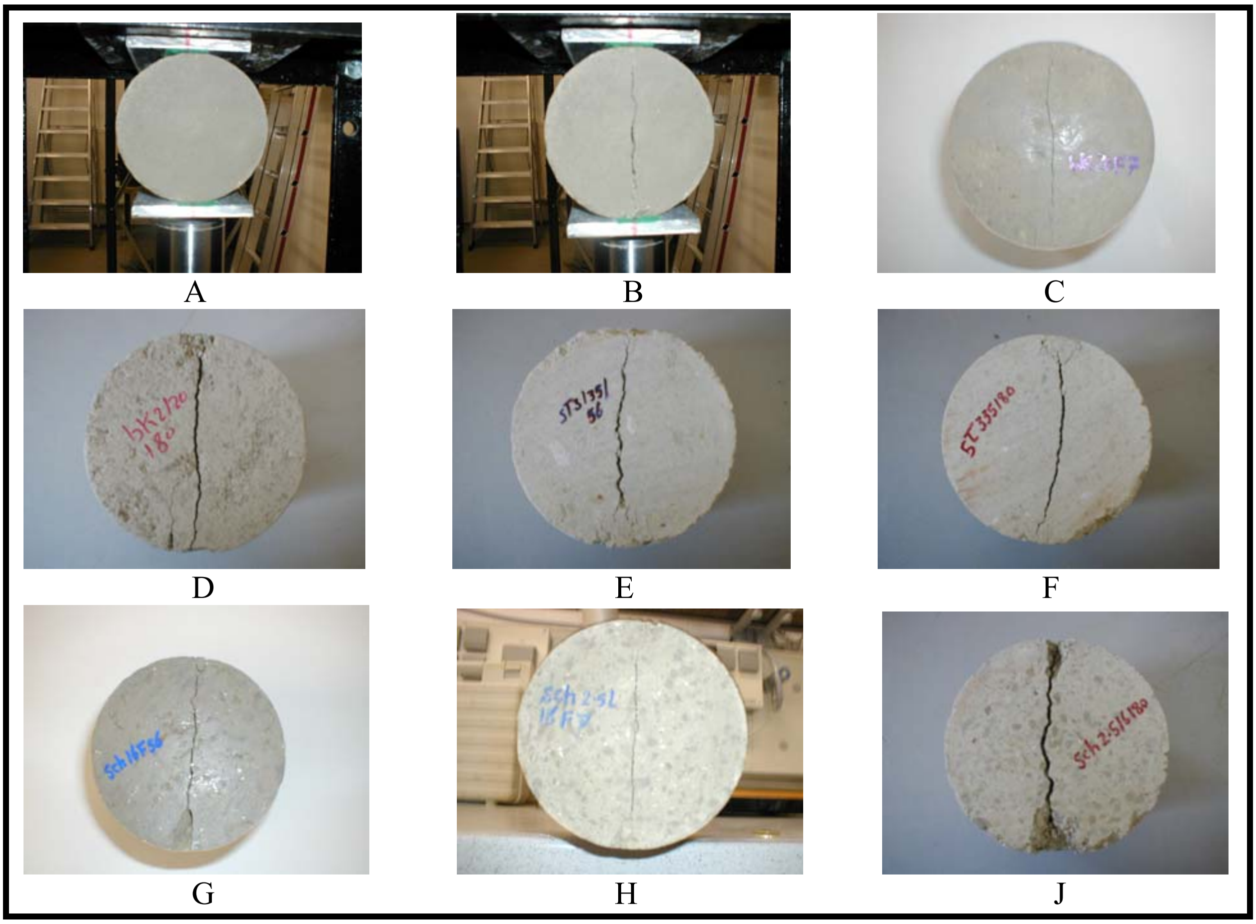


Fig. (5.3) Photos of different treated stabilized soils illustrate the tensile fractures after indirect tensile strength test.

A & B) Organic silt with 3% lime + 20% fly ash (7 days) before and after the test, respectively. C) Organic silt with 20% fly ash (7 days). D) Organic silt with 2% lime + 20% fly ash (180 days). E) Weathered soil with 3% lime + 35% fly ash (56 days).
F) Weathered soil with 3% lime + 35% fly ash (180 days). G) Tertiary clay with 16% fly ash (56 days). H) Tertiary clay with 2.5% lime + 16% fly ash (7 days).
J) Tertiary clay with 2.5%lime +16% fly ash (180 days).

The relative relationship and mobilization between the tensile strength and compressive strength could be discussed by plotting the tensile/compressive strength ratio (σt/qu-ratio) versus curing time. Figure 5.5 shows the effect of curing time on σt/qu-ratio of both the fly ash- and the lime/fly ash-stabilized soils. The average values of the ratio in case of the fly ash-stabilized soils are higher than the average value in case of the lime/fly ash-stabilized soils. The rate of increase of σt/qu-ratio of soil-fly ash mixtures from 7 to 28 days is smaller than the rate of soil-lime/fly ash mixtures at the same time interval. Conversely, the rate of soil-fly ash mixtures from 56 to 180 days is relatively larger than the rate of soil-lime/fly ash mixtures at the same time interval, except for the weathered soil.

Fig. (5.4) Effect of curing time on tensile strength of fly ash- and lime/fly ash-stabilized soils

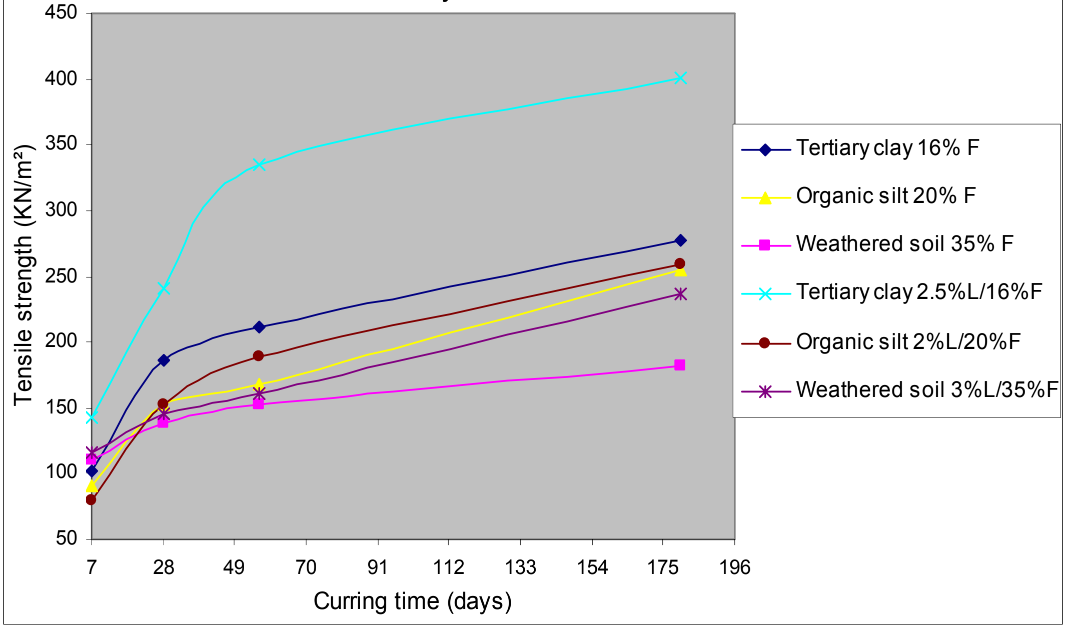
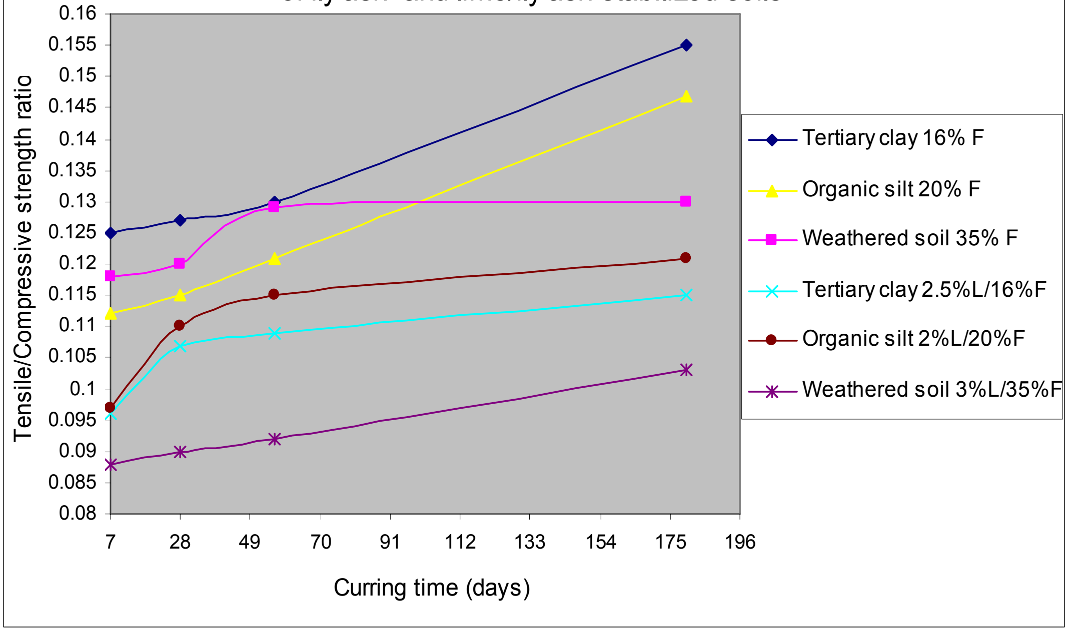


Fig. (5.5) Effect of curing time on tensile/compressive strength ratio of fly ash- and lime/fly ash-stabilized soils



5.2 Conclusions

* In the case of lime-stabilization process, the addition of the optimum lime content (after the pH-method) led to an increase in the σt-values of the three tested soils. Tertiary clay has the highest σt-values, whereas weathered soil has the lowest σt-values.

The tensile strength values of lime-stabilized tertiary clay increased with an increase in the lime content continually (2 and 4% above the optimum lime content). Another behavior of both lime stabilized-organic silt and -weathered soil was observed, where σt-values increased at 2% above the optimum lime content, but at 4% above the optimum lime content, the tensile strength values decreased slightly. Relatively, similar behavior was obtained at qu- and CBR-values measurement.

Tensile/unconfined compressive strength ratio (σt/qu-ratio), due to the addition of the optimum lime content, of tertiary clay, organic silt, and weathered soil is 0.106, 0.199, and 0.153, respectively.

* In the case of fly ash-stabilization process, the addition of fly ash resulted in an increase in the σt-values for the three tested soils. The tensile strength values of both the organic silt and the weathered soil increased with an increase in the fly ash content. The tensile strength values of fly ash-stabilized tertiary clay increased with an increase in the fly ash content from 8 to 20%. Above the 20% fly ash (for example 25%), σt-values decreased slightly. The tensile strength values of fly ash-organic silt mixtures are lower than the σt-values of fly ash-

inorganic tertiary clay mixtures and higher than the σt-values of fly ash-weathered soil mixtures, at the same fly ash contents. The σt/qu-ratio of optimum fly ash-tertiary clay, -organic silt, and -weathered soil mixtures is 0.124, 0.112, and 0.118, respectively.

* In the case of lime/fly ash-stabilization process, the addition of lime and fly ash together increased σt-values of the three tested soils strongly compared to the addition of lime and fly ash separately. Optimum lime/fly ash-tertiary clay mixtures have σt-values higher than the values of optimum lime/fly ash-organic silt and –weathered soil mixtures. The optimum ratio of lime to fly ash is 0.16 in case of the tertiary clay and 0.15 in case of the organic silt (about 1lime: 6 fly ash) and 0.14 (about 1lime: 7 fly ash) in case of the weathered soil. At these ratios, the maximum σt-values of lime/fly ash-soil mixtures were obtained. Similar behavior was reported at measurement of both the qu- and the CBR-values.

* The tensile strength-values of both the fly ash- and the lime/fly ash-mixtures, for the three tested soils, improved and increased by the long-term curing. The influence of curing time was strong on the lime/fly ash-stabilization process compared to the influence on the fly ash-stabilization process, especially with the tertiary clay. The effect of curing time on the σt-values for the fly ash- and lime/fly ash-stabilized weathered soil was weaker than the effect on σt-values of both fly ash and lime/fly ash stabilized-tertiary clay and -organic silt.

* The average values of the σt/qu-ratio in case of the fly ash-stabilized soils are higher than the average value in case of the lime/fly ash-stabilized soils. In the case of fly ash-stabilization, the organic silt has the lowest ratio-values and the tertiary clay has the highest values. In the case of lime/fly ash-stabilization, the weathered soil has the lowest ratio-values and the organic silt has the highest values.

6 Results: Hydraulic conductivity (K)

One of the aims of this study is an investigation of the hydraulic conductivity for the fly ash- and lime/fly ash-treated stabilized soils, especially with long-term curing. Previous studies provide very limited data concerning the hydraulic conductivity (K-value) of the fly ash and lime/fly ash-treated stabilized soils. Townsend & Kylm (1966), Brandl (1981), Nablantoglu & Tuncer (2001), and Nablantoglue & Gucbilmez (2002) reported that the K-value of the treated stabilized soils increased. Conversely, Terashi et al. (1980) and Locate et al. (1996) noted a diminution of the hydraulic conductivity of the treated stabilized soils. According to ASTM D 5239, "Standard practice for characterizing fly ash for use in soil stabilization", the use of self-cementing fly ash can improve the soil properties and reduce the permeability (K-value).

Thus, the permeability of treated stabilized soils is a complex problem and needs in-depth clarification, especially with the long-term curing.

6.1 Hydraulic conductivity of natural and untreated compacted soils

Hydraulic conductivity test according to the laboratory test of DIN 18 130-1 (see chapter 2.4.4) was conducted to characterize the permeability of both the natural and the untreated compacted soils. The K-values of the three tested natural soils are illustrated in Table 2.7. The K-values of natural tertiary clay, organic silt, and weathered soil are 1.9E-11, 5.5E-07, and 3.2E-11 m/sec, respectively. The permeability of tertiary clay, organic silt, and weathered soil (according to DIN 18 130-1) was classified as very poor permeable, poor permeable, and very poor permeable, respectively. The K-value of untreated compacted-tertiary clay, -organic silt, and -weathered soil is 1.7E-11, 8.5E-11, and 2.8E-11, respectively. The permeability of untreated compacted-tertiary clay, -organic silt, and -weathered soil was classified as very poor permeable. The compaction process, without chemical additives, in case of the three tested soils, led to an improvement of the geotechnical properties in the form of a reduction of the permeability. The hydraulic conductivity of organic silt was strongly affected by the compaction process (without chemical additives) compared to the permeability of both tertiary clay and weathered soil.

6.2 Hydraulic conductivity of treated stabilized soils

Hydraulic conductivity of soil-fly ash and soil-lime/fly ash mixtures of tertiary clay, organic silt, and weathered soil (prepared at optimum water content) are given in Tables 6.1, 6.2, and 6.3, respectively (see Appendixes 22, 23, & 24). Soil-fly ash mixtures were prepared at the optimum fly ash contents of 16, 20, and 35% fly ash for tertiary clay, organic silt, and weathered soil, respectively. All the mixtures were compacted at the optimum water content, two hours delay after the mixing. The mixtures were cured for 7 days (at 25°C & 98% humidity) to estimate the effect of fly ash addition on the K-value. Other mixtures were cured for 28, 56, and 180 days to investigate the influence of the long term curing on the K-value. Soil-lime/fly ash mixtures were prepared at (the optimum fly ash content with small percentage of lime) (2.5%lime+16%fly ash), (2%lime+20%fly ash), and (3%lime+35%fly ash) for tertiary clay, organic silt, and weathered soil, respectively. All the mixtures were prepared at the optimum water content and cured for 7 days to evaluate the effect of lime/fly ash addition on the K-value. Other mixtures were cured for 28, 56, and 180 days to study the influence of the long-term curing on the K-value. The K-values of untreated compacted- and treated stabilized- soils are showed in Tables 6.1, 6.2, & 6.3 and in Figures 6.1 a, b, & c.

Table (6.1) K-value and K-value gain factor of untreated compacted- and treated stabilized-tertiary clay.

Admixture mixing		Curing time	K-value	K-value gain factor
L (%)	FA (%)	Days	m/sec	-
0	**0**	**-**	**1.7E-11**	**1.00E+00**
0	**16***	**7**	**1.1E-10**	**6.47E+00**
		28	**6.4E-09**	**2.72E+02**
		56	**4.1E-09**	**2.41E+02**
		180	**1.6E-09**	**9.46E+01**
2.5	**16**	**7**	**1.5E-10**	**8.82E+02**
		28	**2.6E-09**	**1.53E+02**
		56	**5.5E-10**	**3.24E+01**
		180	**5.2E-10**	**3.06E+01**

Notes:
L = Lime content
FA = Fly ash content
K-value = Hydraulic conductivity
K-value gain factor = K-value of treated stabilized soil/ K-value of untreated compacted soil
* Optimum content according to pH-test

Table (6.2) K-value and K-value gain factor of untreated compacted- and treated stabilized-organic silt.

Admixture mixing		Curing time	K-value	K-value gain factor
L (%)	FA (%)	Days	m/sec	-
0	**0**	**-**	**8.5E-11**	**1.00E+00**
0	**20***	**7**	**2.4E-09**	**2.82E+01**
		28	**5.7E-09**	**6.71E+01**
		56	**3.8E-09**	**4.47E+01**
		180	**1.3E-09**	**1.53E+01**
2	**20**	**7**	**5.1E-08**	**6.00E+02**
		28	**1.8E-07**	**7.12E+02**
		56	**4.3E-08**	**5.06E+02**
		180	**1.8E-08**	**2.10E+02**

Table (6.3) K-value and K-value gain factor of untreated compacted- and treated stabilized-weathered soil.

Admixture mixing		Curing time	K-value	K-value gain factor
L (%)	FA (%)	Days	m/sec	-
0	**0**	**-**	**2.8E-11**	**1.00E+00**
0	**35***	**7**	**2.2E-07**	**2.60E+03**
		28	**1.0E-07**	**1.59E+03**
		56	**3.7E-08**	**1.30E+03**
		180	**2.0E-08**	**7.14E+02**
3	**35**	**7**	**1.8E-09**	**6.40E+01**
		28	**4.1E-08**	**1.50E+03**
		56	**4.1E-09**	**1.50E+02**
		180	**2.6E-09**	**9.29E+01**

6.2.1 General effect of fly ash- and lime/fly ash-stabilization process

The general influence of fly ash- and lime/fly ash-stabilization process on the hydraulic conductivity (K-value) of tertiary clay, organic silt, and weathered soil is shown in Figures 6.1 a, b, and c, respectively. The addition of fly ash increased the K-value of the three tested soil compared to the untreated compacted soils. The K-value gain factor (after 7 days curing) of tertiary clay, organic silt, and weathered soil is 6.47E+00, 2.82E+01, and 2.60E+03, respectively (see Fig. 6.2 a, b, & c). Tertiary clay-fly ash mixture has the lowest K-value gain factor and weathered soil-fly ash mixture has the highest K-value gain factor.

The permeability of fly ash stabilized-tertiary clay, -organic silt, and -weathered soil was classified (according to DIN 18 130-1) as very poor, very poor, and poor permeable, respectively. Addition of lime and fly ash together has shown an increase in the K-value for the three tested soils compared to the untreated compacted soils. The K-value gain factor (after 7 days curing) of tertiary clay, organic silt, and weathered soil is 8.827E+02, 6.00E+02, and 6.40E+01, respectively. Weathered soil-lime/fly ash mixture has the lowest K-value gain factor and tertiary clay-lime/fly ash mixture has the highest K-value gain factor. The permeability of lime/fly ash stabilized-tertiary clay, -organic silt, and -weathered soil was classified (according to DIN 18 130-1) as very poor, poor, and very poor permeable respectively.

6.2.2 Effect of curing time

The K-value and the K-value gain factor of soil-fly ash and soil-lime/fly ash mixtures for the three tested soils increased strongly from 7 days to 28 days curing time, except for weathered soil-fly ash mixtures. With an increase in the curing time (56 and 180 days), the K-values and the K-value gain factors of the three tested stabilized soils decreased (see Tables 6.1, 6.2, & 6.3, Fig. 6.1 a, b, & c, and Fig. 6.2 a, b, & c in Appendixes 25, 26, & 27, respectively).

Fig. (6.1, a) Effect of curing time on the hydraulic conductivity (K-value) of fly ash- and lime/fly ash-stabilized tertiary clay

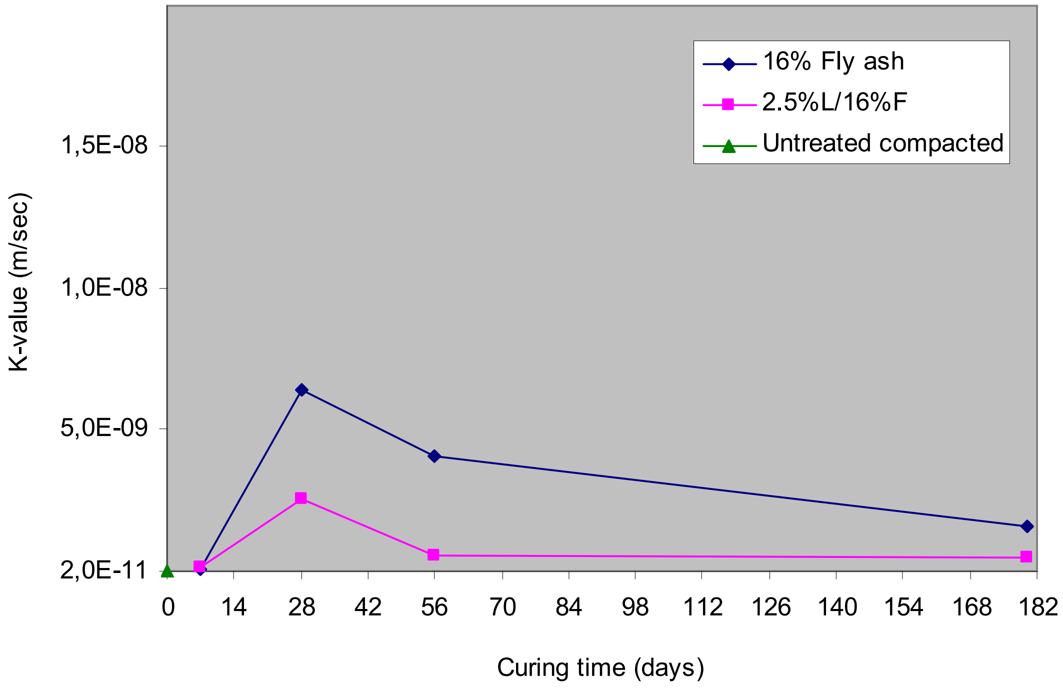


Fig. (6.1, b) Effect of curing time on the hydraulic conductivity (K-value) of fly ash- and lime/fly ash-stabilized organic silt

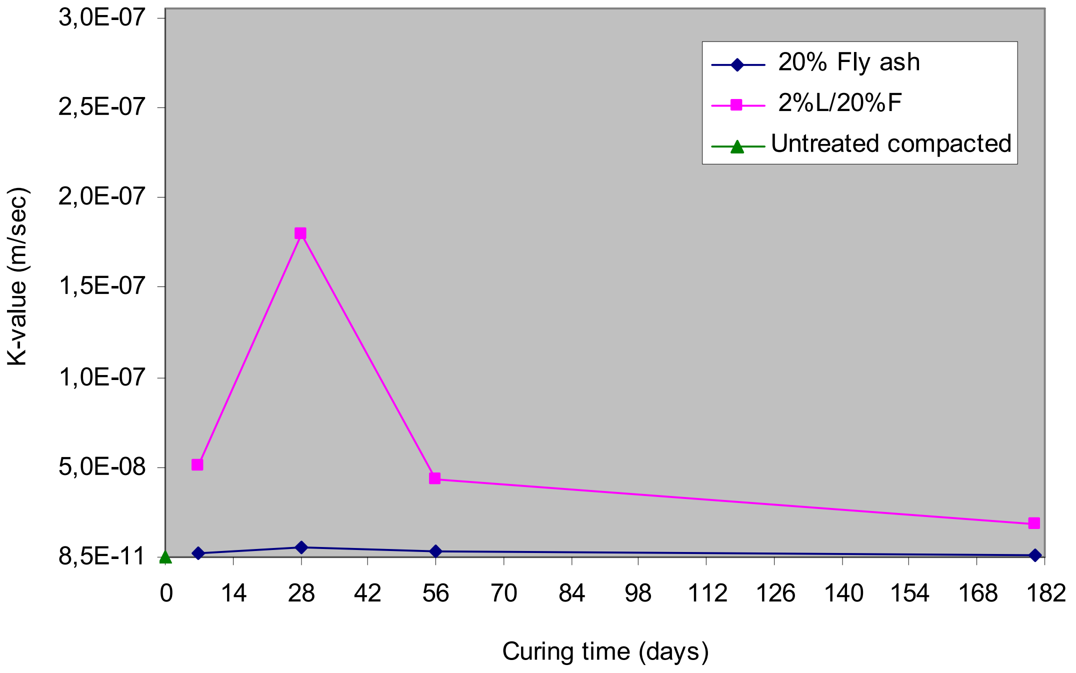
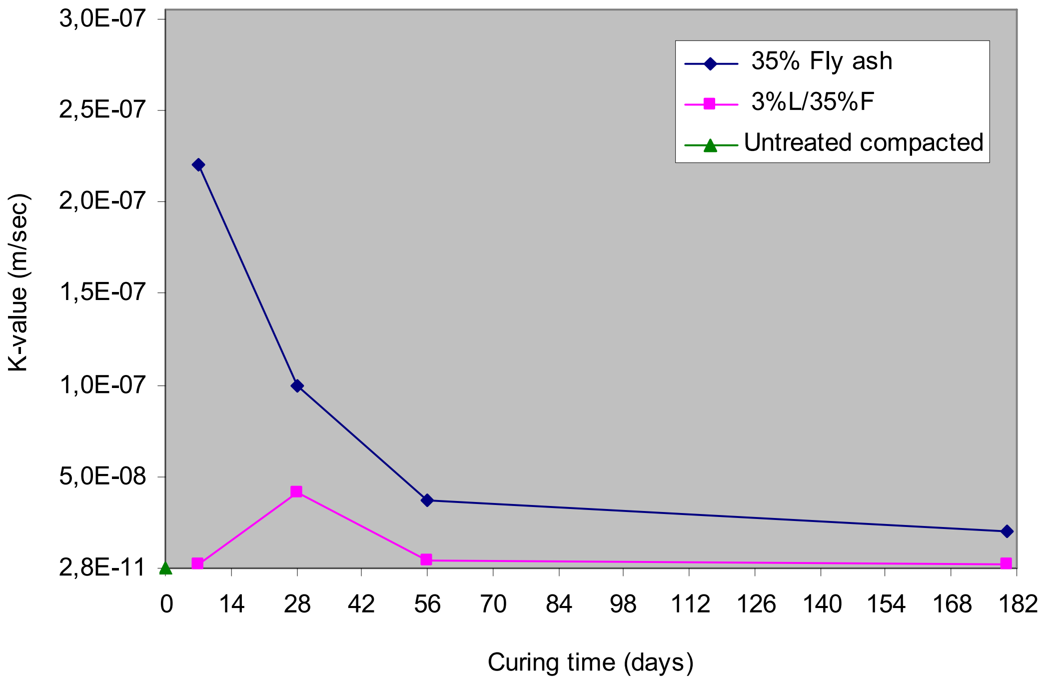


Fig. (6.1, c) Effect of curing time on the hydraulic conductivity (K-value) of fly ash- and lime/fly ash-stabilized weathered soil



6.3 Conclusions

* The compaction process without chemical additives contributed to a reduction of the K-value of the three tested soil compared to the K-value of the natural soils. The K-value of organic silt was strongly affected and reduced by the compaction process compared to both the tertiary clay and the weathered soil.

* In the case of both fly ash- and lime/fly ash-stabilization process, the fly ash- and lime/fly ash-addition resulted in an increment of the hydraulic conductivity for the three tested soils in comparison to the untreated compacted soils.

* In the case of both tertiary clay and weathered soil, the addition of lime to fly ash led to a decrement of the K-values compared to the K-values of soil-fly ash mixtures at 7 days and at long-term curing (28, 56, and 180 days). Conversely, in the case of organic silt, the addition of fly ash alone resulted in a reduction of the K-values in comparison to the K-values of lime/fly ash-soil mixtures.

* With an increase in the curing time from 7 to 28 days, the hydraulic conductivity strongly increased, where fly ash- and lime /fly ash-soil mixtures (in case of the three studied soils with exception of the weathered soil-fly ash mixtures) have the maximum K-values and K-value gain factors at 28 days curing. With an increase in the curing time, 56 and 180 days, the hydraulic conductivity of all the mixtures reduced.

7 Results: Velocity of ultrasonic p-waves (Vp)

Chemical stabilization is used to improve the engineering properties and behavior of soils and similar materials, such as fly ash. Stabilization applications generally involve laboratory determination of the type and the required amount of stabilizing agent and verification of the quality of the resulting stabilized mixture on a field scale (Yesiller et al., 2001). There are two types of methods to evaluate these materials. The first type is destructive test methods, such as unconfined compression strength-, CBR-tests, etc, which are commonly used to determine the geotechnical properties and the mechanical behavior of these materials. The second type is non-destructive test methods, such as ultrasonic testing (in the laboratory). This method (ultrasonic testing) has not been used extensively to date for chemical stabilization applications (ASTM, 1992) & (AASHTO, 1997). Non-destructive test methods, such as ultrasonic testing, can provide a fast and simple alternative approach for analyzing stabilized mixtures (Yesiller et al., 2001). On one hand, steel and concrete are commonly evaluated using ultrasonic testing in civil engineering applications, where established procedures and standards are available for ultrasonic evaluation of these materials (Mclntire, 1991). On the other hand, use of the ultrasonic testing to evaluate the stabilized soils and the similar materials is relatively limited. Ultrasonic velocity tests were conducted to evaluate the mixtures with lime, fly ash and lime/fly ash and to determine the variation and the correlation of the velocity with the type of stabilizing agent, unconfined compression strength, elasticity modulus, California bearing ratio, and tensile strength, also to make a correlation between the variation of the velocity and the variation of both qu- and E_{secant}-values with the curing time. The test procedures and the preparation of the specimens were achieved according to the procedures in chapter 2.4.5 and 2.5, respectively.

7.1 Vp of natural and untreated compacted soils

P-wave velocities (Vp-values) of the three natural and untreated compacted soils (at optimum water content) are given in Table 2.7 and Appendixes 28, 29, & 30. Ultrasonic Vp-velocities were measured for untreated compacted specimens of tertiary clay, organic silt, and weathered soil (before the performance of unconfined compressive strength tests). All the compacted specimens were prepared using a standard proctor test. Vp-value of natural tertiary clay, organic silt, and weathered soil is 643, 424, and 700 m/sec, respectively. Vp-values of untreated compacted soil specimens (without chemical additives) of tertiary clay, organic silt and weathered soil (compacted at optimum water content) are 667, 465, and 721 m/sec, respectively (see chapter 2, Table 2.7, and Appendixes 28, 29, & 30).

7.2 Vp of treated stabilized soils

P-wave velocities (Vp-values) of soil-lime, soil-fly ash, and soil-lime/fly ash mixtures (for the three tested soils) prepared at optimum water content are given in Tables 7.1, 7.2, and 7.3. The Vp-values of specimens for the three tested soils were measured (after 7 days curing) before the performance of both the unconfined compressive and the tensile strength tests.

Soil-lime mixtures of the three tested soils were prepared at the optimum lime content, 2% and 4% above the optimum lime content (compacted at optimum water content) and cured for 7 days (see chapter 2.5.1). For each mixture, average velocity of the specimens was measured. The values of Vp are illustrated in Tables 7.1, 7.2, & 7.3 and Figures 7.1 a & 7.2.

Soil-fly ash mixtures were prepared at 8, 12, 16, 20, and 25% fly ash for both the tertiary clay and the organic silt and at 8, 12, 16, 20, 25, and 35% fly ash for the weathered soil. All the mixtures were compacted at optimum water content (two hours delay after the mixing) and cured for 7 days (see chapter 2.5.2).

Table (7.1) P-wave velocity (Vp_s) and Vp_s/Vp_u ratio (Vp-gain factor) of treated stabilized tertiary clay with several blending.

Admixture mixing		**Curing time**	**Vp_s**	**Vp_s/Vp_u Ratio (Vp-gain factor)**
L (%)	FA (%)	Days	m/sec	-
4.5*	**0**	**7**	**1418**	**2.13**
6.5	**0**	**7**	**1569**	**2.35**
8.5	**0**	**7**	**1600**	**2.40**
0	**8**	**7**	**1039**	**1.56**
0	**12**	**7**	**1215**	**1.82**
0	**16***	**7**	**1309**	**1.96**
		28	**1513**	**2.27**
		56	**1536**	**2.30**
		180	**1600**	**2.39**
0	**20**	**7**	**1537**	**2.30**
0	**25**	**7**	**1485**	**2.23**
2.5*	**8***	**7**	**1314**	**1.97**
1.5	**16**	**7**	**1491**	**2.23**
2.5	**16**	**7**	**1496**	**2.24**
		28	**1655**	**2.48**
		56	**1786**	**2.68**
		180	**1875**	**2.81**
4.5	**16**	**7**	**1435**	**2.15**
6.5	**16**	**7**	**1371**	**2.10**

Notes:

Vp_s = P-wave velocity of stabilized soil (m/sec)

Vp_u = P-waves velocity of untreated compacted soil (m/sec)

Vp_s/Vp_u = Velocity gain factor (Vp-gain factor)

* Optimum content according to pH-test

Table (7.2) P-wave velocity (Vp_s) and Vp_s/Vp_u ratio (Vp-gain factor) of treated stabilized organic silt with several blending.

Admixture mixing		**Curing time**	**Vp_s**	**Vp_s/Vp_u Ratio (Vp-gain factor)**
L (%)	FA (%)	Days	m/sec	-
3*	0	7	852	1.83
5	0	7	961	2.10
7	0	7	917	1.97
0	8	7	781	1.68
0	12	7	876	1.88
0	16	7	953	2.05
0	20*	7	1011	2.18
		28	1228	2.64
		56	1297	2.79
		180	1420	3.10
0	25	7	1098	2.36
2*	12*	7	929	2.00
2	20	7	991	2.13
		28	1178	2.53
		56	1342	2.89
		180	1434	3.10
3	20	7	1013	2.18
5	20	7	949	2.04

Table (7.3) P-wave velocity (Vp_s) and Vp_s/Vp_u ratio (Vp-gain factor) of treated stabilized weathered soil with several blending.

Admixture mixing		**Curing time**	**Vp_s**	**Vp_s/Vp_u Ratio (Vp-gain factor)**
L (%)	FA (%)	Days	m/sec	-
5*	0	7	882	1.22
7	0	7	1034	1.43
9	0	7	1000	1.39
0	8	7	979	1.36
0	12	7	1018	1.41
0	16	7	1020	1.41
0	20	7	1021	1.42
0	25	7	1147	1.59
0	35*	7	1236	1.71
		28	1367	1.89
		56	1371	1.90
		180	1439	2.00
3*	20*	7	1297	1.80
3	35	7	1368	1.90
		28	1402	1.94
		56	1453	2.01
		180	1622	2.25
5	35	7	1392	1.93
7	35	7	1382	1.92
8	35	7	1370	1.90

Soil-fly ash mixtures of tertiary clay, organic silt, and weathered soil were prepared at the optimum fly ash content of 16, 20, and 35%, respectively (at optimum water content) and cured for 28, 56, and 180 days to judge the variation of Vp-value with curing time. The values of Vp, for the three different tested soils, are illustrated in Tables 7.1, 7.2, & 7.3, and in Figures 7.1 b, 7.2, & 7.4.

Soil-lime/fly ash mixtures were prepared at the optimum lime/fly ash contents, according to pH- test, of (2.5%L/8%F), (2%L/12%F), and (3%L/20%F) for tertiary clay, organic silt, and weathered soil, respectively. Other mixtures were prepared at the optimum fly ash content with different lime contents to evaluate the influence of the increase in the lime and the lime/fly ash ratio on the Vp-value of the stabilized soils and on the lime/fly ash-stabilization process (see chapter 2.5.3). All the mixtures were compacted at the optimum water content and cured for 7 days.

Other soil-lime/fly ash mixtures of tertiary clay, organic silt, and weathered soil were prepared at the optimum fly ash content with small percent of lime (at optimum water content) and cured for 7, 28, 56, and 180 days to estimate the effect of long-term curing on the Vp-value and on the lime/fly ash-stabilization process. The values of Vp are shown in Tables 7.1, 7.2, & 7.3 and in Figures 7.1 c, 7.2, 7.3, & 7.4.

7.2.1 General effect of lime-, fly ash- and lime/fly ash-stabilization process

The general effect of lime-, fly ash- and lime/fly ash-stabilization process on the Vp-values of the three tested soils is illustrated in Figures 7.1 a, b, and c, respectively. Addition of optimum lime content resulted in an increase in the Vp-values and -gain factors for the three tested soils. Lime-tertiary clay mixtures have the highest Vp-gain factors and lime-weathered soil mixtures have the lowest Vp-gain factors. The reactivity of tertiary clay with lime is strong and the Vp-gain factor increased with an increase in lime content (2 and 4% above the optimum lime content). The reactivity of both the organic silt and the weathered soil with lime is weak. Although Vp-values of lime-weathered soil mixtures are higher than the values of lime-organic silt mixtures, the lime-organic silt mixtures have higher Vp-gain factors compared to the factors of lime-weathered soil mixtures, since the Vp_u-value of weathered soil is higher than the Vp_u-value of organic silt. Vp-values and -gain factors of both the organic silt and the weathered soil increased with an increase in the lime content (2% above the optimum lime content) and decreased with the continual increase in the lime content (4% above the optimum).

Similar behavior was observed during the measurement of unconfined compressive strength-, CBR-, and tensile strength-values (see Fig. 7.1 a).

The ratio of Vp-value of lime-, fly ash-, and lime/fly ash-stabilized soil (Vp_s) to that of untreated compacted soil (Vp_u) is known as the *Vp-gain factor* (see Fig. 7.2). Vp_s/Vp_u-ratio (Vp-gain factor), due to the addition of optimum lime content, of tertiary clay, organic silt, and weathered soil is 2.13, 1.83, and 1.22%, respectively.

The addition of fly ash content contributed to an increase in the Vp_s-values for the three tested soils. Fly ash-tertiary clay mixtures have the highest Vp_s-values and fly ash-organic silt mixtures have the lowest values, at the same fly ash contents. Vp_s-values of organic silt- and weathered soil-mixtures increased with increase in the fly ash content.

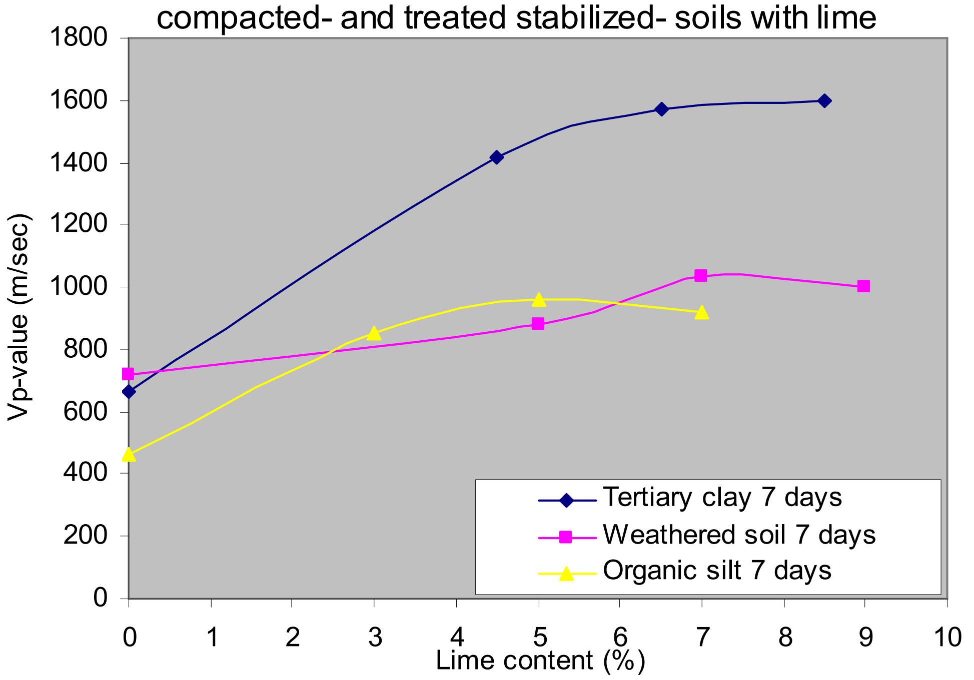


Fig. (7.1, a) P-wave velocity (Vp-value) of untreated compacted- and treated stabilized- soils with lime

The Vp_s-values of tertiary clay increased with increasing the fly ash content until 20% fly ash. Above 20% (i.e. 25%), the Vp_s-value slightly decreased. Relatively, similar behavior was obtained at the unconfined compressive strength-, the CBR-, and the tensile strength-measurements. Although, Vp_s-values of both fly ash-tertiary clay and –weathered soil mixtures are higher than the Vp_s-values of fly ash-organic silt mixtures, the gain factors of fly ash-organic silt mixtures are the highest factors. Because the organic silt has low Vp_u-value (465 m/sec) compared to both the tertiary clay and the weathered soil which have a high Vp_u-value (equal to 667 and 721 m/sec, respectively).

Fig. (7.1, b) P-wave velocity (Vp-value) of untreated compacted- and treated stabilized- soils with fly ash

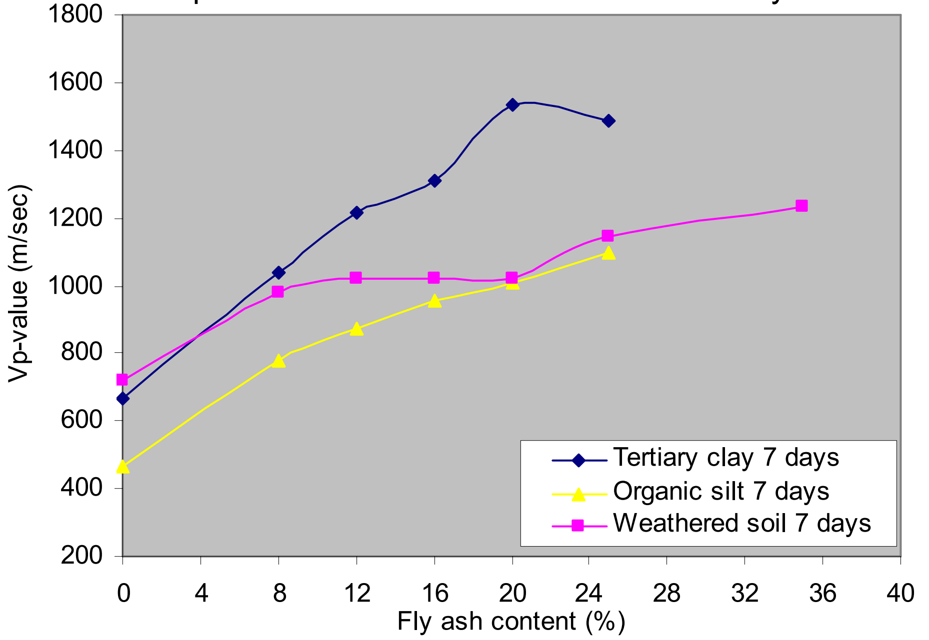
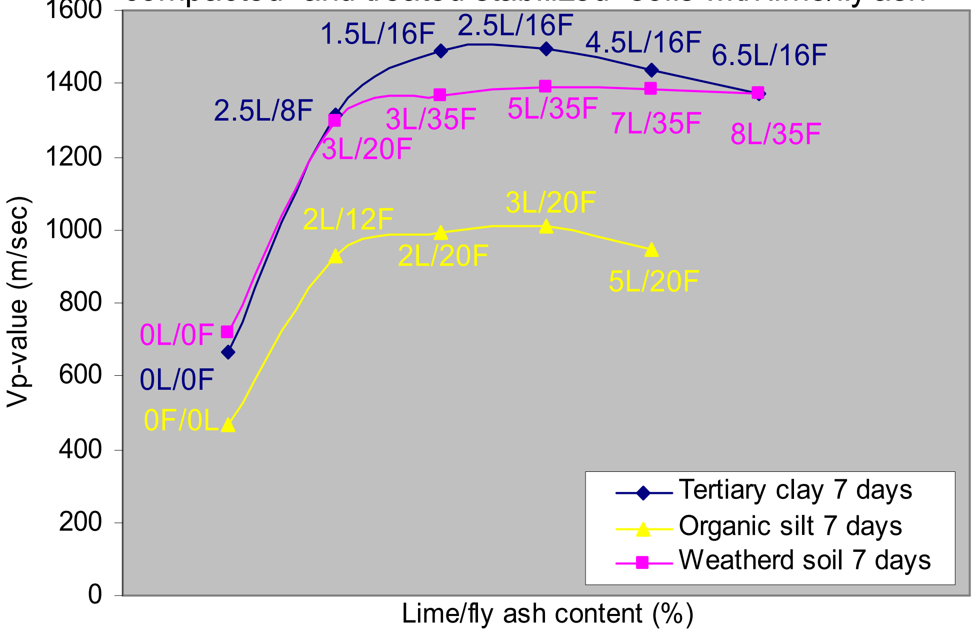


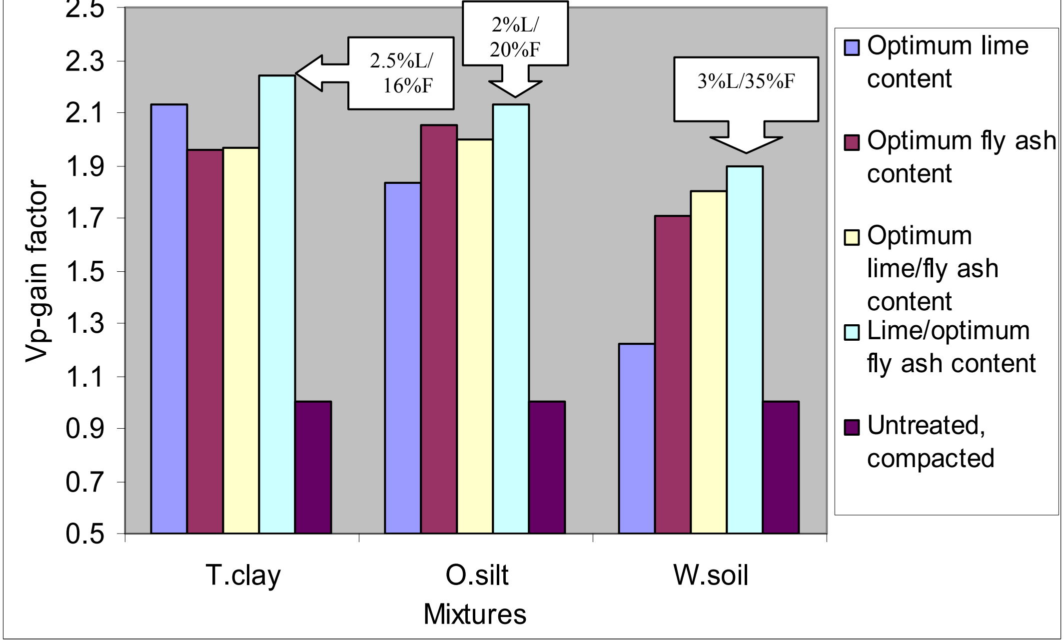
Fig. (7.1, c) P-wave velocity (Vp-value) of untreated compacted- and treated stabilized- soils with lime/fly ash

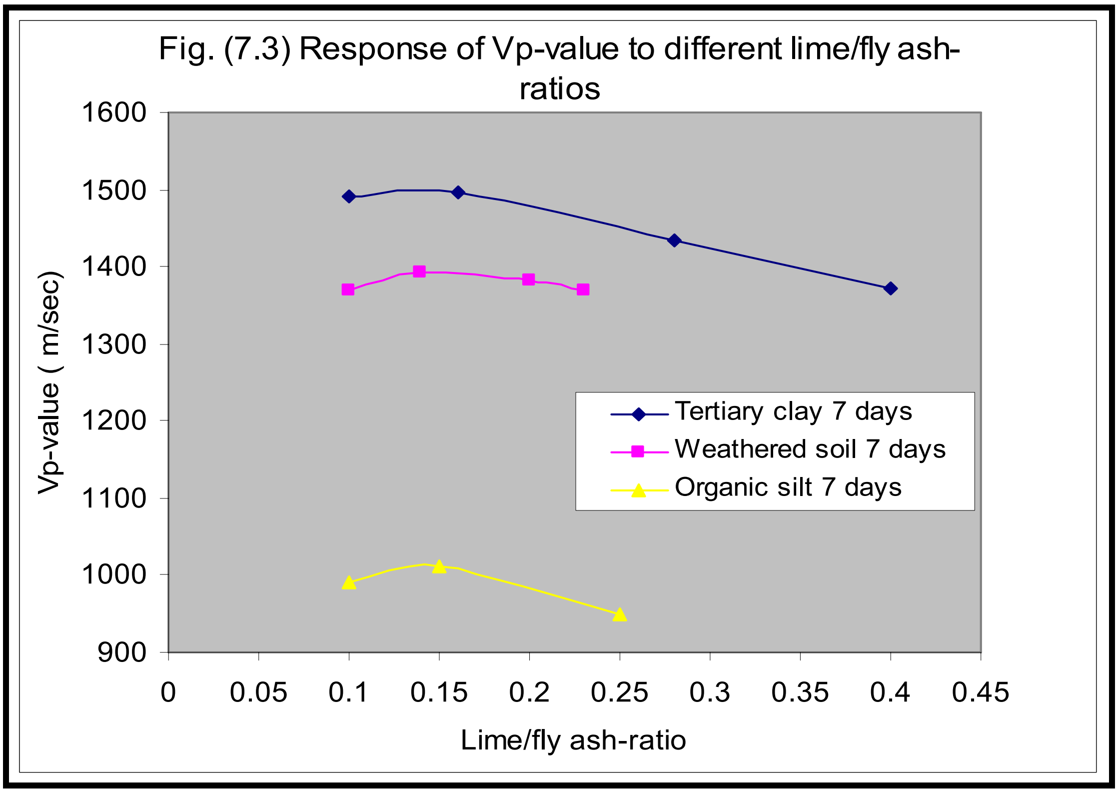


Fly ash-tertiary clay mixtures have Vp-gain factors relatively higher than that of fly ash-weathered soil mixtures, at the same fly ash contents. The Vp-gain factor (due to the addition of the optimum fly ash content) of organic silt, tertiary clay, and weathered soil is 2.18, 1.96, and 1.71, respectively (see Fig. 7.2).

The addition of lime/fly ash to the three tested soils resulted in an increment of Vp-values and Vp-gain factors. Lime/fly ash-stabilized tertiary clay has the Vp-values and the Vp-gain factors higher than both lime/fly ash stabilized-weathered soil and -organic silt. The reactivity of the tertiary clay with lime/fly ash is relatively stronger than the reactivity of the weathered soil and the organic silt. Lime/fly ash-stabilized weathered soil has the lowest Vp-gain factors. Although, the Vp-values of lime/fly ash-organic silt mixtures are lower than the Vp-values of lime/fly ash-weathered soil mixtures, the organic silt has higher Vp-gain factors since Vp_u of the organic silt is lower than Vp_u of the weathered soil. The Vp-gain factor (due to the addition of the optimum lime/fly ash content) of tertiary clay, organic silt and the weathered soil is 1.97, 2.00, and 1.80, respectively. The Vp-gain factor (due to the addition of the optimum fly ash with small percentage of lime) of tertiary clay (2.5%L/16%F), organic silt (2%L/20%F), and weathered soil (3%L/35%F) is 2.24, 2.13, and 1.90, respectively (see Fig. 7.2).

Fig. (7.2) Vp-gain factors of untreated compacted- and treated stabilized-soils





The Vp-values increased with an increment of lime/fly ash-ratio. The maximum lime/fly ash ratio of stabilized-tertiary clay, -organic silt, and -weathered soil is 0.16, 0.15, and 0.14, respectively (see Fig. 7.3). Above the ratio of 0.16 for tertiary clay and of 0.15 for organic silt (about 1 lime: 6 fly ash by weight) and above the ratio of 0.14 for weathered soil (about 1 lime: 7 fly ash by weight), the Vp-values of the stabilized-soils decreased. Similar behavior was observed during the measurement of unconfined compressive strength, CBR, and tensile strength.

In general, the variation of the velocity-values of the three tested stabilized soils with different stabilizing agent contents (lime, fly ash, and lime/fly ash) is relatively similar to the variation of unconfined compressive strength-, CBR-, and tensile strength-values (see Fig. 3.1, 4.1, 5.1, & 7.1).

7.2.2 Effect of curing time

Soil-fly ash and soil-lime/fly ash mixtures were prepared and cured for periods of 7, 28, 56, and 180 days to estimate how the long-term curing affects the Vp-values. The Vp-tests were performed on the specimens before carrying out of both unconfined compressive strength and tensile strength tests. All the specimens were prepared at the optimum water content. For each mixture, average velocity of the specimens (4-measurments) measured at each curing time was presented. The influence of curing time on the Vp-values of soil-fly ash and soil-lime/fly ash mixtures is shown in Figure 7.4. Vp-values of both the soil-fly ash and the soil-lime/fly ash mixtures, in case of the three different tested soils, increased with the long-term curing. The curing time has a stronger effect on the Vp-values of soil-lime/fly ash mixtures compared to soil-fly ash mixtures. The rate of increase in the Vp-values for soil-lime/fly ash mixtures is higher than the rate of the Vp-values for soil-fly ash mixtures. In the case of fly ash-stabilization process, the Vp-values of organic silt-optimum fly ash mixtures were strongly affected by the curing time compared to Vp-values of both the tertiary clay- and the weathered soil-optimum fly ash mixtures. The long-term curing has the weakest effect on Vp-values of fly ash-stabilized weathered soil.

In the case of the lime/fly ash-stabilization process, Vp-values of tertiary clay-lime/optimum fly ash mixtures and organic silt-lime/optimum fly ash mixtures were strongly influenced by the curing time compared to Vp-values of weathered soil-lime/optimum fly ash mixtures (see Fig. 7.4 and Tables 7.1, 7.2, & 7.3).

Figures 7.5 a, b, & c and 7.6 a, b, & c illustrate the sensitivity of the Vp-measurement to unconfined compressive strength and elasticity modulus (E_{secant}) measurements. Figures 7.5 a, b, and c show the variation of Vp- and unconfined compressive strength-values with curing time for the soil-fly ash and -lime/fly ash mixtures. Figures 7.6 a, b, and c show the variation of Vp- and the modulus (E_{secant}) values with curing time for the soil-fly ash and -lime/fly ash mixtures of the three different soils. The correlation provides that the variation of Vp-values with curing time is similar to the variation of both the unconfined compressive strength- and the modulus (E_{secant})-values. The correlation also confirms that the Vp-measurements are sensitive to changes in unconfined compressive strength- and modulus (E_{secant})-values with long-term curing. Yesiller et al. (2001) obtained relatively similar correlation between Vp and E_{secant} for a high plasticity clay stabilized with lime, cement, and fly ash and similar correlation was observed between Vp and both the qu and the E_{secant} for class F fly ash stabilized with lime and cement.

Fig. (7.4) Effect of curing time on the p-wave velocity (Vp-value) of fly ash- and lime/fly ash-stabilized soils

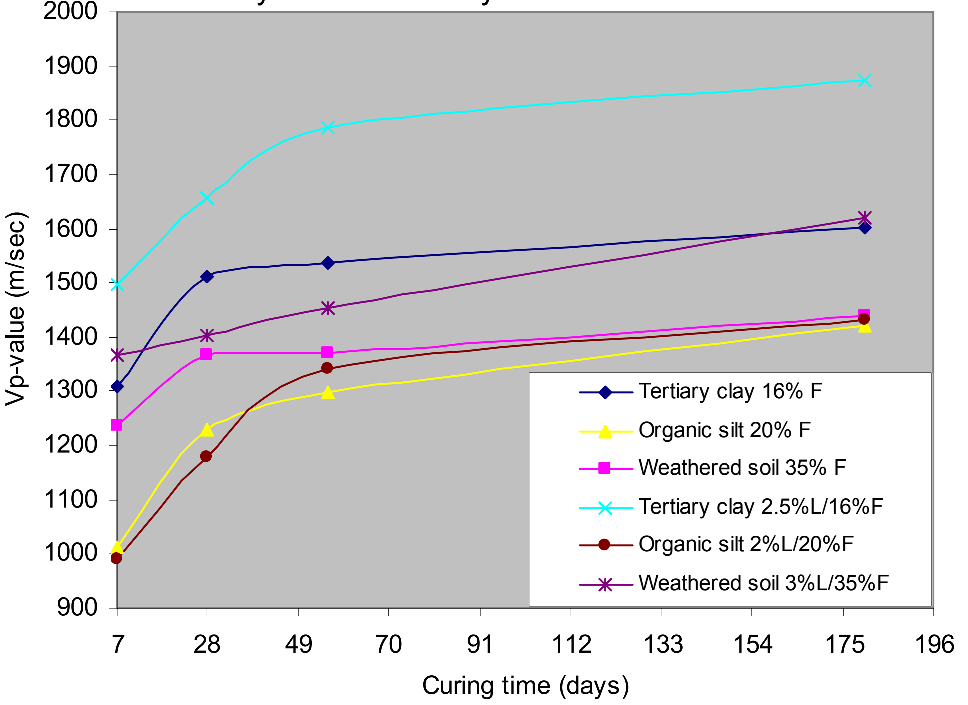


Fig. (7.5, a) P-wave velocity (Vp-value) and unconfined compressive strength (qu-value) versus curing time for fly ash- and lime/fly ash-stabilized tertiary clay

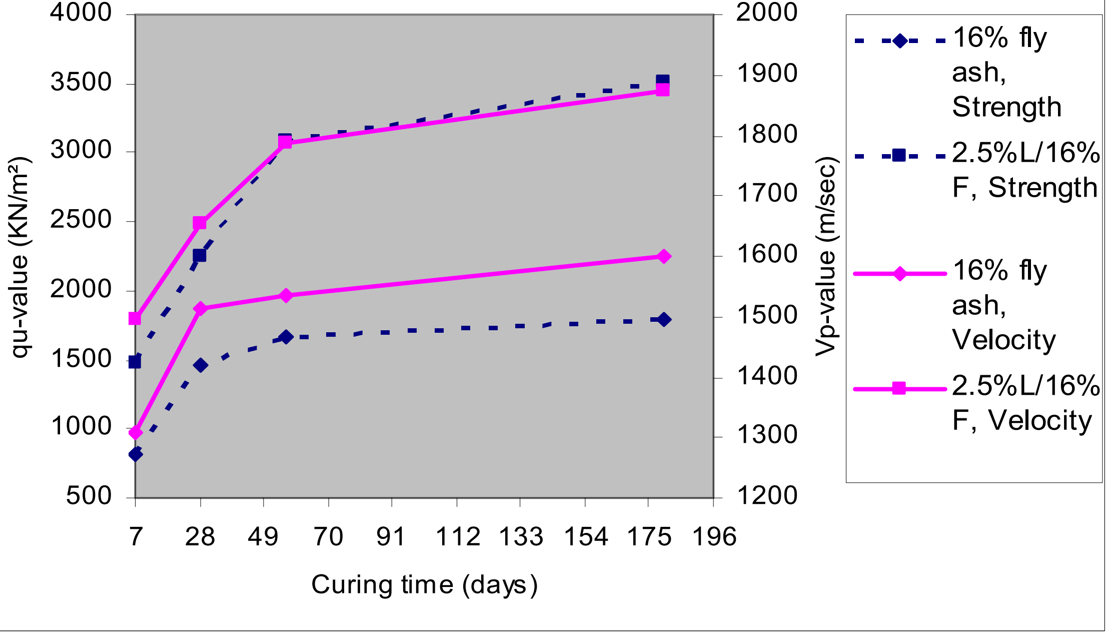


Fig. (7.5, b) P-wave velocity (Vp-value) and unconfined compressive strength (qu-value) versus curing time for fly ash- and lime/fly ash-stabilized organic silt

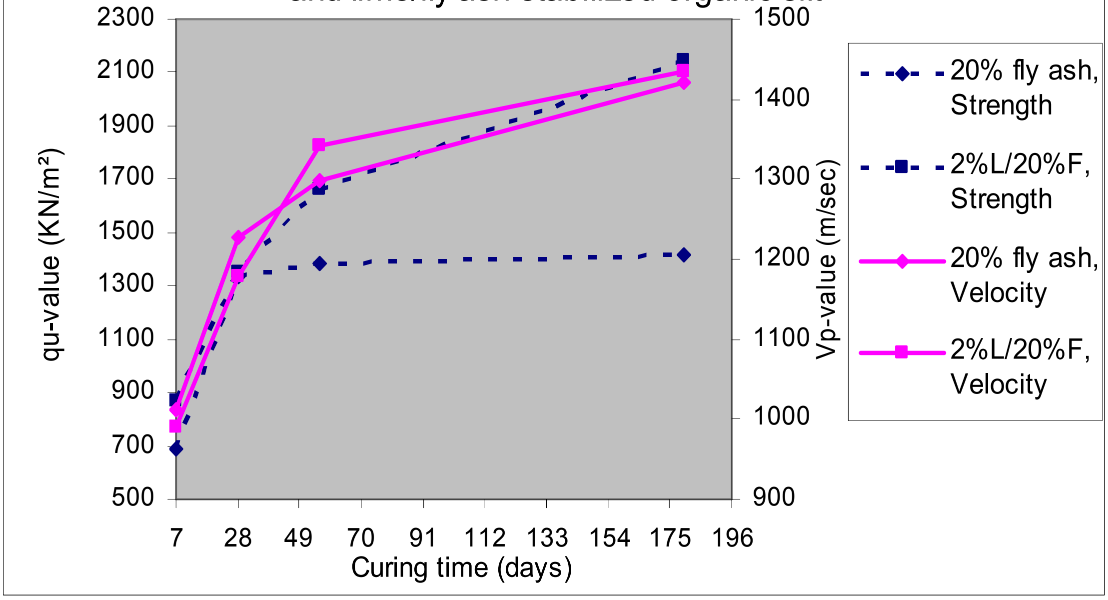


Fig. (7.5, c) P-wave velocity (Vp-value) and unconfined compressive strength (qu-value) versus curing time for fly ash- and lime/fly ash-stabilized weathered soil

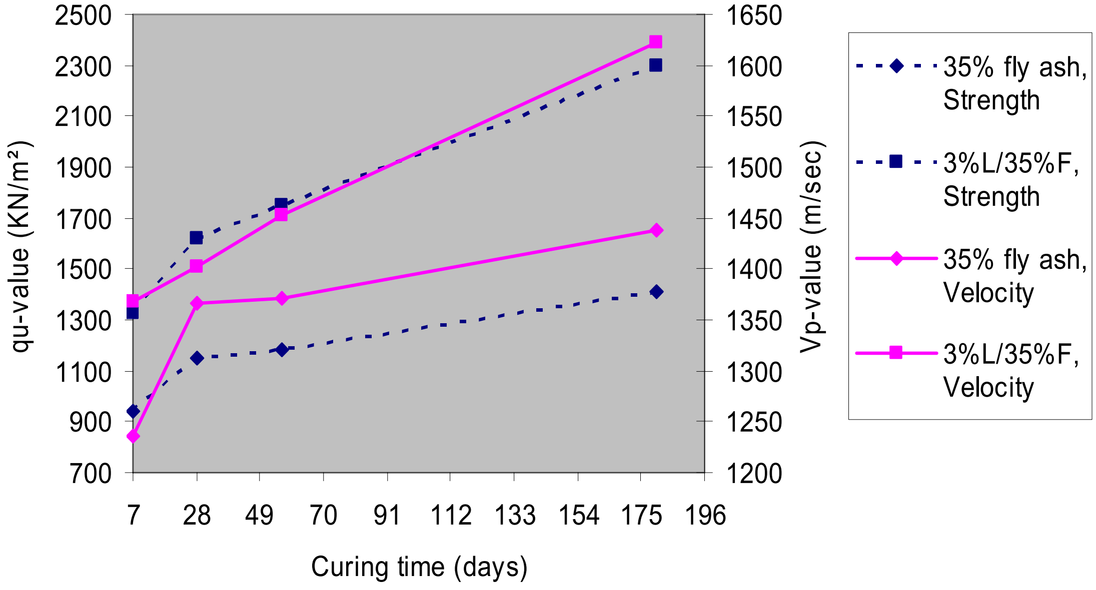


Fig. (7.6, a) P-wave velocity (Vp-value) and elasticity modulus (E_{secant}) versus curing time for fly ash- and lime/fly ash-stabilized tertiary clay

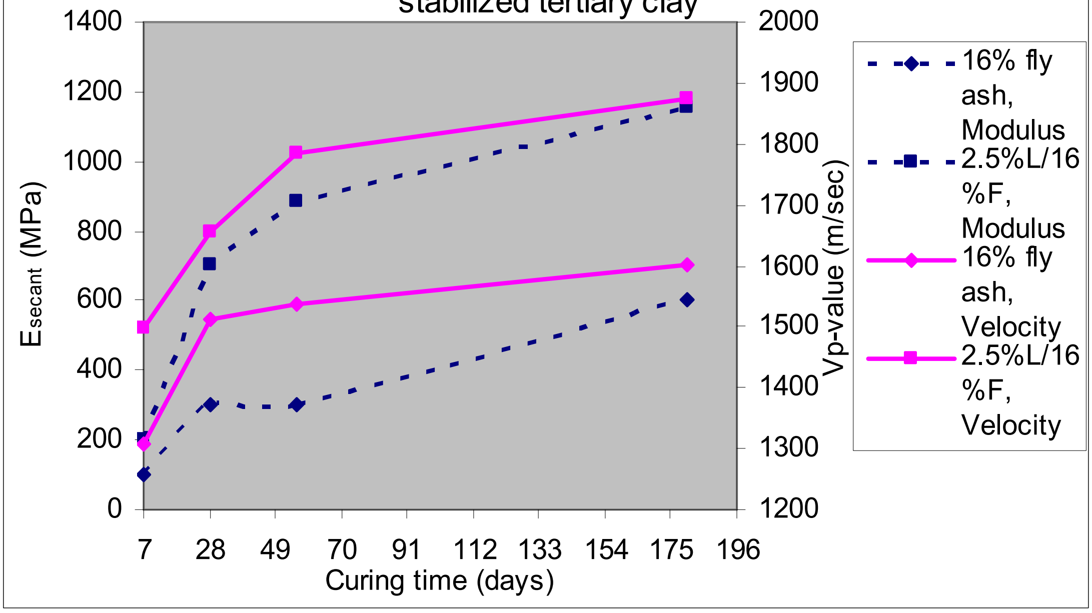


Fig. (7.6, b) P-wave velocity (Vp-value) and elasticity modulus (E_{secant}) versus curing time for fly ash- and lime/fly ash-stabilized organic silt

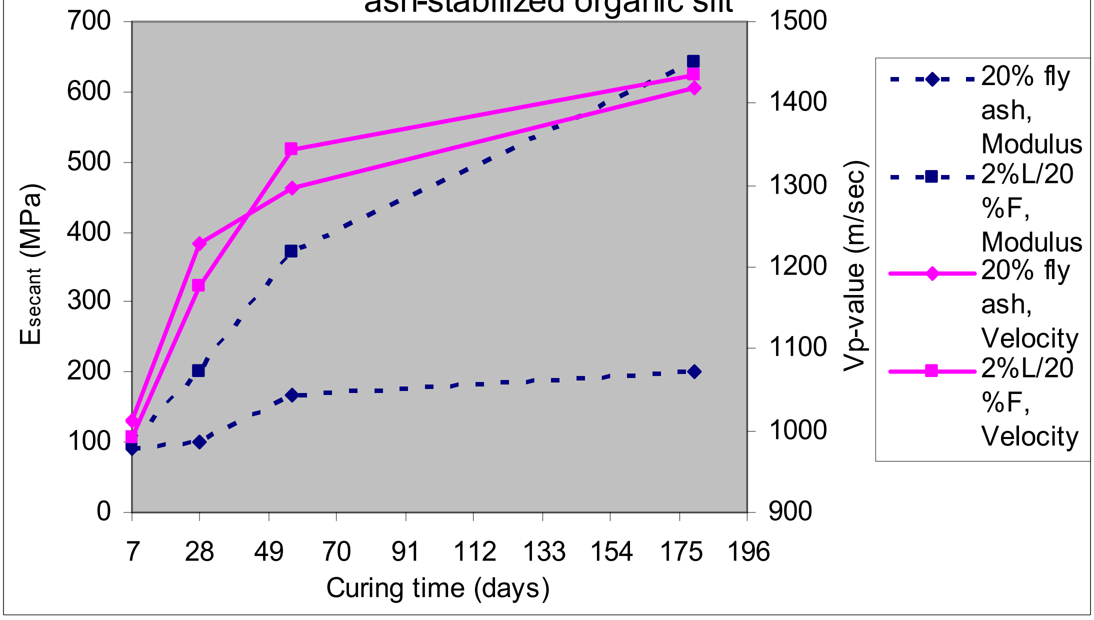
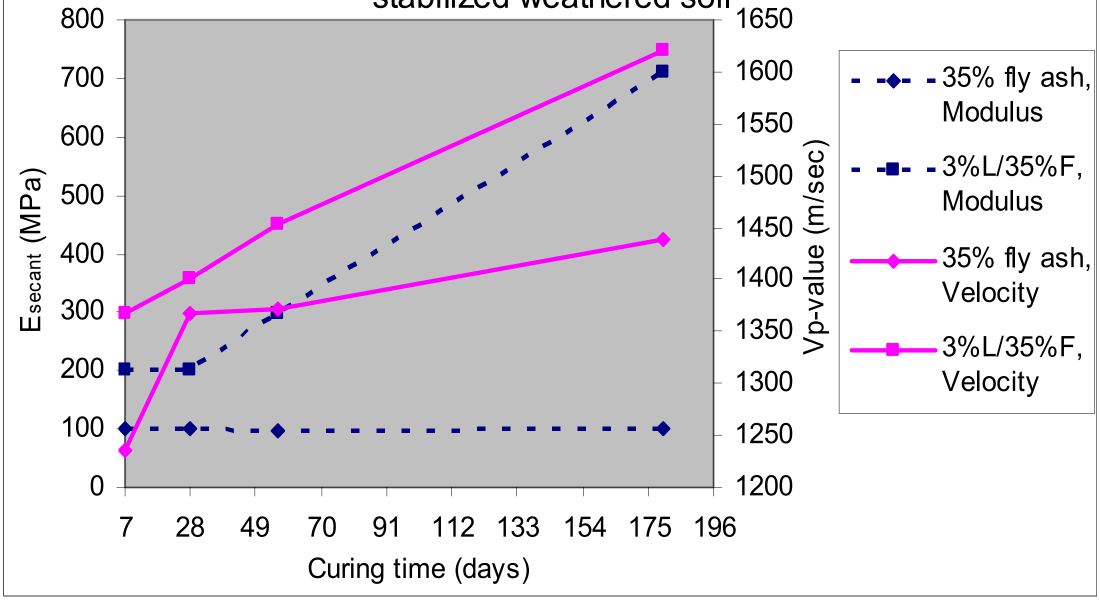


Fig. (7.6, c) P-wave velocity (Vp-value) and modulus (E_{secant}) versus curing time for fly ash- and lime/fly ash-stabilized weathered soil



7.3 Conclusions

* Vp-values increased slightly with an increment of the dry density of the untreated compacted soils whereas the amount of voids filled with air and water decreased by the compaction process. In the case of lime-, fly ash-, and lime/fly ash-stabilized soils, the Vp-values increased strongly compared to untreated compacted soils.

* In the case of lime-stabilization process, the addition of optimum lime content (according to pH-method) led to an increase in the Vp-values of the three tested soils. Optimum lime-tertiary clay mixtures have the highest Vp-values and Vp-gain factors. Conversely, optimum lime-weathered soil mixtures have the lowest Vp-values and -gain factors. The Vp-value of lime-stabilized tertiary clay increased with an increase in lime content continually (2 and 4% above the optimum lime content). Another behavior of both the organic silt and the weathered soil was observed, where the Vp-values increased at 2% above the optimum lime content and decreased slightly at 4% above the optimum. The Vp-gain factors (due to the addition of the optimum lime content) of tertiary clay, organic silt, and weathered soil are 2.13, 1.83, and 1.22, respectively.

* In the case of fly ash-stabilization process, the addition of fly ash contents contributed to an increase in the Vp-values and the Vp-gain factors for the three tested soils. The Vp–values and the Vp-gain factors of both the organic silt and the weathered soil increased with an

increment of the fly ash content continuously. The Vp-values of tertiary clay increased with an increment of fly ash content from 8 to 20%. Above 20% fly ash (i.e. 25%), the Vp-values decreased. The Vp-gain factors of fly ash-organic silt mixtures are slightly higher than the Vp-gain factors of fly ash-tertiary clay mixtures, at the same fly ash contents. The Vp-gain factors of fly ash-weathered soil mixtures have the lowest values compared to both fly ash-organic silt and –tertiary clay mixtures. Similar behavior was obtained with the CBR-measurement.

* In the case of lime/fly ash-stabilization process, addition of lime and fly ash together increased the Vp-values of the three tested soils strongly compared to addition of lime and fly ash separately. Lime/fly ash-tertiary clay mixtures have Vp-gain factors higher than lime/fly ash-organic silt and –weathered soil mixtures. The optimum ratio of lime to fly ash is about 1 lime: 6 fly ash (0.16 in case of the tertiary clay and 0.15 in case of the organic silt) and about 1 lime: 7 fly ash (0.14 in case of the weathered soil). Similar results were obtained at qu-, CBR-, and σt-measurement.

The Vp-values and the Vp-gain factors of both the fly ash- and the lime/fly ash-stabilized soils (for the three tested soils) increased with the long-term curing. The Vp-values of the lime/fly ash-stabilized soils were strongly affected by the long-term curing compared to the values of fly ash-stabilized soils, especially lime/fly ash stabilized-tertiary clay and –organic silt. The curing time has the weakest influence on Vp-values of fly ash- and lime/fly ash-stabilized weathered soil in comparison to the influence on Vp-value of both fly ash- and lime/fly ash-stabilized tertiary clay and organic silt. Similar behavior was observed at qu-, CBR-, and σt-measurement.

* The correlation between qu-, CBR-, and σt-measurement (on one hand) and Vp-measurement (on the other hand) for the three tested stabilized soils shows that the variation of Vp-values of the three studied soils, due to the addition of lime, fly ash, and lime/fly ash, (cured at 7 days) is relatively similar to the variation of qu-, CBR-, and σt-values (see Fig. 3.1, 4.1, 5.1, & 7.1).

* The correlation between Vp-, qu-, and E_{secant} -measurement of the three tested lime-, fly ash-, and lime/fly ash-stabilized soils with long-term curing proves that the variation of Vp-values with curing time is similar to the variation of both unconfined compressive strength (qu) and modulus (E_{secant}) values with curing time. The correlation also points to the fact that the Vp-measurement is sensitive to changes in unconfined compressive strength (qu) and modulus (E_{secant}) values with curing time.

8 Results: SEM-, XRD-, and Calorimetry-analysis

8.1 SEM-analysis (Microstructural analysis)

The changes of microstructural and microstructural development of soils due to lime-, fly ash-, and lime/fly ash-addition play significant role in the geotechnical properties and the mechanical behavior of these stabilized soils. The changes of the microstructural of the three tested soils due to lime-, fly ash-, and lime/fly ash-addition were investigated using JEOL-JSM-6300 scanning electron microscope operated at 20KV. Undisturbed cubical specimens (10 mm * 10 mm) of both the natural and the stabilized soils were prepared and dried in the oven at 40°C and subsequently subjected to vacuum. The tested specimens were glued on aluminum holders for scanning. The fractured surface of the specimens was coated with gold instead of carbon to get images with a good quality.

8.1.1 Microstructural analysis of natural soils

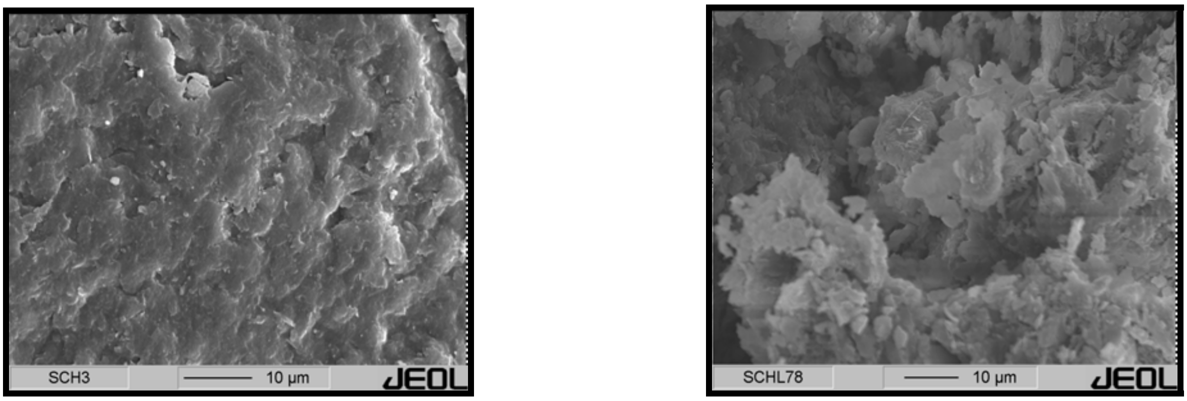
Figure 8.1 (a) shows the SEM-micrograph of natural untreated tertiary clay which indicates the sheet-like structure and flaky arrangement of the clay particles. It is a closed fabric. This confirms that the hydraulic conductivity is very poor permeable (K-value is 1.9E-11 m/sec). Figure 8.2 (a) illustrates the SEM-micrograph of natural untreated organic silt. The micrograph shows an occurrence of detrital grains of silt and fine sand fractions and little amount of clay as a matrix between the detrital grains. The specimen has silt-fine sand like structure and characterized by open fabric system and occurrence of relatively large voids distributed in the specimen. This proves that the K-value of natural organic silt (5.5E-07 m/sec) is higher than the K-value of both the natural tertiary clay and the weathered soil. Figure 8.3 (a) illustrates the micrograph of natural untreated weathered soil which shows flaky arrangements of clay particles (Kaolinite) as matrix between the detrital fine grains (the concentration and the distribution of the clay particles are not regular, some parts contain more clay particles and less detrital grains and others contain more detrital grains and less clay particles). The occurrence of clay particles as matrix leads to a reduction of the K-value (3.2E-11 m/sec) compared to K-value of organic silt.

8.1.2 Micostructural analysis of treated stabilized soils

Treated stabilized tertiary clay:

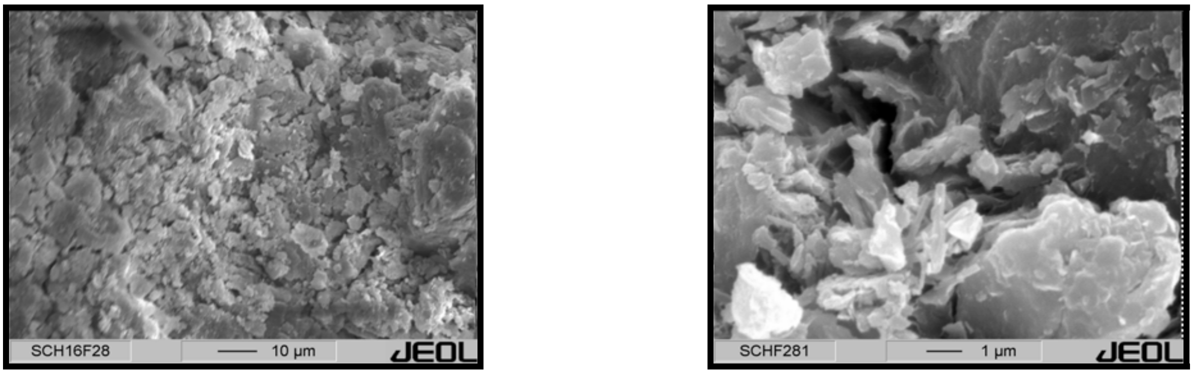
Figure 8.1 (b) illustrates a micrograph of tertiary clay soil treated and stabilized with 4.5% lime (optimum lime content) and cured for 7 days. The micrograph shows crumbs of floccules with a porous nature and cementitous compounds (calcium aluminum hydrate and calcium silicate hydrate) coating the relics of the clay particles and the flocs. The edges of the relics of the clay particles were attacked by lime and their boundaries have a ragged-form. Additionally, the reaction of lime with clay led to the formation of aggregates of various sizes and this is responsible for the increase in porosity of the soil system. Similar microfabric was observed by (Eades and Grim 1960, Croft 1964, Narasimha Rao and Rajasekaran 1992, Rajasekaran et al. 1995, and Rajasekaran and Narasimha Rao 1998). Figure 8.1 (c) illustrates a micrograph of tertiary clay treated and stabilized with 16% fly ash and cured for 28 days. The micrograph shows aggregated arrangements due to flocculation and the formation of hydration reaction products coating and cementing the soil- and the fly ash-particles together. This microfabric is silt-fine sand like structure and highly porous (open fabric). Figure 8.1 (d) shows the same specimen at a high magnification. The micrograph illustrates the flocculated arrangements and the hydration reaction products coated the relics of both soil- and fly ash-particles. It is highly porous fabric.

The pore spaces are large (> 1μm) and some individual diameters of pores is about 3 μm. This confirms the increase in the K-value at 28 days curing, where K-value is equal to 6.4E-09 m/sec. Figure 8.1 (e) shows a micrograph of tertiary clay treated and stabilized with 16% fly ash and cured for 180 days. The micrograph illustrates the formation of more new cementitous compounds after long-term curing (spiny crystals) as a result of the pozzolanic reaction coating the aggregates and the fly ash particles and filling the pore spaces (voids) between the flocs. These spiny crystals led to the development of network of reinforcement and to an increase in the strength in the long-term curing. The new cementitous compounds, in the long-term curing, were grown within the pore spaces resulting in a reduction of the radius of the pore spaces, where the pore spaces after 180 days curing are relatively smaller than the pore spaces of the same mixture after 28 days curing. This confirms the reduction of the hydraulic conductivity at the long-term curing (K-value = 1.6E-09). Figure 8.1 (f) shows a micrograph of tertiary clay stabilized with 2.5% lime and 16% fly ash together and cured for 28 days. The micrograph illustrates silt-fine sand like structure (open fabric) due to the flocculated arrangements. The hydration reaction compounds coated both the relics of clay and fly ash particles. The microstructure characterized by relatively high porous system, where the pore spaces are relatively large. This shows the relatively high permeability-value (K-value = 2.6E-09 m/sec) of the mixture at 28 days curing. Figure 8.1 (g) illustrates the microstructural development due to the long-term curing (180 days) of tertiary clay stabilized with 2.5% lime and 16% fly ash together. The micrograph shows a new formation of mineral-



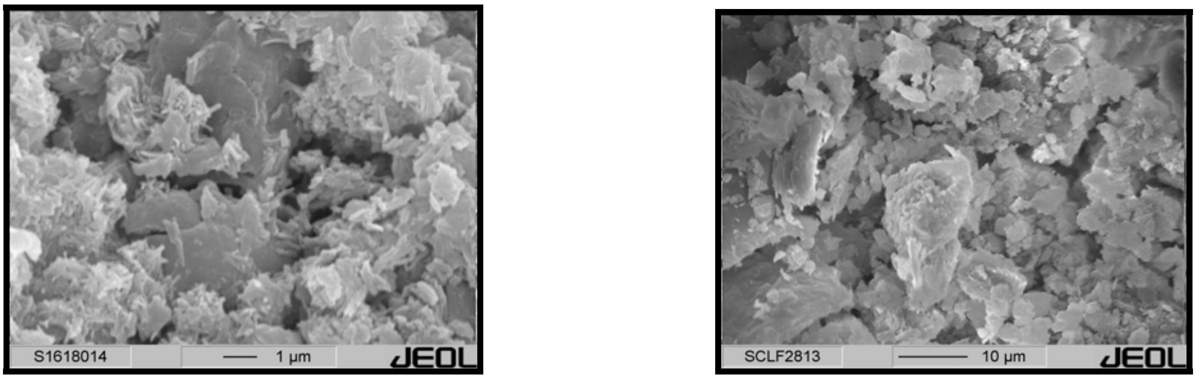
(a) Natural tertiary clay

(b) 4.5% lime, 7 days curing



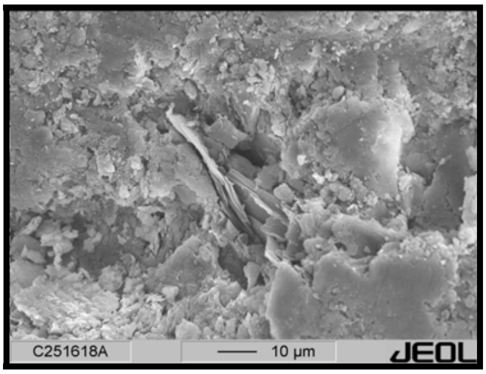
(c) 16% fly ash, 28 days curing

(d) 16% fly ash, 28 days curing



(e) 16% fly ash, 180 days curing

(f) 2.5% lime+16% fly ash, 28 days curing



(g) 2.5% lime+16% fly ash, 180 days curing

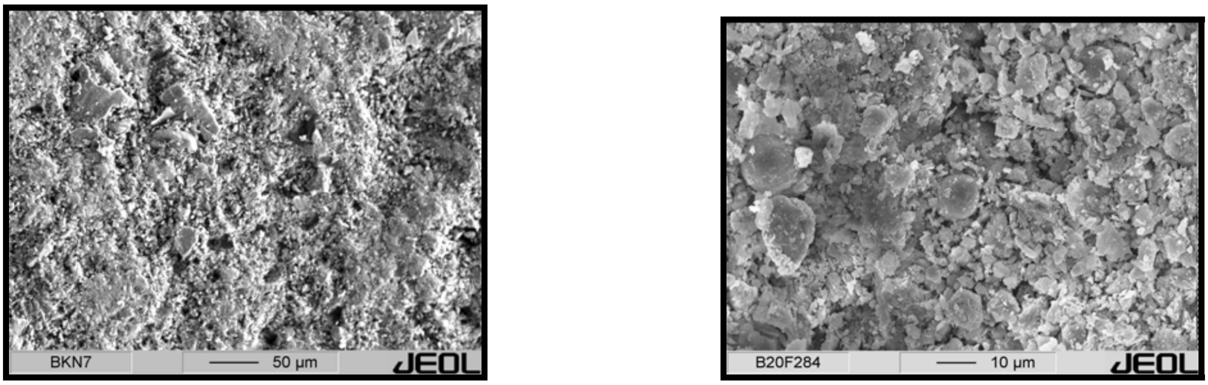
Fig. (8.1) Scanning electron micrographs illustrate the microstructural changes of tertiary clay due to lime-, fly ash-, and lime/fly ash-stabilization process.

crystal (as a product of pozzolanic reaction at long-term curing) within the pore spaces. This leads to an increase in the strength gain and a reduction of the radius of the pore spaces and subsequently reducing the K-value. K-value (5.2E-10) at 180 days curing is smaller than the value of the same mixture at 28 days curing. No evidence of ettringite has been found in the tested specimens of lime-, fly ash-, and lime/fly ash-stabilized tertiary clay.

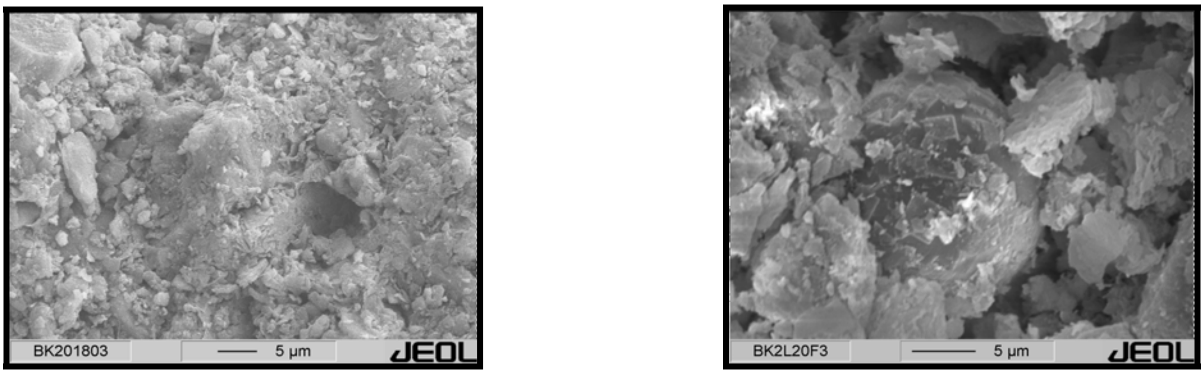
Treated stabilized organic silt:

Figure 8.2 (b) illustrates SEM-micrograph of organic silt stabilized with 20% fly ash and cured for 28 days. The micrograph shows cementitous compounds (due to pozzolanic reaction) coated and joined both the fly ash- and the soil-particles (as network). The microstructure is characterized by flocculated arrangements and has silt-fine sand like structure (porous system). This proves the relatively high value of the hydraulic conductivity at 28 days curing (K-value = 5.7E-09). Figure 8.2 (c) shows a micrograph of the same mixture cured for long-term (180 days). The micrograph illustrates a massive formation of cementitous compounds and new mineral crystals (as a result of the pozzolanic reaction) coating the surface of both the fly ash- and the soil-particles and filled the pore spaces. This contributed to an increase in the strength strongly and a reduction of the hydraulic conductivity relatively in comparison to the K-value of the same mixture for 28 days curing. Figure 8.2 (d) illustrates SEM-micrograph of organic silt stabilized with 2% lime and 20% fly ash together and cured at 28 days. The microstructure shows both the fibrous and the gel hydration reaction products. The hydration reaction products coated both the fly ash- and the soil-particles and filled the voids partially between the particles. The microstructure is highly porous due to the flocculation and the increase in the diameter of the flocs by production of the cementitous compounds surrounded these flocs. This proves an increase in the hydraulic conductivity where it has the maximum value (1.8E-07 m/sec) at 28 days curing. Figure 8.2 (e) shows the same specimen at high magnification.

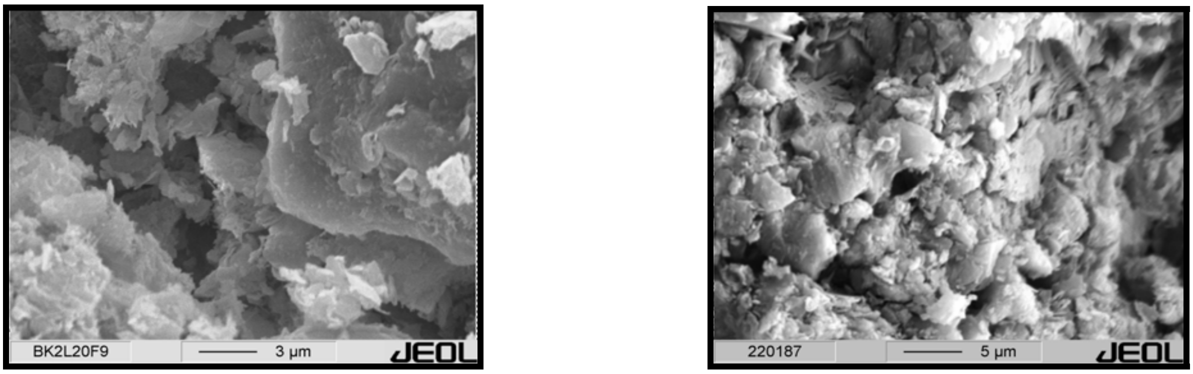
Figure 8.2 (f) illustrates SEM-micrograph of the above mentioned mixture cured at 180 days. The micrograph shows new cementitous compounds (as a result of the pozzolanic reaction after long-term curing). The cementitous compounds coated the surface of the soil particles and filled the voids partly. This led to joining the soil particles strongly, reducing the diameter of the voids, and decreasing the K-value (1.8E-08 m/sec) compared to the K-value of the same mixtures after 28 days curing. The pore spaces of lime/fly ash-stabilized organic silt are relatively larger than the pore spaces of fly ash-stabilized organic silt after 180 days curing. This confirms that the K-value of lime/fly ash-stabilized organic silt after 180 days is relatively higher than the K-value of fly ash-stabilized organic silt at the same curing time.



(a) Natural organic silt (b) 20% fly ash, 28 days curing



(c) 20% fly ash, 180 days curing (d) 2% lime+20% fly ash, 28 days curing



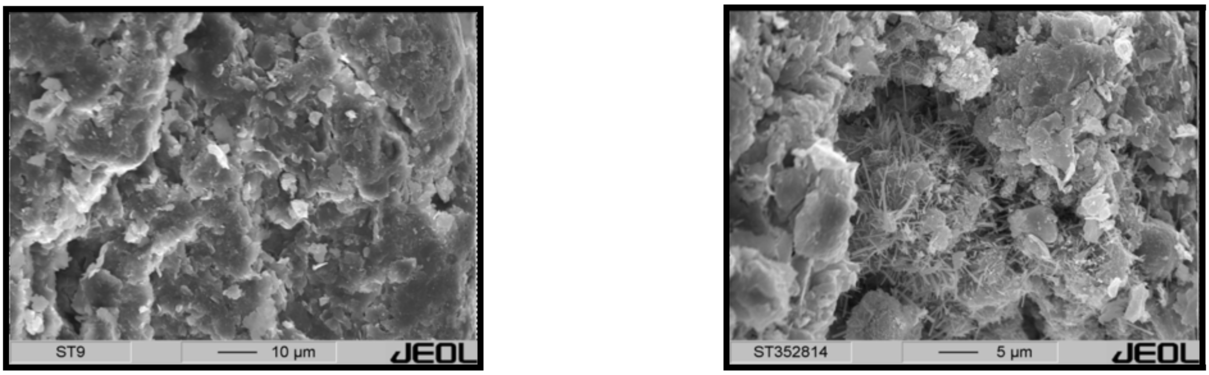
(e) 2% lime+20% fly ash, 28 days curing (f) 2% lime+20% fly ash, 180 days curing

Fig. (8.2) Scanning electron micrographs illustrate the microstructural changes of organic silt due to fly ash- and lime/fly ash-stabilization process.

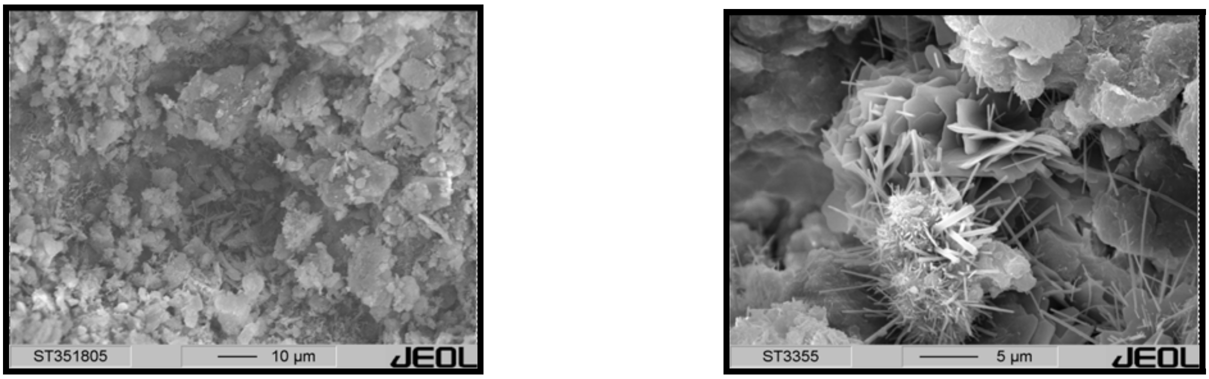
Treated stabilized weathered soil:

Figure 8.3 (b) illustrates a micrograph of weathered soil stabilized with 35% fly ash and cured for 28 days. The micrograph shows rod-like crystals “ettringite” (as hydration reaction product) growing on the relics of clay particles. The microstructure is highly porous where pore spaces are relatively large. This confirms the high value of the hydraulic conductivity

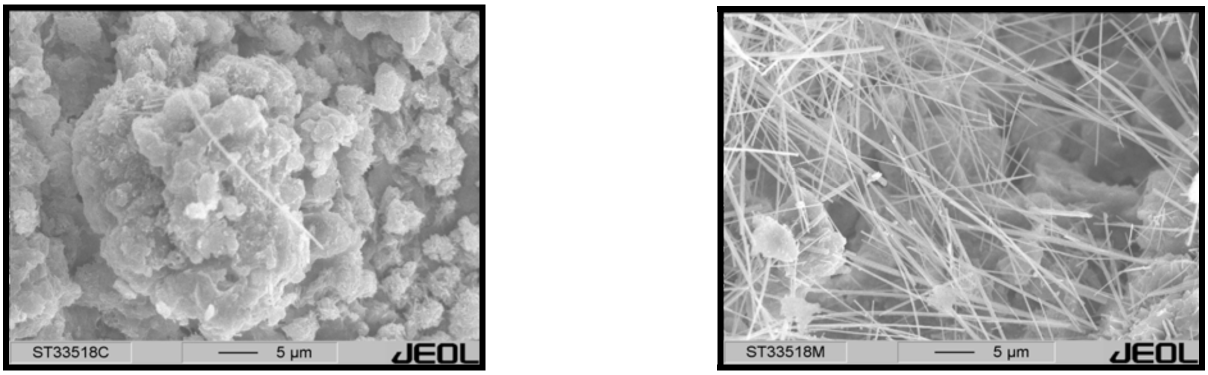
(K-value = 1.0E-07 m/sec) at 28 days curing compared to K-value of the same mixture at 180 days curing. Figure 8.3 (c) shows the microstructural development of weathered soil stabilized with 35% fly ash and cured for 180 days.



(a) Natural weathered soil (b) 35% fly ash, 28 days curing



(c) 35% fly ash, 180 days curing (d) 3% lime+35% fly ash, 28 days curing



(e) 3% lime+35% fly ash, 180 days curing (f) 3% lime+35% fly ash, 180 days curing

Fig. (8.3) Scanning electron micrographs illustrate the microstructural changes of weathered soil due to fly ash- and lime/fly ash-stabilization process.

The micrograph illustrates cementitious compounds (as pozzolanic reaction products) and massive formation of ettringite (rod-like crystals) joining together and filling the pore spaces. This led to join fly ash and soil particles together and to increase strength gain. Subsequently, this contributed to a reduction of the K-value (about 2.0E-08 m/sec) at the long-term curing. Figure 8.3 (d) illustrates micrograph of weathered soil stabilized with 3% lime and 35% fly ash together and cured for 28 days. The micrograph shows the growth of ettringite crystals at the surface of clay-relics and fly ash particles nucleated with ettringite crystals. The micrograph also illustrates cementitous compounds coating the flocs. The microstructure seems highly porous where the pore spaces are relatively large compared to the pore spaces of the same mixtures after 180 days curing. This confirms an increase in the hydraulic conductivity at the maximum value (K-value = 4.1E-08 m/sec) after 28 days curing. Figure 8.3 (e and f) shows the microstructural development of weathered soil stabilized with 3% lime and 35% fly ash together after 180 days curing. Micrograph 8.3 (e) illustrates flouclated arrangement (due to the flocculation) with pore spaces relatively smaller than the pore spaces of the same mixture at 28 days curing. The reduction of pore spaces diameter resulted from the formation of a new cementitous compounds (new formation of mineral crystals as a result of pozzolanic reaction at long-term curing) surrounded and within the pore spaces (voids) between fly ash- and soil-particles. This led to an increase in the strength gain and a reduction of the hydraulic conductivity (K-value = 2.6E-09 m/sec). Figure 8.3 (f) shows massive formation of long rods of ettringite crystals after 180 days curing. The amount and the length of ettringite crystals are large in comparison to the fly ash-weathered soil mixture after 180 days curing. This shows that the reaction of lime and fly ash together with the soil is stronger than the reaction of fly ash alone and the addition of lime to sulfate-bearing soil leads to an increase in the ettringite-formation.

8.2 XRD-analysis (X-rays powder diffraction analysis)

The mineralogical analysis of the treated stabilized soil is very important to determine the changes in the mineralogical phases due to pozzolanic reactions. These reactions depend on the chemical and the mineralogical composition of each soil and additive. In the present study, XRD-analysis was conducted on the three tested natural soils. The results showed that the weathered soil contains gypsum (hydrated calcium sulfate) (see Table 2.6 and Appendix 3).

The presence of sulfate (due to the presence of naturally-occurring gypsum crystals) can cause problems when soil is stabilized with any calcium-based additive (e.g. lime, Portland cement, fly ash etc). Sulfate in the soil reacts with calcium and alumina from clay (in the presence of water) to form the minerals ettringite and/or thaumasite. The formation of ettringite and/or thaumasite after the compaction process can lead to significant pavement heaving and loss of strength (National Lime Association, 2004).

Sulfate leads to the alteration of soil-lime, soil-fly ash, and soil-lime/fly ash reaction. In the absence of sulfate, the reactions between calcium (in the lime or in the fly ash) and soil, in the presence of water, produce calcium silicate hydrate of varying calcium to silica ratios. In the presence of sulfate, the reactions are modified and ettringite (hydrated calcium aluminum sulfate hydroxide) and/or thaumasite (hydrated calcium silicon carbonate sulfate hydroxide) are formed (Braga Reis, 1981). Hunter (1988) described the sequence of the reaction as follows:

$$6Ca + 2AL\,(OH)^-_4 + (OH)^- + 3(SO_4)^{2-} + 26H_2O \longrightarrow \underset{\text{“ettringite”}}{Ca_6\,[(AL\,(OH)_6\,)]_2\,.\,(SO_4)_3\,.\,26H_2O}$$

Problems of sulfate-presence and ettringite-formation according to previous studies:

Hunter (1988) and Mitchell (1986) observed that lime-treated sulfate-bearing clays swelled and disintegrated after a few years when used for road construction. Abdi and Wild (1993, part I) observed unrestrained expansion and swelling pressure for gypsum containing lime-stabilized clays. This resulted in the possible swelling mechanisms associated with ettringite formation. Wild et al. (1993, part II) reported that the period of volume instability and swelling coincides with the period of gypsum consumption and ettringite formation. However, swelling is not caused by growth of crystalline ettringite but is the result of water adsorption. Sridharan et al. (1995) showed that the presence of sulfate led to an increase in the compressibility of lime-treated black cotton soil after curing for long periods. Sivapullaiah, et al. (2000) studied the behavior of lime-treated montmorillonite black soil in the presence of different sulfate contents after long-term curing (>365 days). They reported that the presence of sulfate in soils considerably reduces the shear strength of lime-treated black cotton soil for the long time. The reduction in shear strength reflects a reduction in the effective cohesion intercept. This resulted in the prevention of cementation of particles by sulfate and the formation of ettringite.

In the present study, SEM-analysis conducted on the three tested stabilized soils illustrated that both fly ash- and lime/fly ash-stabilized weathered soil (after 28 and 180 days curing) contain needle or rod like crystals. These crystals are like ettringite crystals. X-ray powder diffraction analysis were conducted on fly ash- and lime/fly ash-stabilized weathered soil (after 28 days curing) to confirm the occurrence of ettringite crystals (see Appendixes 4 & 5). The analysis showed the occurrence of ettringite crystals confirming the SEM-analysis. After 28 days moist curing no gypsum was observed in both the fly ash and the lime/fly ash-stabilized weathered soil specimens. The only new crystalline phase was ettringite where gypsum was consumed to form ettringite crystals.

In the case of both fly ash- and lime/fly ash-stabilized weathered soil cured for 28 and 180 days (moist-curing without water soaking), the presence of sulfate led to the formation of ettringite and the reduction of the strength gain development especially with long-term curing (see chapter 3.4.2). No critical expansion of stabilized weathered soil after 28 and 180 days moist-curing was observed. The expansion of fly ash- and lime/fly ash-stabilized weathered soil (due to ettringite-formation) was less than 1% (without water-soaking).

8.3 Calorimetry-analysis

The heat of the hydration reaction is an important factor to characterize cements, mortars and other materials in building industries (Poellmann et al., 1991). The hydration reaction of cement, lime (CaO), and mortar etc (without mixing with soil) is relatively fast and spans hours, so that the use of the calorimeter is significant. Conversely, the hydration or pozzolanic reaction of the mixtures of soil (clay or silt) with chemical additives [cement, lime CaO or Ca $(OH)_2$, fly ash, and lime/fly ash together etc.] is slow and spans days, months, and sometime more than one year. The pozzolanic reactions between lime and lime/fly ash stabilizers and soils generally occur slower than reactions between soils and other stabilizing agents such as cement (Yesiller et al., 2001). In the present study, during carrying out of consistency tests of different mixtures of the tested soils with lime, fly ash, and lime/fly ash, it was observed that the mixtures of organic silt with optimum lime content and with lime/fly ash contents (after the mixing with water and curing for 1 day) lost their flow-ability and set quickly after 1 day curing. However, this was not observed for the mixtures of both tertiary clay and weathered soil with optimum lime content and lime/fly ash contents. This behavior of optimum lime- and lime/fly ash-organic silt mixtures was unexpected. In order to interpret this behavior and to evaluate the heat of the hydration-reactions of the soil-chemical additive mixtures, heat-flow calorimetry was conducted on the mixtures of the three tested soils with optimum lime contents and lime/fly ash contents. Heat-flow calorimeter (developed by Poellmann et al., 1991) was used. The calorimeter runs either in an isothermal mode or in a slow scanning mode over a temperature range up to 110°C. When exothermic or endothermic reactions take place, the temperature difference between the measuring system and the surrounding occurs. Therefore, a heat current flows as long as isothermal conditions are installed again and the reaction heat (ΔQ) can be calculated by the integral of temperature differences (ΔT) as a function of time (t) (Poellmann et al., 1991).

$$\Delta Q = {}^{t1}\!\int_{t2} k\ (T)\ \Delta T\ (t)\ dt \qquad \text{Where } k\ (T) = \text{Calibration factor (constant)}$$

Figure 8.4 illustrates the calorimeter with its 4-cells and peripheral tools. The calorimeter contains 4-cells in alluminum block with 4-symmetric drills (quadruplet). The tested specimens (lime- and lime/fly ash-soil mixtures) are put in three measuring-cells (in copper-pots with covers) "M". The fourth cell used as reference "R". The reference-cell is used to compensate the difference between the surrounding temperature and the measuring-cells. The water is poured on the mixtures [water (W)/solid (S) =1] by injection using a syringe through a pore of the copper-pot cover. The measured-data (temperature differences) can be caught on digital-voltmeter (PREMA 6110) and saved on the personal computer. The data can be represented and evaluated using graphical and statistical-software.

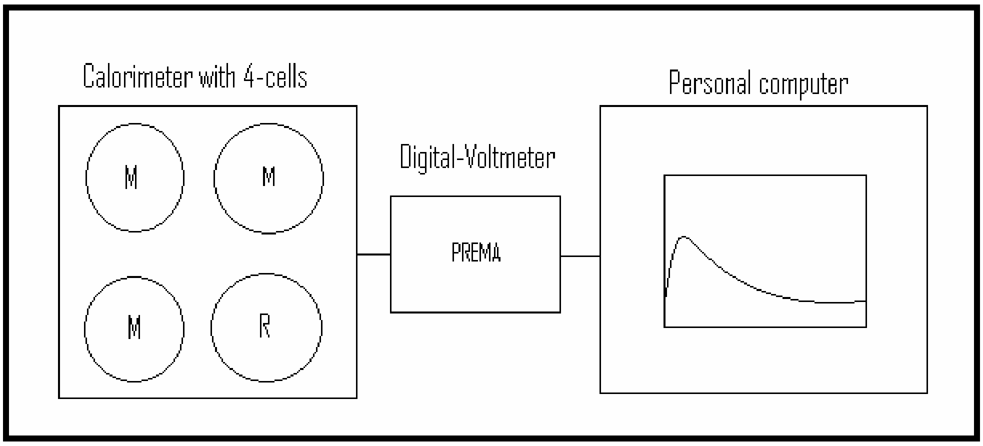


Fig. (8.4) Calorimeter with 4-cells and its peripheral tools (modified after Poellmann et al., 1991).

The calorimeter analysis illustrated that no hydration-reactions were recorded for the mixtures of optimum lime- and lime/fly ash-teritary clay and -weathered soil until 3 days. However, there are fast hydration reactions of optimum lime- and lime/fly ash-organic silt mixtures (see Fig. 8.5). The mean peak height of the optimum lime-organic silt mixture was at 3.5 hours and the mean peak heights of two lime/fly ash-organic silt mixtures were at 7.5 hours. The entire heat flow of 3% lime-, 2% lime/20% fly ash-, and 3% lime/20% fly ash-organic silt mixtures is 59.28, 99.02, and 85.85 J/g, respectively. These fast-hydration reactions prove the loss of flow-ability and the quik-set of the optimum lime- and lime/fly ash-organic silt mixtures after mixing with water and curing for 1 day. Natural organic silt contains a high value of CaO-content (18.7%) compared to low values of CaO-content of both natural tertiary clay (2.41%) and weathered soil (1.34%). The high value of CaO-content of natural organic silt led to an acceleration of the hydration reaction of the chemical additives. Additionally, the prescence of calcite mineral (see Appendix 2) in the natural orgaic silt also contributed to an acceleration of the hydration reaction. Shi et al. (2004) reported that the crushed limestone dust (waste material) can be used to produce self-consolidation concrete

(SCC) with properties similar to those of SCC containning coal fly ash. SCC mixtures containing limestone dust loses its flow-ability and sets faster than the mixtures containing fly ash. This is due to an acceleration of the hydration of portland cement by the limestone powder.

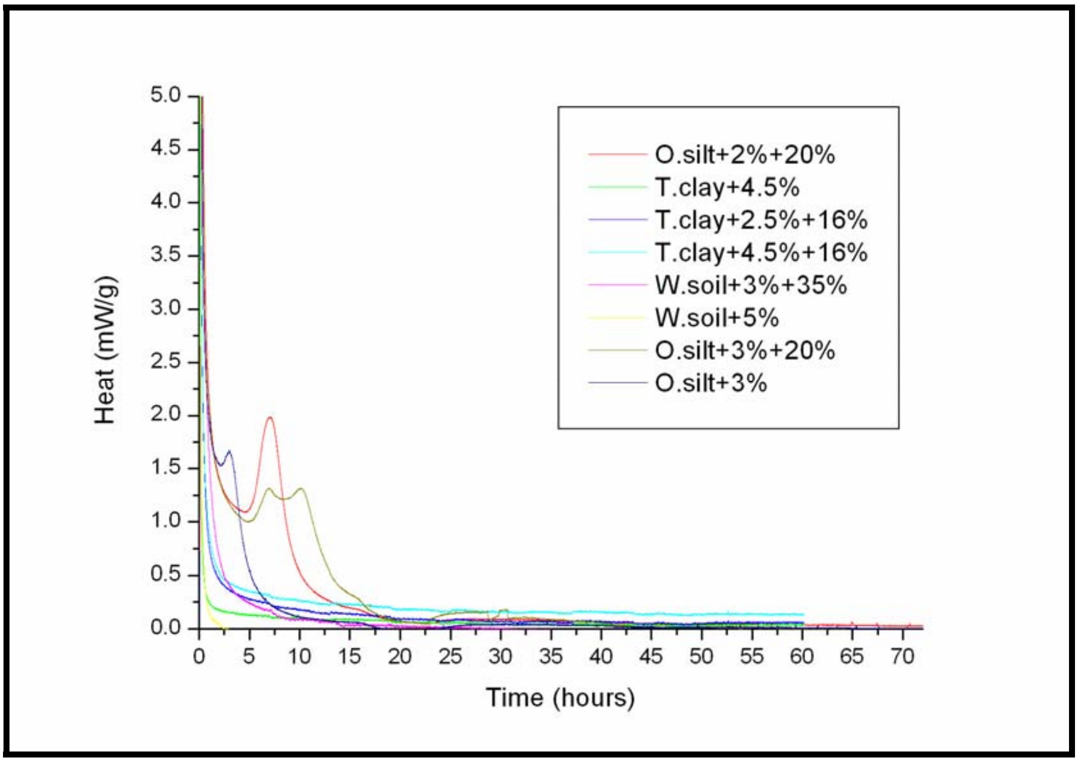


Fig.(8.5) Calorimetric curves of the three studied soils mixed with optimum lime and lime/fly ash contents (W/S = 1).

8.4 Conclusions

Scanning electron microscope studies indicated that the microstructures of the tested soils was changed due to lime-, fly ash- and lime/fly ash-stabilization process and developed through the long-term curing. The stabilization process led to the formation of a silt-fine sand like structure (open fabric) characterized by a highly porous system. Additionally, SEM-micrograph of fly ash- and lime/fly ash-stabilized weathered soil showed rod-like crystals (ettringite).

SEM-micrographs of natural and treated stabilized soils indicated that the formation of new cementitous compounds and mineral crystals as a pozzolanic reaction product through the long-term curing contributing to increasing the strength gain. These cementitous compounds join the soil particles together and increase the strength gain. XRD-analysis confirmed that the presence of sulfate in the weathered soil led to ettringite-formation. After

28 days moist-curing of both the fly ash- and the lime/fly ash-stabilized weathered soil, the only new crystalline phase was ettringite where gypsum was consumed to form the ettringite crystals. There is no critical volume-changes (expansion) of both fly ash- and lime/fly ash stabilized weathered soil (moist-curing at 28 and 180 days) due to ettringite-formation (the increase of volume < 1%). The calorimetry-analysis illustrated that the high value of CaO-content and the presence of calcite mineral in the natural organic silt led to an acceleration of the hydration reaction of the optimum lime- and lime/fly ash-organic silt mixtures. The maximum value of the heat of the hydration reaction (main peak height) for optimum lime (3%)-organic silt mixture spanned 3.5 hours and the entire heat flow was 59.28 J/g. The maximum value of the heat of the hydration reaction (main peak height) for both 3% lime/20% fly ash- and 2% lime/20% fly ash-organic silt mixtures spanned 7.5 hours and the entire heat flow was 99.02 and 85.85 J/g, respectively.

9 Discussions, final conclusions, and suggestions for the future

The present study is a geotechnical laboratory program to estimate how the use of fly ash (a by-product from a local electric power plant at Lippendorf, south of Leipzig city, Saxony, Germany), hydrated lime, and lime/fly ash could improve the geotechnical properties [including consistency limits, compaction properties, unconfined compressive strength (qu), elasticity modulus (E_{secant}), durability, California bearing ratio (CBR), indirect tensile strength (σt), and the hydraulic conductivity (K-value)] of three different soft fine-grained soils [tertiary clay, organic silt, and weathered soil] collected from Halle-city region, Saxony-Anhalt, Germany. One of the main aims of the study is to use the ultrasonic p-wave velocity testing as a non-destructive method to evaluate the improvement of the geotechnical properties of the stabilized soils and to correlate the p-wave velocity of the stabilized soils with the other geotechnical parameters (qu-, E_{secant}-, CBR-, and σt-value). In addition to this, the study is an attempt to estimate the effect of lime-, fly ash-, and lime/fly ash-stabilization process on the microstructures and on the mineralogical composition of the three studied soils using SEM- and XRD-analysis, respectively. Furthermore, this study is designed to evaluate the heat flow of the soil-chemical additive mixtures using calorimetry-analysis. The results of the present study illustrated the following findings:

9.1 Plasticity and compaction

* The addition of lime, fly ash, and lime/fly ash to the three tested soils led to a reduction of the plasticity index. This reduction occurs by decreasing the thickness of the double layer of the clay particles, as a result of cation exchange reaction, which causes an increase in the attraction force leading to flocculation of the particles (Nalbantoglu & Gucbilmez, 2001). Unlike the pozzolanic reaction, flocculation tends to modify the soil without producing new secondary minerals (Marks & Haliburton, 1972)**.** The addition of fly ash and lime/fly ash increased the percent of sand particles fraction and decreased the percent of fines (Nalbantoglue, 2001)**.** This increase of coarse grain size resulted in a reduction of the plasticity index and the swell potential, especially in the case of tertiary clay, which contains relatively large amount of clay particles (< 2µm = 47%) including kaolinite, montmorillonite (expansive clay mineral with high surface area, it reacts with the chemical additives strongly compared to the reaction of kaolinite), and halloysite.

* The addition of lime, fly ash, and lime/fly ash to the studied soils contributed to an increase in the optimum moisture content and a decrease in the maximum dry density. The moisture-

density curves of the stabilized soils have a typical flattened form. This typical flattening of the compaction curves makes it easier to achieve the required density over a wider range of possible moisture contents. This change in the shape and characteristics of the peak of the compaction curves can allow for significant savings in time, effort, and energy (Nicholson et al., 1994).

9.2 Strength, bearing capacity, and ultrasonic p-wave velocity

* The qu-, E_{secant}-, CBR-, and Vp-values increased slightly with increment of the dry density of the untreated compacted soils (due to the compaction process without chemical additives) whereas the amount of voids filled with air and water decreased. In the case of stabilized soils, the values increased strongly (compared to untreated compacted soils) due to the addition of the chemical stabilizing agents (lime, fly ash, and lime/fly ash) whereas the formed cementitious compounds (as a result of the chemical reactions between the silica and the alumina and the additives) reduced the volume of the void spaces and joined the soil particles.

9.2.1 Lime-stabilization

* In the case of lime stabilization process, the optimum lime content (according to pH-method) of tertiary clay, organic silt, and weathered soil is 4.5, 3, and 5%, respectively. Tertiary clay is strongly reactive with lime. Unconfined compressive strength, California bearing ratio, indirect tensile strength, and p-wave velocity of the lime-stabilized tertiary clay increased continuously with the increase in lime content, because it contains a high amount of the clay particles (< 2µm = 47%) including kaolinite, montmorillonite, and halloysite where montmorillonite reacts strongly and fast with additional lime. Both the organic silt and the weathered soil react weakly with lime where they contain relatively small amount of the clay particles including only kaolinite (in weathered soil) and halloysite (in organic silt) which react slowly with the additional lime in comparison to montmorillonite in tertiary clay. Weathered soil and organic silt have failed in the reactivity test. The qu-, CBR-, σt-, and Vp-values increased at the optimum lime content and 2% above the optimum compared to the values of untreated compacted specimens, but the continual increase in lime (4% above the optimum lime content) resulted in the decrease in the values. This may be due to the fact that both the organic silt and the weathered soil contain low amount of clay particles. Additionally, the weathered soil is a problematic acidic soil where its pH-value < 7 (= 2.65).

9.2.2 Fly ash-stabilization

* In the case of fly ash stabilization process, the optimum fly ash content (according to pH-method) of tertiary clay, organic silt, and weathered soil is 16, 20, and 35%, respectively. The qu-, CBR-, σt-, and Vp-values increased with increasing fly ash content in case of both the organic silt and the weathered soil. In case of the tertiary clay, the values increased with increasing fly ash content (from 8 to 20%) and decreased with a continual increase in fly ash content (above 20%). The reactivity of both the organic silt and the weathered soil with fly ash is relatively weaker than the reactivity of tertiary clay, at the same fly ash contents. CBR- and Vp-gain factors of fly ash-organic silt mixtures are higher than the gain factors of fly ash-tertiary clay mixtures, at the same fly ash contents. This is on the one hand due to the lower CBR- and Vp-value of the untreated compacted organic silt compared to the values of tertiary clay, on the other hand due to the finding that a part of fly ash particles filled the large void spaces in the organic silt. The hydration reaction (in these void spaces) was not inhibited by the organic matter (Hebit & Farrel, 2003). The CBR- and the Vp-gain factors of fly ash-weathered soil mixtures have the lowest values compared to both fly ash-organic silt and –tertiary clay mixtures.

9.2.3 Lime/fly ash-stabilization

* In the case of lime/fly ash-stabilization process, the optimum lime/fly ash content (according to pH-method) of tertiary clay, organic silt, and weathered soil is (2.5%L+8%F), (2%L+12%F), and (3%L+20%F), respectively. The addition of lime and fly ash together increased the qu-, CBR-, σt-, and Vp-values of the three studied soils strongly compared to lime and fly ash separately. Lime/fly ash-tertiary clay mixtures have qu-, CBR-, σt-, and Vp-values higher than the values of both lime/fly ash-organic silt and –weathered soil mixtures. When lime is mixed with fly ash, it has been suggested that for “a given ratio of lime to fly ash, the compressive strength of lime-fly ash mixture will increase with an increase in the amount of lime and fly ash used” (Chu et al., 1955). There has been some discussion that there may be an upper limit at which no further strength gain should be expected. In the present study, the strength gain increased for all the three tested soils with increase of both the lime and the fly ash contents, but this increase has upper limit at the optimum lime/fly ash ratio. Virendra & Narendra (1997) reported that the use of fly ash from Obra thermal power station led to an increase in the unconfined compressive strength- and the California bearing ratio-values of an alluvial soil stabilized with 15% of lime and fly ash in proportion of 1 : 3 by

weight (about 0.33 lime of the fly ash weight), less or more than this ratio (for example 1 lime : 4 fly ash = “0.25” or 1 lime: 2 fly ash = “0.5”), the unconfined compressive strength and the California bearing ratio decreased. In the present study, the qu-, CBR-, σt-, and the Vp-values increased with increasing lime/fly ash ratio and the maximum values of these parameters are at the optimum lime/fly ash-ratio, and above the optimum lime/fly ash-ratio, the values decreased. The optimum lime/fly ash-ratio of tertiary clay and organic silt is 0.16 and 0.15, respectively (about 1 lime: 6 fly ash by weight) and the ratio of weathered soil is 0.14 (about 1 lime: 7 fly ash by weight). This means that the ratios, in the present study, are relatively small, as Lippendorf fly ash (off-specification type, self-cementing, and close to class C fly ash) contains a very high CaO-content = 38.3% compared to the other types of fly ashes such as Obra fly ash (non-self-cementing and close to class F fly ash) which contains a very low CaO-content = 2.2%.

* In case of the three studied stabilized soils, elasticity modulus (E_{secant}) increased and failure axial strain (ε_f) decreased as a consequence of either the separate or the joined effects of lime and fly ash contents. The E_{secant} increased and the failure axial strain decreased dramatically with the addition of both the lime and the fly ash together, especially in the case of tertiary clay. The mechanical behavior of the three studied soils was developed from ductile to brittle. This development was relatively weak in case of the acidic weathered soil. The development of the mechanical behavior from ductile to brittle of the three stabilized soils was strong through the long-term curing except the stabilized weathered soil. The qu-, E_{secant}-, the failure axial strain (ε_f)-, CBR-, σt-, and Vp-value of both fly ash-and lime/fly ash-mixtures, for the three studied soils, were improved through the long-term curing (180 days). The influence of curing time was strong on the lime/fly ash-stabilization process compared to the effect on the fly ash-stabilization process, especially tertiary clay where the improvement of lime/fly ash tertiary clay mixtures with long-term curing was dramatic. This is due to the occurrence of enough pozzolanic material “clay- and fly ash-particles as a source of silica and alumina” in the mixture to react with lime in both the short- and the long-term chemical reactions and to increase in the amount of cemintitious compounds with long-term curing. The effect of long-term curing on fly ash- and lime/fly ash-stabilized weathered soil was weaker than the effect on both the fly ash- and the lime/fly ash stabilized-tertiary clay and -organic silt.

* The mechanical behavior of the stabilized soils depends on different factors such as the granular packing and the amount of cementitious compounds. Consoli et al. (2001) suggested that tensile strength is a function of the amount of cementitious compounds formed, which increases with curing time. The unconfined compressive strength is a function of both the

granular packing and the amount of cementitious compounds where the latter increases with curing time and the former being constant.

* The average values of the σt/qu-ratio in case of the fly ash-stabilized soils are higher than the average value in case of the lime/fly ash-stabilized soils. This resulted from the higher qu-values of lime/fly ash-stabilized soils compared to that of fly ash-stabilized soils (at 7 days and at long-term curing). On the other hand, the difference between σt-values of fly ash- and lime/fly ash-stabilized soils (at 7 days and at the long-term curing) is relatively small. This indicates that the addition of lime to fly ash leads to an improvement and an increase in the qu-value stronger than the σt-value.

* The correlation between qu-, CBR-, and σt-measurement (on one hand) and Vp-measurement (on the other hand) for the three tested stabilized soils shows that the variation of Vp-values of the three studied soils (due to the addition of lime, fly ash, and lime/fly ash and curing for 7 days) is relatively similar to the variation of qu-, CBR-, and σt-values (see Fig. 3.1, 4.1, 5.1, & 7.1).

* The correlation between Vp-, qu-, and E_{secant}-measurement of the three tested lime-, fly ash-, and lime/fly ash-stabilized soils with long-term curing provides that the variation of Vp-values with curing time is similar to the variation of both the unconfined compressive strength (qu)- and the elasticity modulus (E_{secant})-values. The correlation also points to the fact that the Vp-measurement is sensitive to changes in unconfined compressive strength (qu) and elasticity modulus (E_{secant}) with curing time.

* The ultrasonic testing method is a practical, simple, and fast method to evaluate lime-, fly ash-, and lime/fly ash-stabilized soil characteristics and the soil stabilization process. It can be applied in stabilization applications (as a non-destructive method) to estimate the uniformity of the lime-, fly ash-, and lime/fly ash-stabilized soils on the field scale. This method requires many investigations in the future to develop guidelines for different soils stabilized with different chemical additives.

9.3 Hydraulic conductivity

* The compaction process without chemical additives contributed to a reduction of the K-value of the three tested soils compared to the K-value of the natural soils. The K-value of organic silt was strongly affected by the compaction process compared to both tertiary clay and weathered soil. This is due to an improvement of the granular packing after the compaction process of the organic silt which leads to better improvement compared to the improvement of both tertiary clay and weathered soil. This is due to the fact that natural

organic silt has better uniformity compared to both tertiary clay and weathered soil. The K-value of tertiary clay was slightly influenced and reduced by the compaction.

* In the case of both fly ash- and lime/fly ash-stabilization process, the fly ash- and lime/fly ash-addition resulted in an increment of the hydraulic conductivity for the three tested soils in comparison to the untreated compacted soils.

* In the case of both tertiary clay and weathered soil, the addition of lime to fly ash led to a decrement of the K-values compared to the K-values of soil-fly ash mixtures at 7 days and at long-term curing (28, 56, and 180 days). In case of the organic silt, the addition of fly ash alone resulted in a reduction of the K-values in comparison to the K-values of lime/fly ash-soil mixtures. This may partly be as a result of a better improvement of the grain size distribution (improvement of uniformity) after mixing of organic silt with the silt-fine sand fraction fly ash alone and subsequently, better improvement of the granular packing after the compaction process compared to the improvement of both the inorganic tertiary clay and the weathered soil. Moreover, this is partly due to (as the early mention) a filling of the relatively large void spaces in the organic silt with the fly ash particles where the hydration reaction was not inhibited (in these void spaces) by organic matter (Hebit & Farrel, 2003).

* With an increase in curing time from 7 to 28 days, the hydraulic conductivity strongly increased, where fly ash- and lime/fly ash-soil mixtures (except for the weathered soil-fly ash mixtures) have the maximum K-values and K-value gain factors at 28 days curing.

* The increment of the hydraulic conductivity with fly ash- and lime/fly ash-addition is explained by the development of new cementitious compounds due to pozzolanic reaction and this leads to the formation of a new soil structure with highly open fabric arrangement (Narasimha & Rajasekarm, 1996). The pozzolanic reaction causes an alteration in the mineral structure of the treated stabilized soils and results in a more stable silt-sand like structure. This structure produces a soil with a more open fabric which leads to an increase in the hydraulic conductivity values (Nalbantoglu & Gucbilmez, 2002).

* In the present study, the maximum increase of K-value was at 28 days in case of both the fly ash and the lime/fly ash stabilized soils (except, the K-values of fly ash-stabilized weathered soil after 7 days were higher than the K-values after 28 days). This increase in the hydraulic conductivity resulted from both the flocculation (after the cation exchange “short-term” reaction, this leads to an aggregation and increase of the particles diameter) and the pozzolanic reaction (formation of cementitious compounds and new secondary minerals). The new cementitious compounds grow (until 28 days curing) around the particles and the aggregates resulting in additional increase in the soil particles diameter and generation of a

stabilized soil with silt-fine sand like structure (open fabric). This contributes to an increase in the hydraulic conductivity. With an increase in the curing time, 56 and 180 days, the hydraulic conductivity reduced. This is due to the formation of additional new cementitious compounds within the existing pore spaces (pore spaces of the silt-fine sand like structure) of the stabilized soil as a matrix and this is responsible for the reduction of the hydraulic conductivity by reducing the effective pore radius with the long-term curing.

* From the above, it is clear that, the influence of lime, fly ash, and lime/fly ash on the geotechnical properties is unique for each soil and the chemical additive. The three tested soils are relatively inhomogenous and have relatively different microstructural properties due to the difference of physico-chemical nature of their components. Additionally, the chemical reactions between the soil particles and the chemical additives varied depending on the chemical and mineralogical composition of each soil and additive.

9.4 Effect of soil type and organic matter on the stabilization process:

Inorganic tertiary clay has the strongest reactivity with the chemical additives and its geotechnical properties were strongly improved compared to that of both the organic silt and the weathered soil. This is because of the fact that, it contains the highest amount of clay particles < 2µm = 47% (kaolinite, montmorillonite, and halloysite), the highest percent fines (= 91%), the lowest organic content (= 3%), the highest specific surface area (28.53 m^2/g), and a pH-value > 7 (= 7.65) compared to the organic silt and the weathered soil. In addition, this results from different mineralogical composition where tertiary clay contains montmorillonite which reacts strongly with the chemical additives (lime, fly ash, and lime/fly ash) in comparison to kaolinite and halloysite in the weathered soil and the organic silt, respectively, which react slowly and weakly with the chemical additives (Kézdi, 1979). The organic silt has weaker reactivity with lime and fly ash and smaller improvement of its geotechnical properties compared to that of inorganic tertiary clay and has relatively stronger reactivity with lime and fly ash compared to that of the acidic weathered soil. This is believed to be caused by the inhibition of the hydration reaction by organic matter in the soil, where organic silt contains organic content of 6.4%. Moreover, it contains relatively smaller specific surface area (13.56 m^2/g), the smallest amount of clay particles < 2µm = 6% (halloysite) compared to a larger amount of clay particles < 2µm = 47% in the case of inorganic tertiary clay. The weathered soil has the weakest reactivity with lime and fly ash and its geotechnical properties were slightly improved, because it contains small amount of clay particles < 2µm = 7% (kaolinite), relatively small percent fines (= 85%), the smallest specific surface area (8.45

m^2/g), and its pH-value is < 7 (= 2.65). It is a problematic acidic soil. This acidic environment of the weathered soil prevents the complete dissolution of silica and alumina from the clay- and fly ash-particles resulting in the formation of a small amount of soluble silica and alumina which react with calcium (lime) to form the cementitous compounds. Additionally, the weathered soil contains natural gypsum crystals (as a source of sulfate). The presence of sulfate led to the formation of ettringite crystals (after the compaction) which resulted in the destruction of the compacted soil structure and subsequently, a reduction of the strength gain development, especially with the long-term curing.

9.5 Durability

* In the case of durability (water-soaking) test, both the fly ash- and the lime/fly ash-organic silt mixtures have the lowest strength loss (strength loss due to water-soaking process = qu-soaked/ qu-unsoaked * 100). The fly ash- and lime/fly ash-weathered soil mixtures had the highest strength loss. Theoretically, the vacuum saturated soaked samples should have less strength in comparison to the strength of plain soaked samples. In the present study, some plain soaked samples (lime/fly ash-tertiary clay, -weathered soil, and –organic silt mixtures) have strength-values lower than the strength-values of the vacuum soaked samples, for the same mixtures. This may be due to the fact that the trapping of air bubbles in the plain soaked samples results in an excess internal damage during the unconfined compressive testing. All the tested stabilized mixtures passed successively in the freeze-thaw durability test. Three mixtures (lime/fly ash-tertiary clay and fly ash- and lime/fly ash-organic silt mixtures) passed successively in the wet-dry durability test and the other three mixtures (fly ash-tertiary clay and fly ash- and lime/fly ash-weathered soil mixtures) failed. Lime/fly ash-tertiary clay mixtures are more durable than fly ash-tertiary clay mixtures. Both the fly ash- and the lime/fly ash-organic silt mixtures are durable. In case of the wet-dry durability test, both the fly ash-and the lime/fly ash-weathered soil mixtures failed due to the formation of ettringite crystals, after the compaction process, which leads to destruction of the compacted soil structure and the binding between the soil particles resulting in a reduction of the strength gain.

9.6 SEM-, XRD-, and Calorimetry-analysis

* Scanning electron microscope studies indicated that the microstructures of the tested soils changed due to lime-, fly ash- and lime/fly ash-stabilization process and developed with the long-term curing. The stabilization process resulted in the formation of a silt-fine sand like structure (open fabric) characterized by a highly porous system. Additionally, the SEM-micrograph of fly ash- and lime/fly ash-stabilized weathered soil showed rod-like crystals (ettringite). The SEM-micrographs of natural and treated stabilized soils indicated the formation of new cementitous compounds and mineral crystals as a pozzolanic reaction product through the long-term curing contributed to increase the strength gain. These cementitous compounds join the soil particles together and increase the strength gain. The XRD-analysis confirmed that the presence of sulfate in the weathered soil led to ettringite-formation. After 28 days moist-curing of both fly ash- and lime/fly ash-stabilized weathered soil, the only new crystalline phase was ettringite where gypsum was consumed to form the ettringite crystals. There is no critical volume-changes (expansion) of both the fly ash- and the lime/fly ash stabilized weathered soil (moist-curing at 28 and 180 days) due to ettringite-formation (the increase of the volume < 1%). The calorimetry-analysis illustrated that the high value of CaO-content and the presence of calcite mineral in the natural organic silt contributed to an acceleration of the hydration reaction of the optimum lime- and lime/fly ash-organic silt mixtures. The maximum value of the heat of the hydration reaction (main peak height) for optimum lime (3%)-organic silt mixture spanned 3.5 hours and the entire heat flow was 59.28 J/g. The maximum value of the heat of the hydration reaction (main peak height) for both 3% lime/20% fly ash- and 2% lime/20% fly ash-organic silt mixtures spanned 7.5 hours and the entire heat flow was 99.02 and 85.85 J/g, respectively.

* Finally, Lippendorf fly ash can be utilized to treat and stabilize the soft fine grained soils (especially, tertiary clay and organic silt) as economical (cheaper) alternative to Portland cement and other (expansive) chemical stabilizers. The use of fly ash for stabilization applications is an environmental solution of the problems associated with its disposal process.

9.7 Emphasis

1- The use of hydrated lime alone is very suitable to stabilize the clayey soil which has a high amount of clay particles and high plasticity index.

2- The use of self-cementing (close to class C) fly ash alone is suitable to stabilize the fine grained soils which contain low amount of clay particles and high amount of silt particles such as organic silt.

3- The use of self-cementing or non-self cementing fly ash (close to class F) with lime is valuable to stabilize the fine grained soils, whether they contain low or high amount of clay particles, where the additional fly ash (pozzolanic material) is a source of the silica and the alumina which react with lime. In this case, the soil type has relatively less importance, whereas the reaction is between the fly ash and the lime.

4- In case of the clayey soil, the addition of fly ash content above 20% does not result in a further improvement of the geotechnical properties.

5- Sulfate content of fly ash must be not more than 10%, when fly ashes (having sulfate contents of 5 to 10 percent) have lower rate of initial hydration and the compaction delay has less influence on the compressive strengths. If fly ash contains sulfate content above 10%, high initial strengths have been observed, however, durability may be dramatically reduced. Furthermore, during initial hydration, there is a possibility to form ettringite crystals leading to expansion (Ferguson, 1993). Other investigators (i.e. Thomas & White, 2003) reported that sulfate content of fly ash must be not more than 5%.

6- Sulfate (Gypsum) bearing soil (such as weathered soil), treated with lime, requires to special technical methods *in situ*. National Lime Association (2004) reported that these technical methods *in situ* include “two applications of lime, the first before the first mixing and the second after the mellowing period. The moisture content of the soil is raised to 5% over the optimum during a multi-day mellowing period to soluble as many as sulfates as possible and to force ettringite to form before compaction. Once formed, ettringte is relatively stable and is unlikely to cause further problems. After the mellowing period, additional lime is added to the soil and construction proceeds normally”.

7- The acidic soil (pH-value < 7) is a problematic soil and needs a relatively more amount of lime and fly ash to treat.

9.8 Suggestions for the future:

* The use of fly ash for chemical soil-stabilization in Germany requires many investigations in the future including classification and identification of the different fly ash-types which derive from different sources of coal. Furthermore, various studies would be required to evaluate the use of fly ash to treat and stabilize different soil-types and to establish German guidelines and standard specifications for the fly ash- and lime/fly ash-stabilization process.

* The use of ultrasonic p-wave velocity method is a simple and practical (non-destructive) tool to evaluate the stabilized soils using lime, fly ash, and lime/fly ash and would need many studies to establish a guideline and standard specification.

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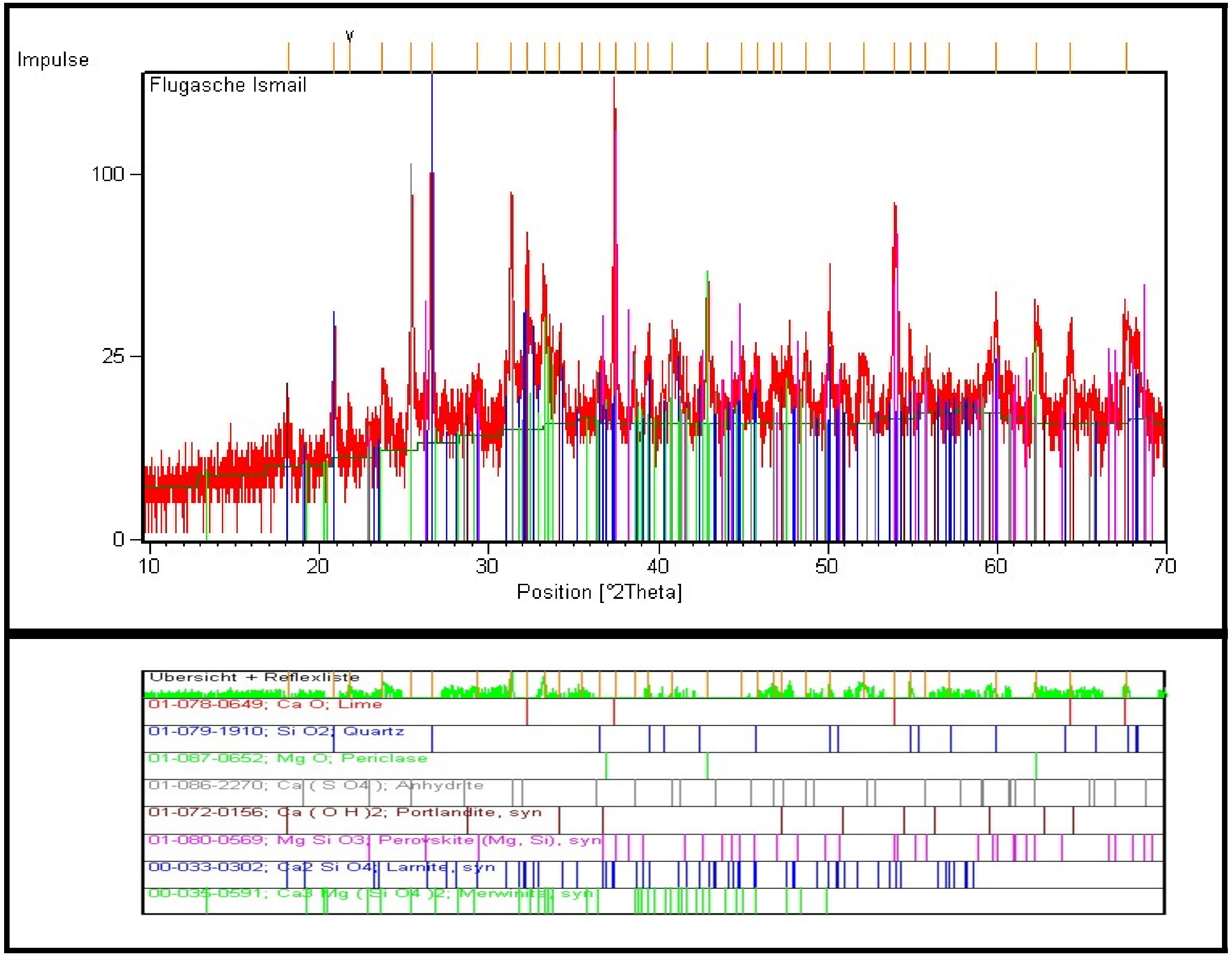
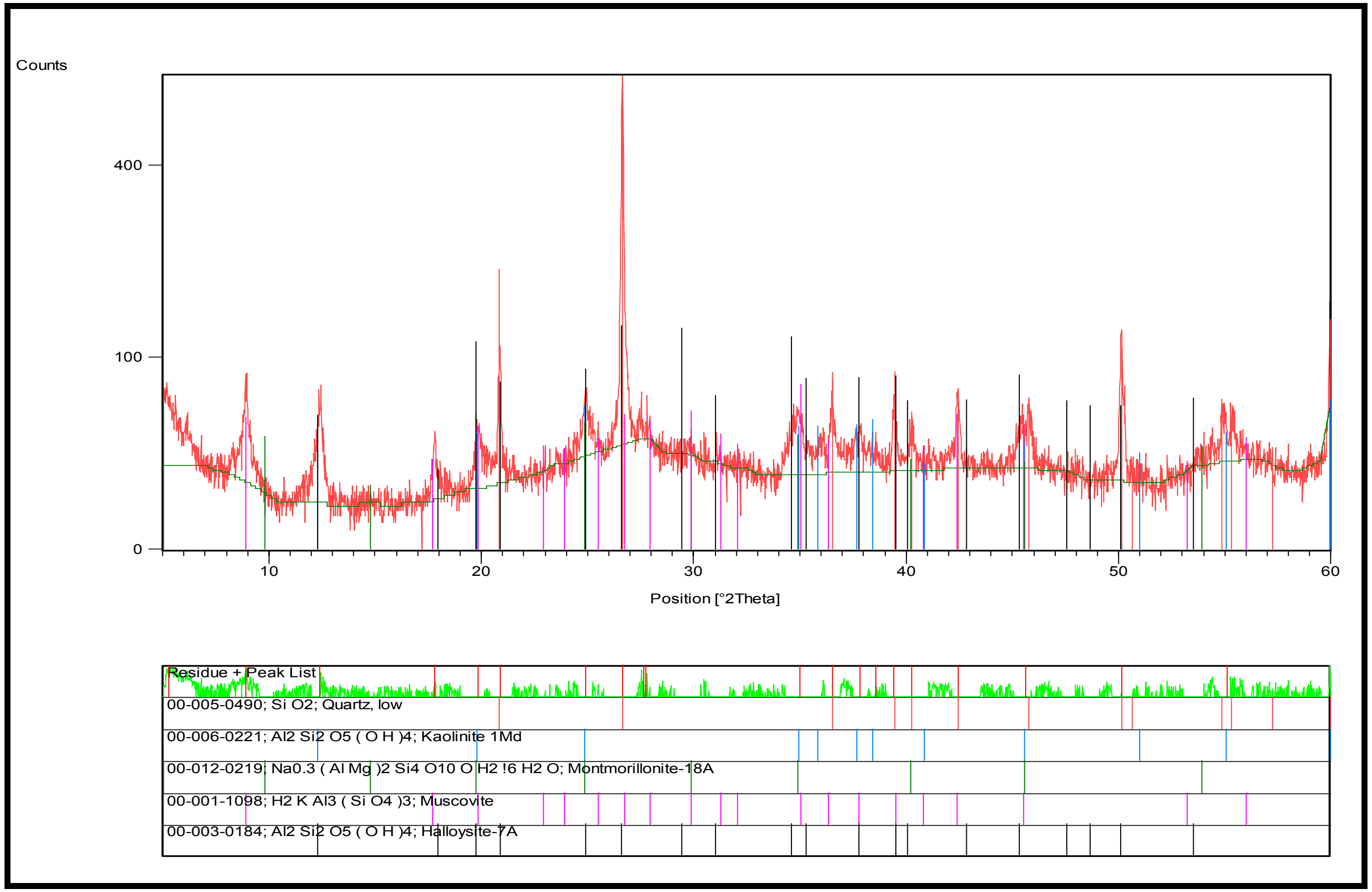
Appendixes (from No. 1 to No. 30

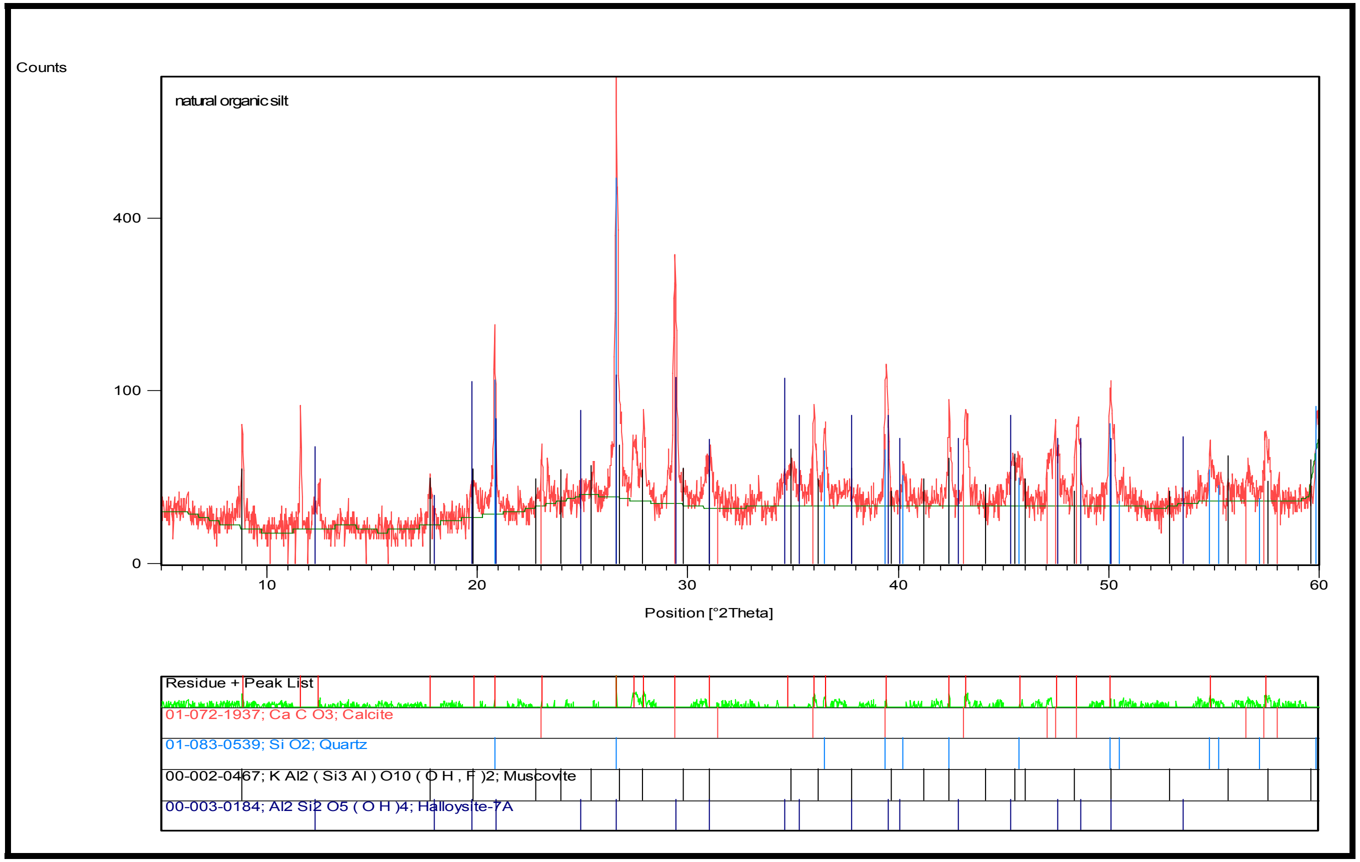
Appendix (1a) X-ray powder diffractogram of Lippendorf fly ash.



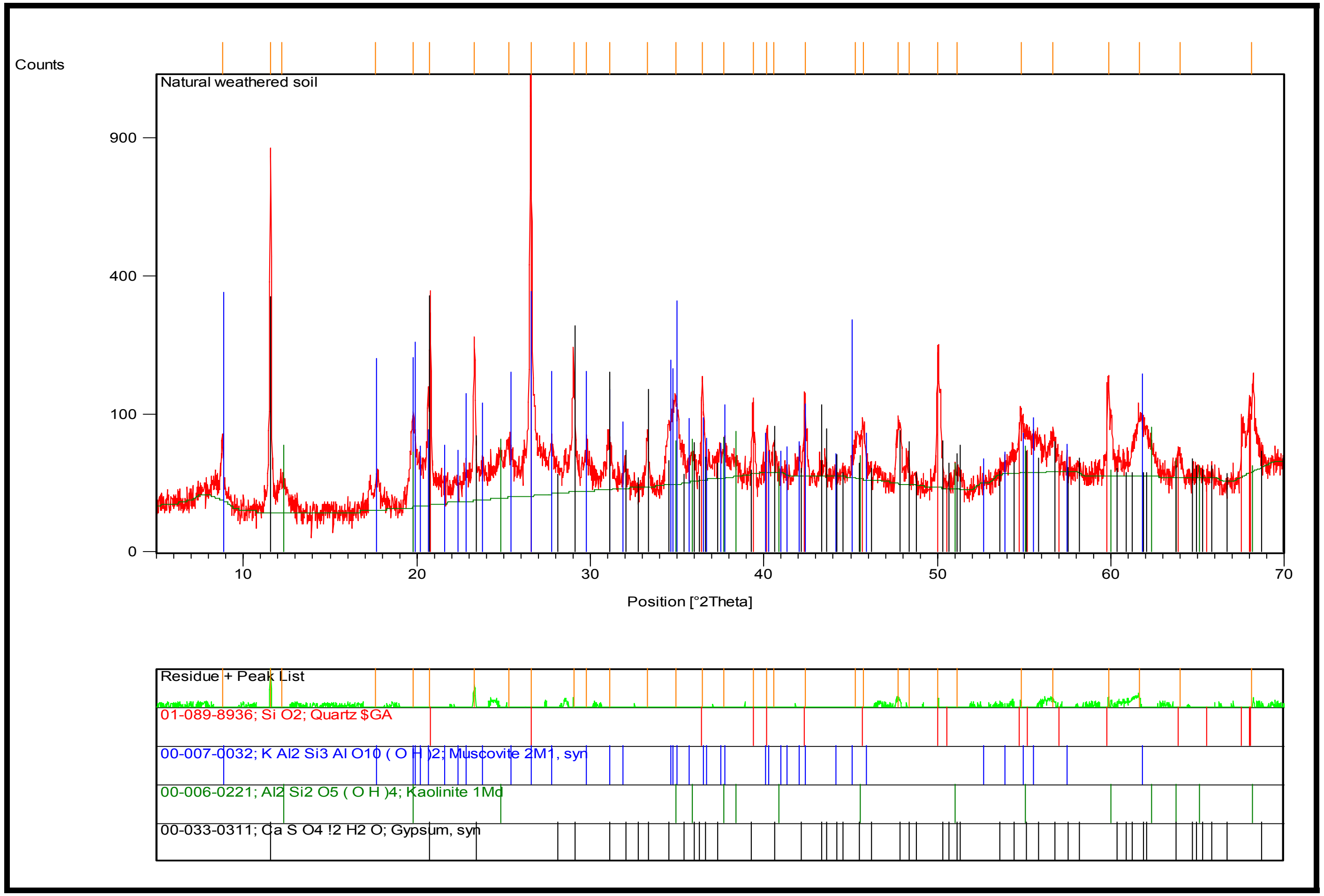
Appendix (1b) X-ray powder diffractogram of natural teriary clay.



Appendix (2) X-ray powder diffractogram of natural organic silt

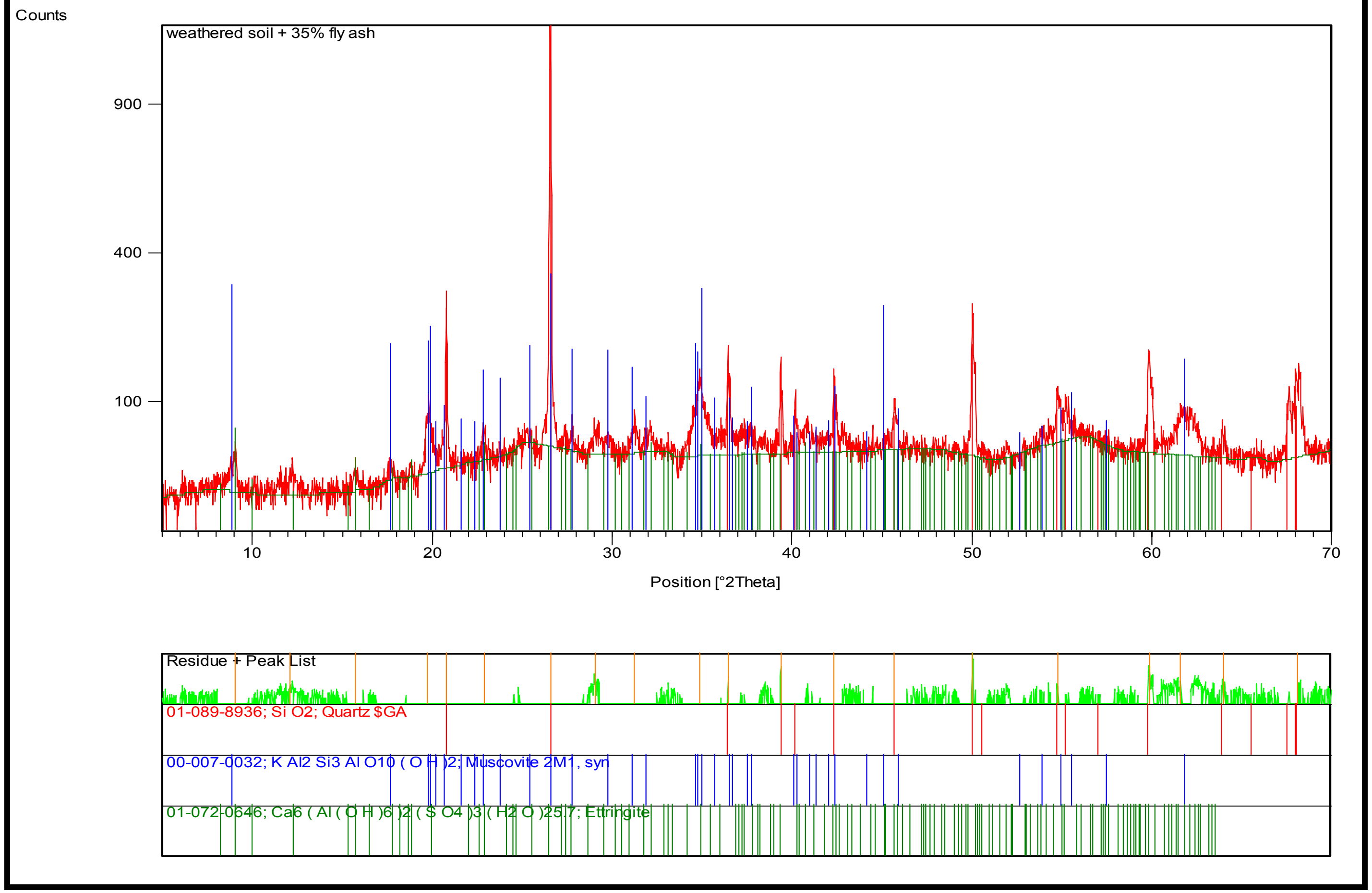


Appendix (3) X-ray powder diffractogram of natural weathered soil



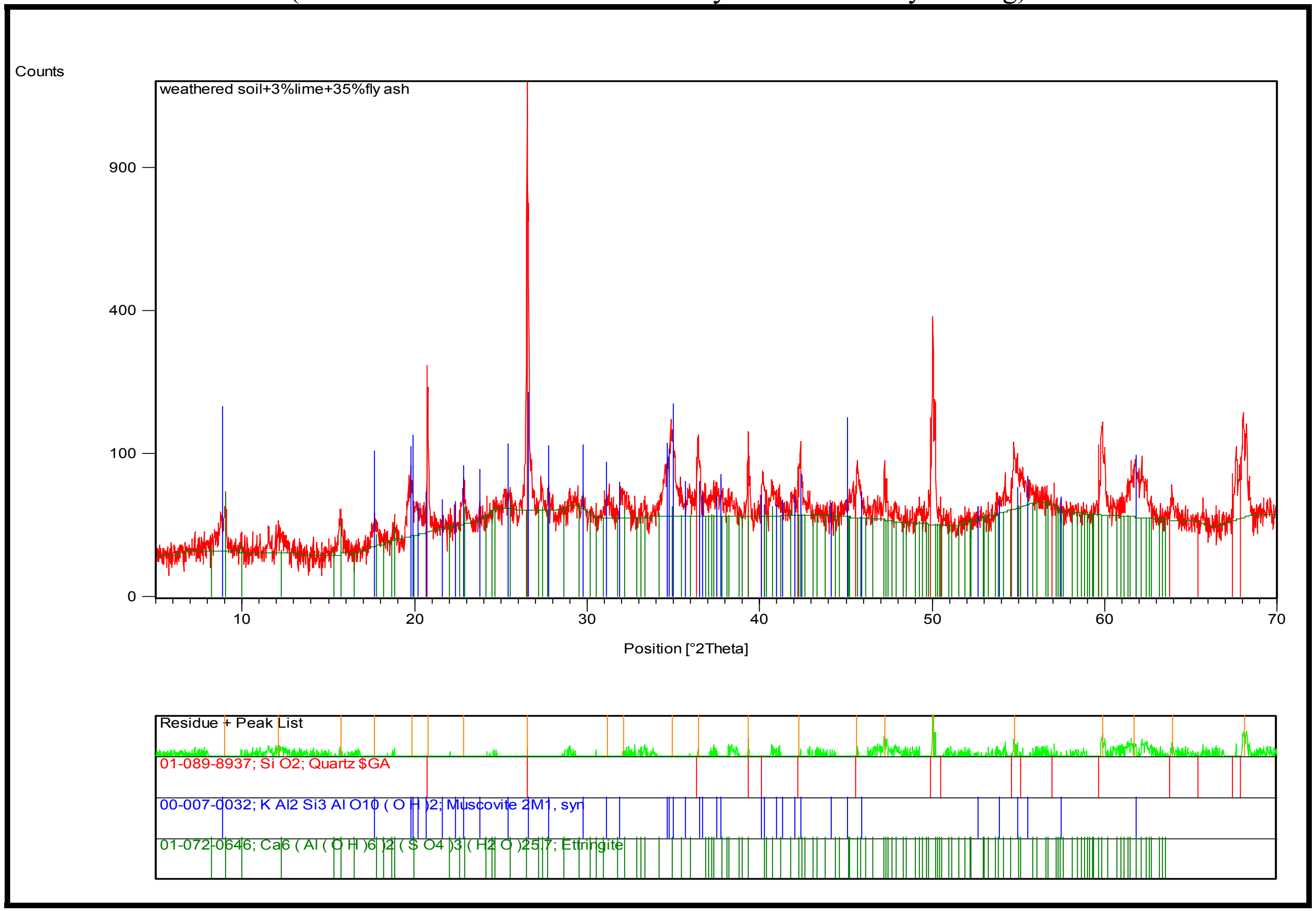
Appendix (4) X-ray powder diffractogram of treated stabilized weathered soil

(weathered soil + 35% fly ash after 28 days curing)



Appendix (5) X-ray powder diffractogram of treated stabilized weathered soil

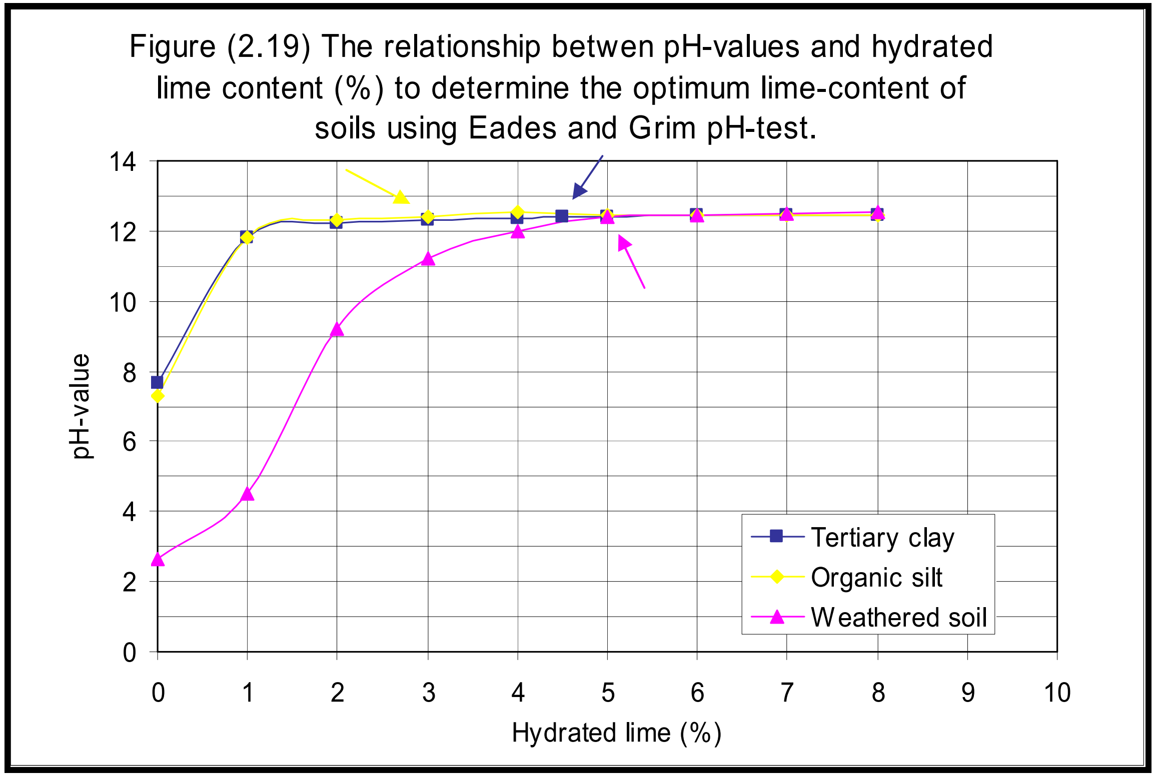
(weathered soil + 3% lime + 35% fly ash after 28 days curing)



Appendix (6) pH-measurement of natural soils and soil-lime, soil-fly ash, and soil-lime/fly ash mixtures to determine the optimum lime-, fly ash- and lime/fly ash-content for the studied soils. T= Temperature (°C), L% is the optimum lime content minus 2% in case of the tertiary clay and the weathered soil and is the optimum lime content minus 1% in case of the organic silt. The Blue numbers indicate to the optimum chemical additive content according to pH-test.

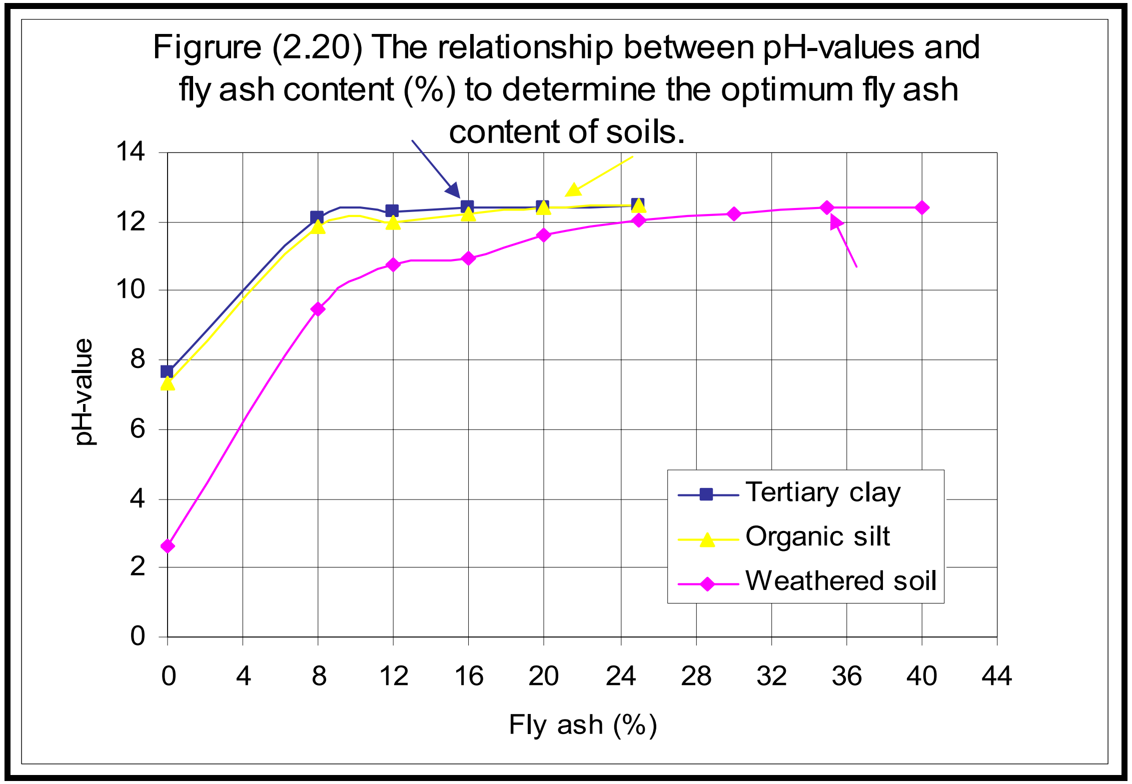
Type of chemical additive	Percent of chemical additive %	Tertiary clay		Organic silt		Weathered soil	
		Average pH	T	Average pH	T	Average pH	T
Without agent	0	7.65	25.00	7.31	25.00	2.65	25.00
Hydrated lime	1	11.80	25.00	11.80	25.00	4.50	25.00
	2	12.23	25.00	12.32	25.00	9.20	25.00
	3	12.32	25.00	12.40	25.00	11.20	25.00
	4	12.38	25.00	12.43	25.00	12.00	25.00
	4.5	12.40	25.00	-	25.00	-	25.00
	5	12.42	25.00	12.44	25.00	12.40	25.00
	6	12.44	25.00	12.45	25.00	12.45	25.00
	7	12.46	25.00	12.46	25.00	12.50	25.00
	8	12.47	25.00	12.47	25.00	12.55	25.00
Fly ash	8	12.09	25.00	11.85	25.00	9.50	25.00
	12	12.31	25.00	12.00	25.00	10.75	25.00
	16	12.40	25.00	12.20	25.00	10.97	25.00
	20	12.43	25.00	12.42	25.00	11.62	25.00
	25	12.46	25.00	12.45	25.00	12.02	25.00
	30	-	25.00	-	25.00	12.25	25.00
	35	-	25.00	-	25.00	12.40	25.00
	40	-	25.00	-	25.00	12.40	25.00
Lime/fly ash	L% /8%F	12.47	25.00	12.36	25.00	12.28	25.00
	L% /12%F	12.49	25.00	12.45	25.00	12.38	25.00
	L% /16%F	12.51	25.00	12.48	25.00	12.39	25.00
	L% /20%F	12.52	25.00	12.52	25.00	12.42	25.00
	L% /25%F	12.53	25.00	12.54	25.00	12.46	25.00

Appendix (7) Relationship between pH-value and hydrated lime content



The arrows point to the optimum lime content

Appendix (8) Relationship between pH-value and fly ash content

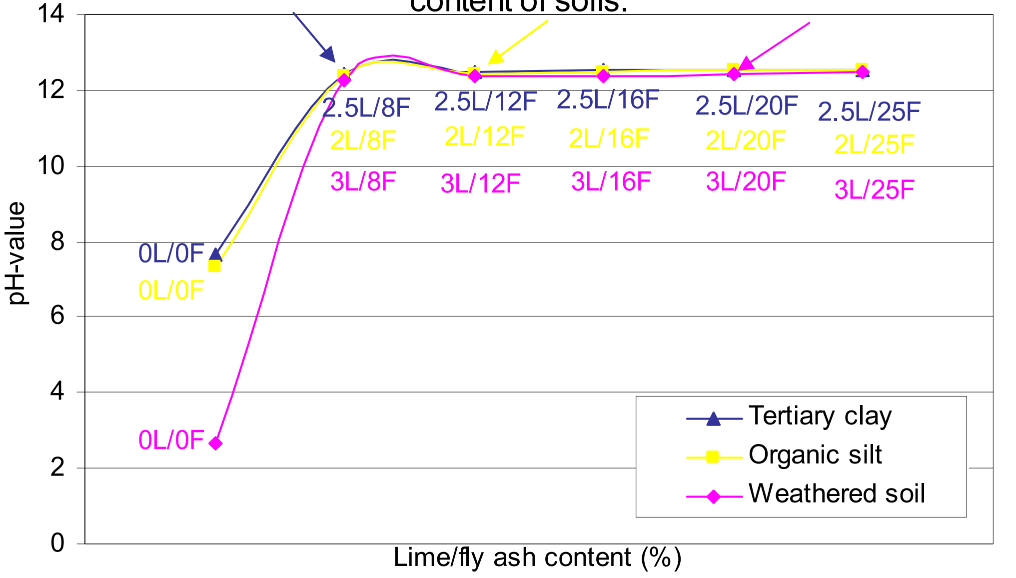


Figrure (2.20) The relationship between pH-values and fly ash content (%) to determine the optimum fly ash content of soils.

The arrows point to the optimum fly ash content for each soil

Appendix (9) Relationship between pH-value and hydrated lime/fly ash content

Fig. (2.21) The relationship between pH-values and lime/fly ash content (%) to determine the optimum lime/fly ash-content of soils.



The arrows point to the optimum lime/fly ash content for each soil

Appendix (10) Geotechnical properties of untreated compacted- and treated stabilized-tertiary clay with several blending.

Admixture mixing		Atterberg limits		Plasticity index	Gs	Compaction		Curing time	CBR-value	qu-value	Tensile strength σt-	Vp-value
L (%)	FA (%)	LL (%)	PL (%)	PI (%)	g/cm³	MDD g/cm³	OMC (%)	Days	(%)	kN/m²	kN/m²	m/sec
0	0	61.47	28.57	32.90	2.650	1.445	23.77	-	4.6	131.21	-	667
4.5*	0	65.32	48.61	16.71	2.573	1.366	29.40	7	60	1034.00	109.25	1418
6.5	0	62.95	46.13	16.82	2.534	1.349	31.12	7	61.8	1147.80	160.25	1569
8.5	0	66.70	46.10	20.6	2.456	1.328	32.53	7	62.3	1221.70	183.60	1600
0	8	76.37	46.80	29.57	2.614	1.369	30.85	7	21.9	366.88	43.70	1039
0	12	73.97	48.96	25.01	2.607	1.364	31.17	7	36.6	594.90	72.90	1215
0	16*	70.75	52.34	18.41	2.606	1.361	31.26	7	39.4	820.38	102.00	1309
								28	73.8	1459.27	185.80	1513
								56	77.0	1660.64	211.26	1536
								180	94.9	1791.37	276.82	1600
0	20	69.27	51.64	17.62	2.588	1.353	31.53	7	61.1	1064.97	153.00	1537
0	25	65.88	50.20	15.68	2.594	1.344	31.61	7	51.6	950.00	152.95	1485
2.5*	8*	71.52	48.51	23.01	2.605	1.328	30.39	7	46.8	905.59	123.85	1314
1.5	16	70.40	48.90	21.50	2.615	1.333	30.50	7	59.7	1424.42	131.15	1491
2.5	16	70.34	51.96	18.38	2.582	1.327	30.53	7	87.2	1476.28	142.06	1496
								28	109	2256.57	240.4	1655
								56	168.3	3081.50	335.10	1786
								180	234.2	3505.23	400.66	1875
4.5	16	70.20	51.70	18.50	2.579	1.310	30.81	7	81.8	1363.75	138.41	1435
6.5	16	54.30	35.20	19.10	2.613	1.323	32.57	7	79.5	1159.56	127.49	1371

L = Lime content (%)
FA = Fly ash content (%)
MDD = Maximum dry density (in Proctor test)
OMC = Optimum moisture content (in Proctor test)
LL = Liquid limt
PL = Plastic limit
Gs = Specific gravity
CBR = California bearing ratio
qu = Unconfined compression strength
Vp = P-wave velocity
* Optimum content according to pH-test

Appendix (11) Geotechnical properties of untreated compacted- and treated stabilized-organic silt with several blending.

Admixture mixing		Atterberg limits		Plas-ticity index	Gs	Compaction		Curing time	CBR-value	qu-value	Tensile strength σt	Vp-value
L (%)	FA (%)	LL (%)	PL (%)	PI (%)	g/cm^3	MDD g/cm^3	OMC (%)	Days	(%)	kN/m^2	kN/m^2	m/sec
0	0	50.00	29.16	20.84	2.550	1.455	27.61	-	7.4	136.91	-	465
3*	0	55.33	41.75	13.58	2.510	1.386	28.40	7	16.8	439.49	87.40	852
5	0	54.75	42.12	12.63	2.482	1.383	30.29	7	23.4	634.40	109.30	961
7	0	54.62	42.68	11.94	2.500	1.345	30.76	7	23.4	628.03	101.95	917
0	8	53.43	36.91	16.52	2.551	1.401	28.94	7	14.7	322.29	37.90	781
0	12	54.01	37.80	16.21	2.559	1.404	29.74	7	28.7	532.48	58.30	876
0	16	55.11	38.95	16.16	2.559	1.407	29.94	7	37.0	620.38	72.85	953
0	20*	54.10	39.92	14.14	2.560	1.410	30.34	7	45.8	685.35	76.49	1011
								28	58.2	1331.26	153.00	1228
								56	94.4	1385.55	167.55	1297
								180	105.0	1738.98	254.97	1420
0	25	52.4	38.9	13.47	2.562	1.416	30.75	7	47.8	964.33	109.27	1098
2*	12*	51.05	37.18	13.87	2.552	1.404	26.53	7	35.2	729.65	65.55	929
2	20	51.00	3738	13.01	2.619	1.410	27.85	7	38.4	866.43	80.13	991
								28	61.1	1348.78	152.98	1178
								56	80.4	1648.44	189.41	1342
								180	103.1	2148.35	258.61	1434
3	20	50.01	38.21	11.80	2.555	1.404	26.74	7	40.2	871.88	83.78	1013
5	20	49.90	37.60	12.30	2.610	1.394	27.10	7	27.4	796.11	72.90	949

Appendix (12) Geotechnical properties of untreated compacted- and treated stabilized-weathered soil with several blending.

Admixture mixing		**Atterberg limits**		**Plas-ticity index**	**Gs**	**Compaction**		**Curing time**	**CBR-value**	**qu-value**	**Tensile strength σt**	**Vp-value**
L (%)	**FA (%)**	**LL (%)**	**PL (%)**	**PI (%)**	**g/cm³**	**MDD g/cm³**	**OMC (%)**	**Days**	**(%)**	**kN/m²**	**kN/m²**	**m/sec**
0	**0**	**63.72**	**32.36**	**31.36**	**2.64**	**1.502**	**25.74**	**-**	**3.2**	**173.25**	**-**	**721**
5*	**0**	**85.80**	**58.36**	**27.44**	**2.579**	**1.282**	**33.34**	**7**	**17.1**	**309.55**	**47.35**	**882**
7	**0**	**87.90**	**60.69**	**27.21**	**2.551**	**1.265**	**33.72**	**7**	**17.6**	**329.90**	**50.95**	**1034**
9	**0**	**88.30**	**61.26**	**27.03**	**2.574**	**1.260**	**33.91**	**7**	**16.8**	**298.09**	**40.05**	**1000**
0	**8**	**88.65**	**53.78**	**34.88**	**2.64**	**1.343**	**28.34**	**7**	**15.3**	**294.27**	**36.40**	**979**
0	**12**	**87.84**	**53.96**	**33.88**	**2.638**	**1.313**	**31.11**	**7**	**17.5**	**301.54**	**40.04**	**1018**
0	**16**	**86.32**	**54.69**	**31.63**	**2.635**	**1.253**	**32.06**	**7**	**22.8**	**335.50**	**50.95**	**1020**
0	**20**	**86.29**	**55.44**	**30.85**	**2.628**	**1.247**	**32.27**	**7**	**26.4**	**389.81**	**69.20**	**1021**
0	**25**	**86.28**	**57.03**	**29.25**	**2.618**	**1.350**	**32.30**	**7**	**43.5**	**526.12**	**72.90**	**1147**
0	**35***	**84.63**	**60.40**	**24.23**	**2.604**	**1.370**	**32.70**	**7**	**50.2**	**938.85**	**110.75**	**1236**
								28	**51.0**	**1151.07**	**138.41**	**1367**
								56	**62.9**	**1184.21**	**152.98**	**1371**
								180	**95.3**	**1412.79**	**182.12**	**1439**
3*	**20 ***	**79.44**	**57.12**	**22.32**	**2.624**	**1.240**	**30.94**	**7**	**44.5**	**713.39**	**109.30**	**1297**
3	**35**	**69.09**	**45.93**	**23.16**	**2.613**	**1.360**	**25.79**	**7**	**79.2**	**1327.30**	**116.56**	**1368**
								28	**84.4**	**1619.81**	**145.70**	**1402**
								56	**103.3**	**1751.11**	**160.27**	**1453**
								180	**115.0**	**2297.16**	**236.80**	**1622**
5	**35**	**74.24**	**48.89**	**25.35**	**2.584**	**1.357**	**25.72**	**7**	**80.2**	**1421.54**	**182.10**	**1392**
7	**35**	**74.56**	**45.31**	**28.27**	**2.573**	**1.345**	**25.54**	**7**	**76.6**	**1416.49**	**174.80**	**1382**
8	**35**	**83.72**	**49.76**	**33.96**	**2.571**	**1.346**	**24.10**	**7**	**72.4**	**1376.68**	**149.34**	**1370**

Appendix (13) Calculation of elasticity modulus (E_{Secant}) of untreated compacted- and treated stabilized-soils for different admixtures and curing times

Soil type	Mixtures	$\Delta\sigma/\Delta\varepsilon$	E_{Secant} (MPa)
Tertiary clay	Untreated compacted	40.704-7.643/0.01042-0.00342	4.70
	4.5% lime 7 days	639.49-31.847/0.00758-0.00358	100.50
	16% fly ash 7 days	385.99-112.102/0.00192-0.00042	100.80
	16% fly ash 28 days	659.47-112.56/0.00358-0.00342	300.40
	16% fly ash 56 days	977.991-402.797/0.00559-0.00358	300.00
	16% fly ash 180 days	1274.21-365.84/0.00517-0.00367	600.10
	2.5%lime/8% fly ash 7	657.422-222.967/0.00758-0.00433	100.30
	2.5%L/16% fly ash 7	842.68-399.21/0.00667-0.00508	200.80
	2.5%L/16% fly ash 28	2043.59-1193.91/0.01025-0.00917	700.90
	2.5%L/16% fly ash 56	1734.957-632.007/0.00417-0.00292	882.30
	2.5%L/16%fly ash 180	2041.568-1555.754/0.00442-0.00400	1156.70
Organic silt	Untreated compacted	39.548-8.479/0.01533-0.00175	2.30
	3% lime 7 days	133.758-20.382/0.00333-0.00067	42.60
	20% fly ash 7 days	389.809-54.777/0.00925-0.00533	90.00
	20% fly ash 28 days	1217.922-393.928/0.01058-0.00533	100.60
	20% fly ash 56 days	1046.466-662.411/0.00408-0.00183	168.90
	20% fly ash 180 days	1563.6-289.609/0.00667-0.00167	200.60
	2% L/12% fly ash 7	282.037-69.236/0.00342-0.00142	100.10
	2% L/20% fly ash 7	542.250-216.639/0.00583-0.00283	100.90
	2% L/20% fly ash 28	1009.422-510.142/0.00650-0.004	200.00
	2% L/20% fly ash 56	1223.943-669.011/0.00600-0.00450	369.96
	2% L/20% fly ash 180	1508.086-1026.55/0.0045-0.00375	642.01
Weathered soil	Untreated compacted	59.873-25.478/0.01375-0.00867	6.80
	5% lime 7 days	192.357-38.217/0.00942-0.00467	33.00
	35% fly ash 7 days	626.752-62.420/0.00617-0.00108	100.10
	35% fly ash 28 days	979.222-416.546/0.00967-0.006	100.30
	35% fly ash 56 days	675.07-260.608/0.00767-0.00342	97.50
	35% fly ash 180 days	675.079-459.879/0.00967-0.008	100.50
	3% L/20% fly ash 7	203.688-411.884/0.00275-0.00058	72.00
	3% L/35% fly ash 7	1223.847-917.989/0.00908-0.00783	200.30
	3% L/35% fly ash 28	828.896-411.884/0.00683-0.005	200.50
	3% L/35% fly ash 56	1286.556-711.755/0.01067-0.00875	299.40
	3% L/35% fly ash 180	1509.289-1034.149/0.00342-0.00275	709.16

Appendix (14) calculation of the loss of weight for the stabilized soils after 12 cycles freezing/thawing–durability test (ASTM D560).

sSample	wet weight after 7 days	W %	B gm	C 1 gm	C 2	C 3	C 4	C 5	C 6	C 7	C 8	C 9	C 10	C 11	C 12	qu- KN/m²	W %	F.D. W gm.	Loss of weight %	Re-sult
Clay 16F	1735.9	31.2	1323.1	1735.2	1724.8	1711.5	1721.1	1722.4	1710.9	1702.3	1695.7	1692.7	1690.3	1680.5	1677.7	891.72	28.20	1308.7	1.10	P
Clay 2.5L /16F	1710.2	30.5	1310.5	1708.4	1707.2	1706.5	1704.4	1704.7	1703.6	1701.9	1701.1	1700.2	1700.5	1699.9	1699.8	1044.6	32.32	1284.7	1.97	p
Silt 20F	1710.4	30.3	1312.7	1703.3	1703.0	1699.2	1693.2	1681.3	1676.0	1672.6	1664.5	1663.5	1660.1	1598.3	1690.7	1082.8	30.26	1297.9	1.13	P
Silt 2L/ 20F	1667.9	26.8	1315.4	1655.5	1645.2	1631.3	1623.7	1616.5	1587.6	1575.3	1570.4	1565.4	1558.1	1552.5	1546.7	957.96	23.79	1249.5	5.00	P
w. soil 35F	1660.2	32.7	1251.1	1626.6	1614.9	1612.3	1611.7	1610.9	1608.8	1604.7	1602.8	1602.5	1601.6	1600.1	1598.4	1031.8	28.87	1240.3	1.00	P
w. soil 3L/ 35F	1637.1	25.8	1301.4	1636.4	1634.7	1633.2	1632.6	1627.6	1626.1	1623.9	1623.6	1621.2	1615.7	1612.3	1609.7	1464.9	26.75	1269.9	2.40	P

C = Cycle, P = Pass, F = Fail

B = Original calculated oven-dry weight   A= Original calculated oven-dry weight minus final corrected oven-dry weight.

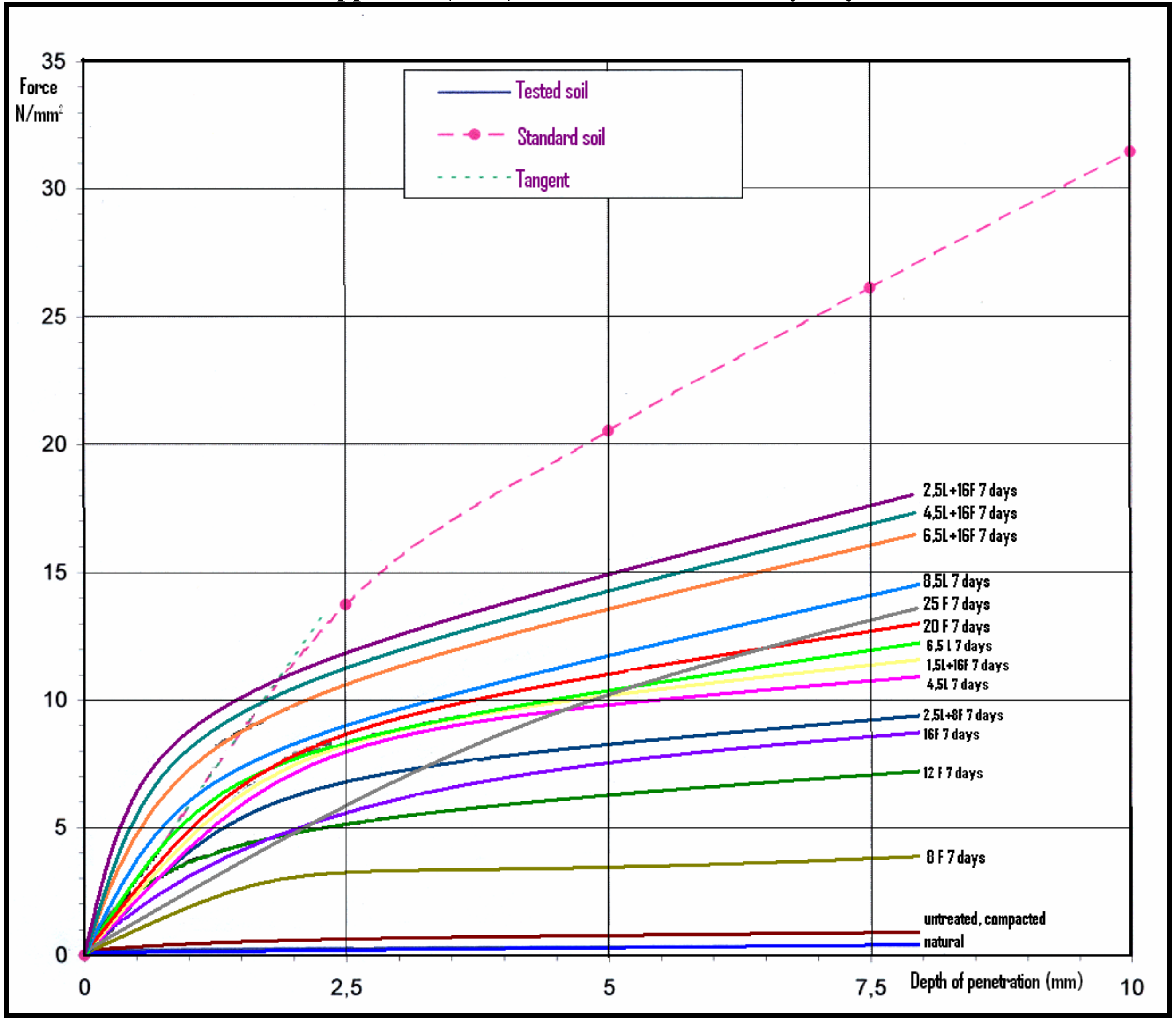
W= Water content, F.D.W = Final corrected oven-dry weight.   Loss of weight, % = (A/B) * 100

Appendix (15) calculation of the loss of weight for the stabilized soils after 12 cycles wetting/drying–durability test (ASTM D559).

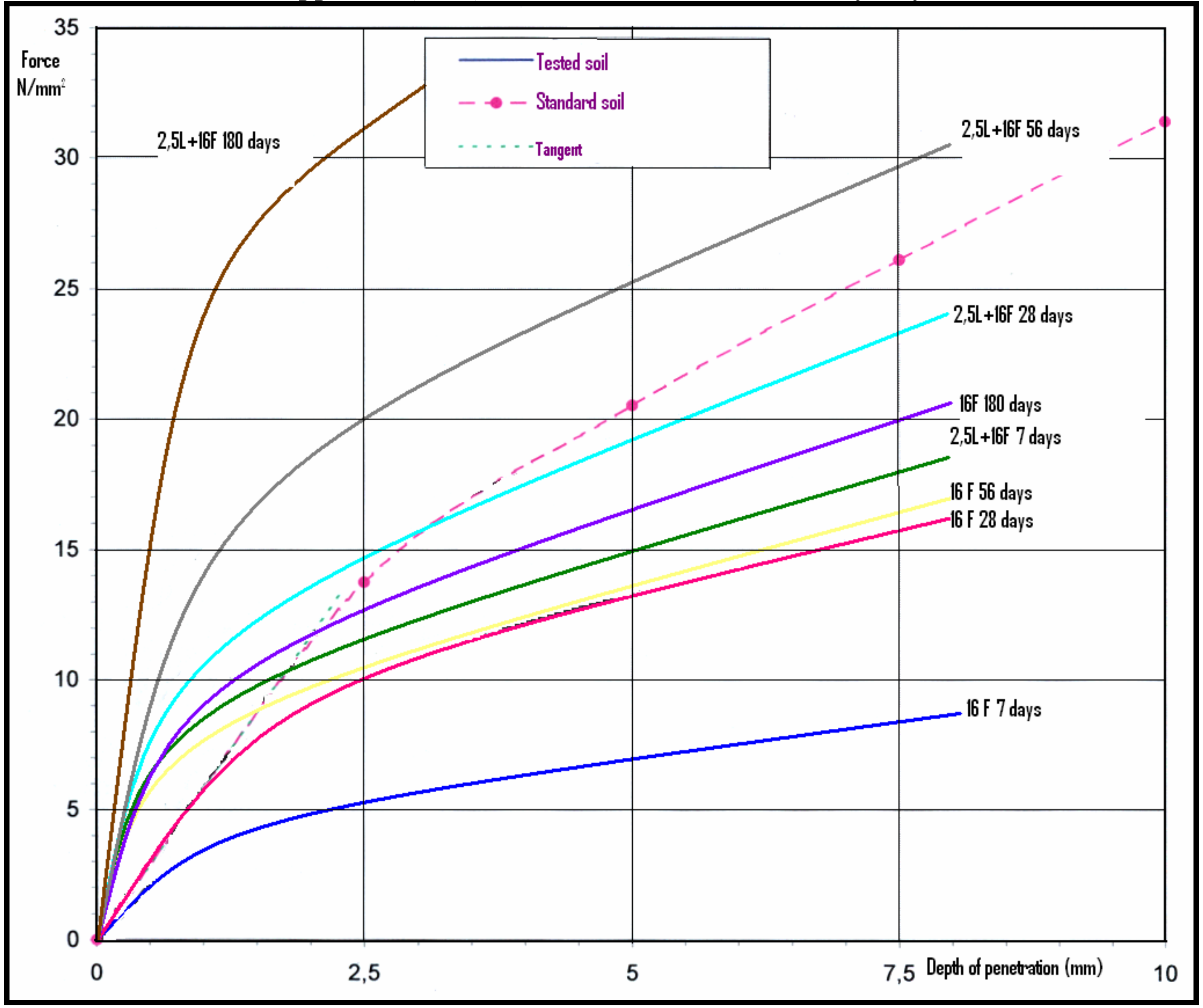
sSample	wet weight after 7 days	W %	B gm	C 1 gm	C 2	C 3	C 4	C 5	C 6	C 7	C 8	C 9	C 10	C 11	C 12	qu KN/ m²	W %	F.D. W gm.	Loss of weight %	Re- sult
Clay 16F	1732.8	31.2	1323.1	1609.8	1584.9	1561.6	1551.1	1512.8	1490.6	1477.5	1459.6	1420.7	1414.6	1410.4	1405.5	649.68	26.57	1110.8	16.10	F
Clay 2.5L /16F	1720.3	30.5	1310.2	1646.9	1618.7	1615.7	1593.1	1590.5	1583.8	1581.6	1577.3	1575.4	1573.1	1571.5	1569.1	1439.5	26.90	1236.5	5.60	P
Silt 20F	1705.2	30.3	1308.7	1646.9	1621.2	1606.9	1597.6	1595.0	1594.7	1592.3	1590.7	1593.5	1589.1	1583.2	1580.4	1477.7	23.30	1281.8	2.10	P
Silt 2L/ 20F	1653.6	26.8	1304.1	1625.6	1625.0	1623.3	1619.4	1611.3	1600.6	1596.5	1588.6	1580.5	1574.2	1562.4	1555.8	1541.4	21.12	1284.5	1.50	P
w.soil 35F	1618.7	32.7	1219.8	1566.1	1501.8	1499.8	1438.2	1410.3	1400.5	1398.6	1380.5	1345.8	1319.6	1311.2	1292.6	458.60	22..30	1056.9	13.40	F
w.soil 3L/ 35F	1654.7	25.8	1315.3	1566.6	1564.9	1546.5	1505.3	1503.1	1492.1	1480.4	1471.0	1469.5	1460.9	1455.3	1448.5	1019.1	23.60	1171.9	10.90	F

B = Original calculated oven-dry weight. A = Original calculated oven-dry weight minus final corrected oven-dry weight.
Loss of weight, % = (A/B) * 100
C = Cycle, P = Pass, F = Fail
W= Water content, F.D.W = Final corrected oven-dry weigh

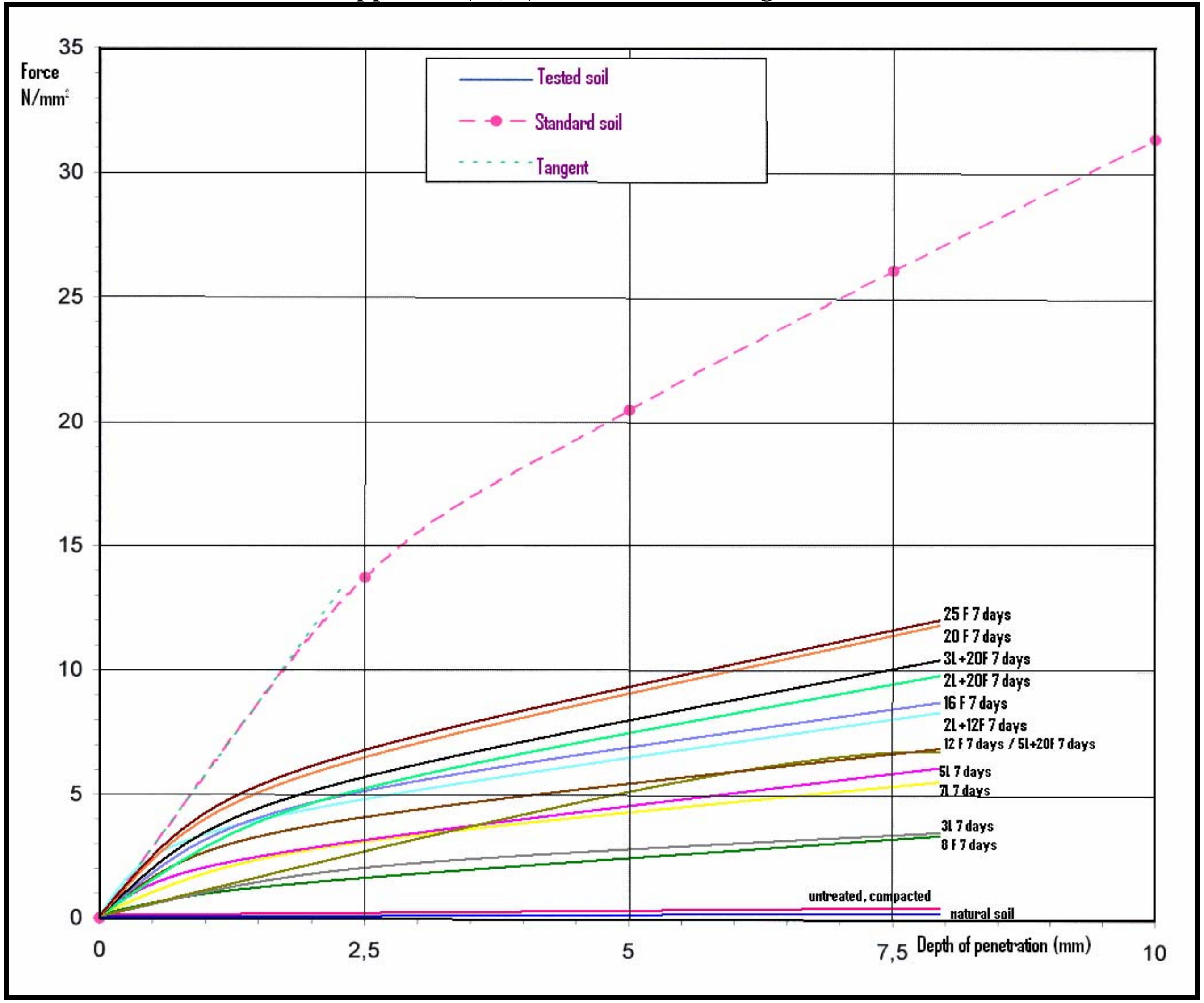
Appendix (16, a) CBR-curves of tertiary clay



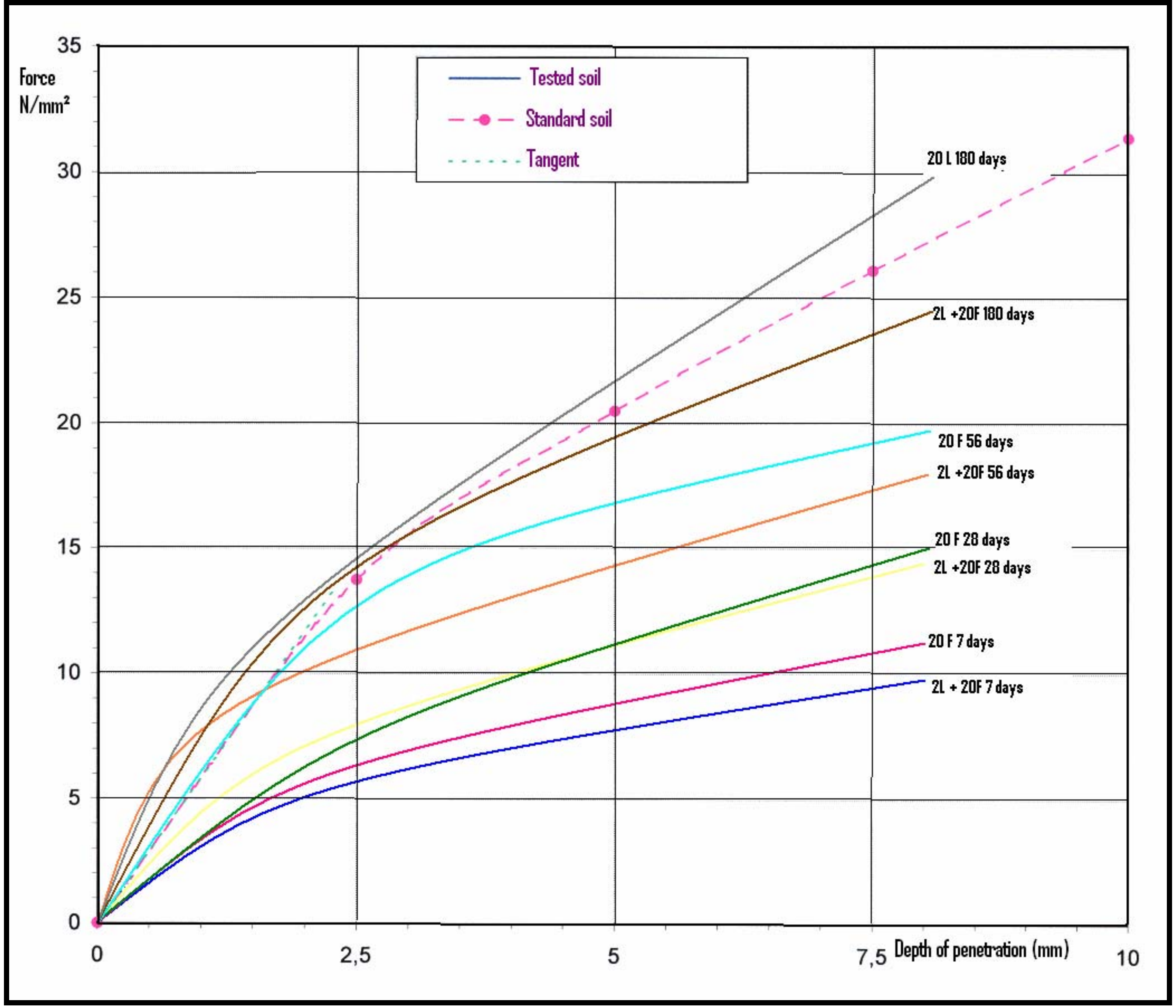
Appendix (16, b) follow: CBR-curves of tertiary clay



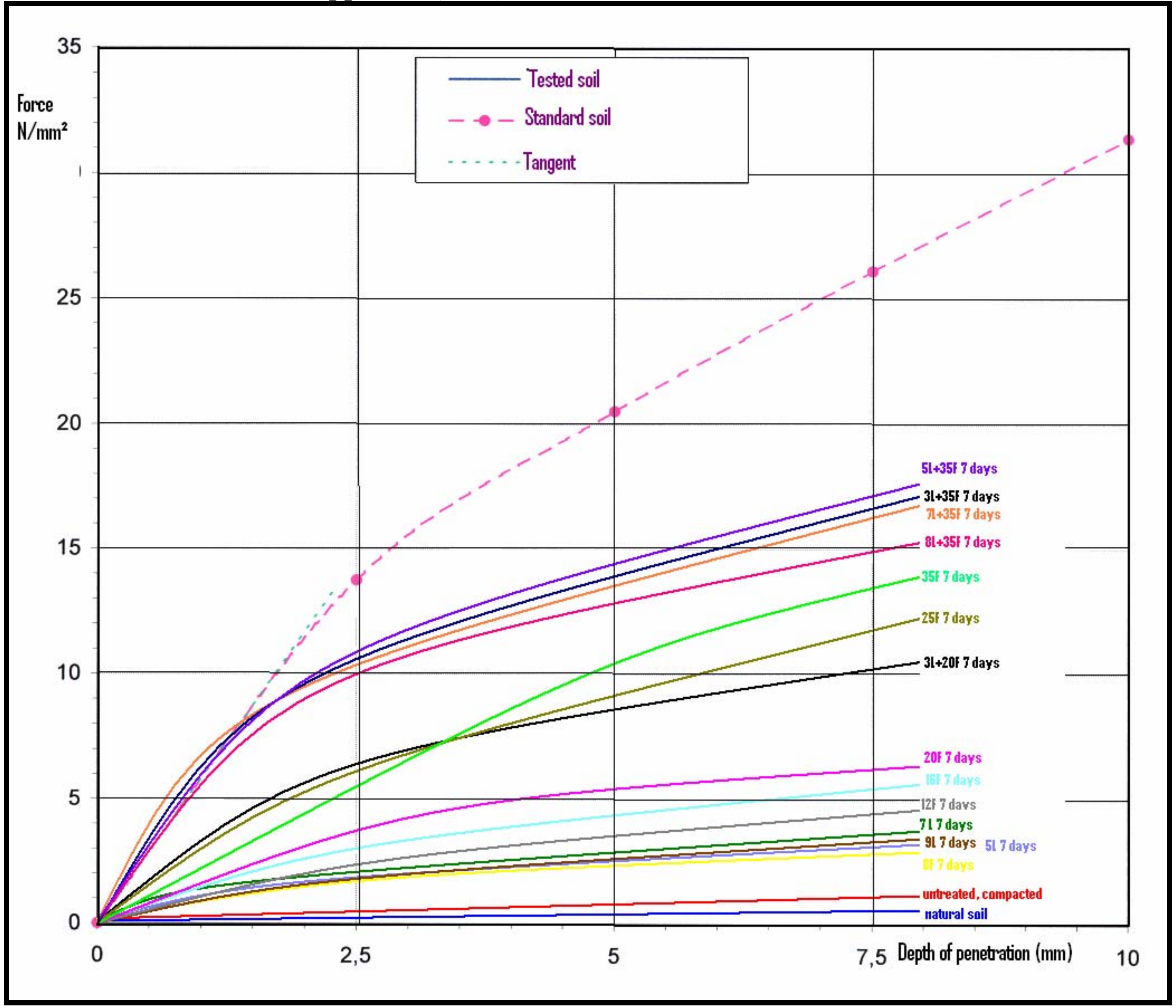
Appendix (17, a) CBR-curves of organic silt



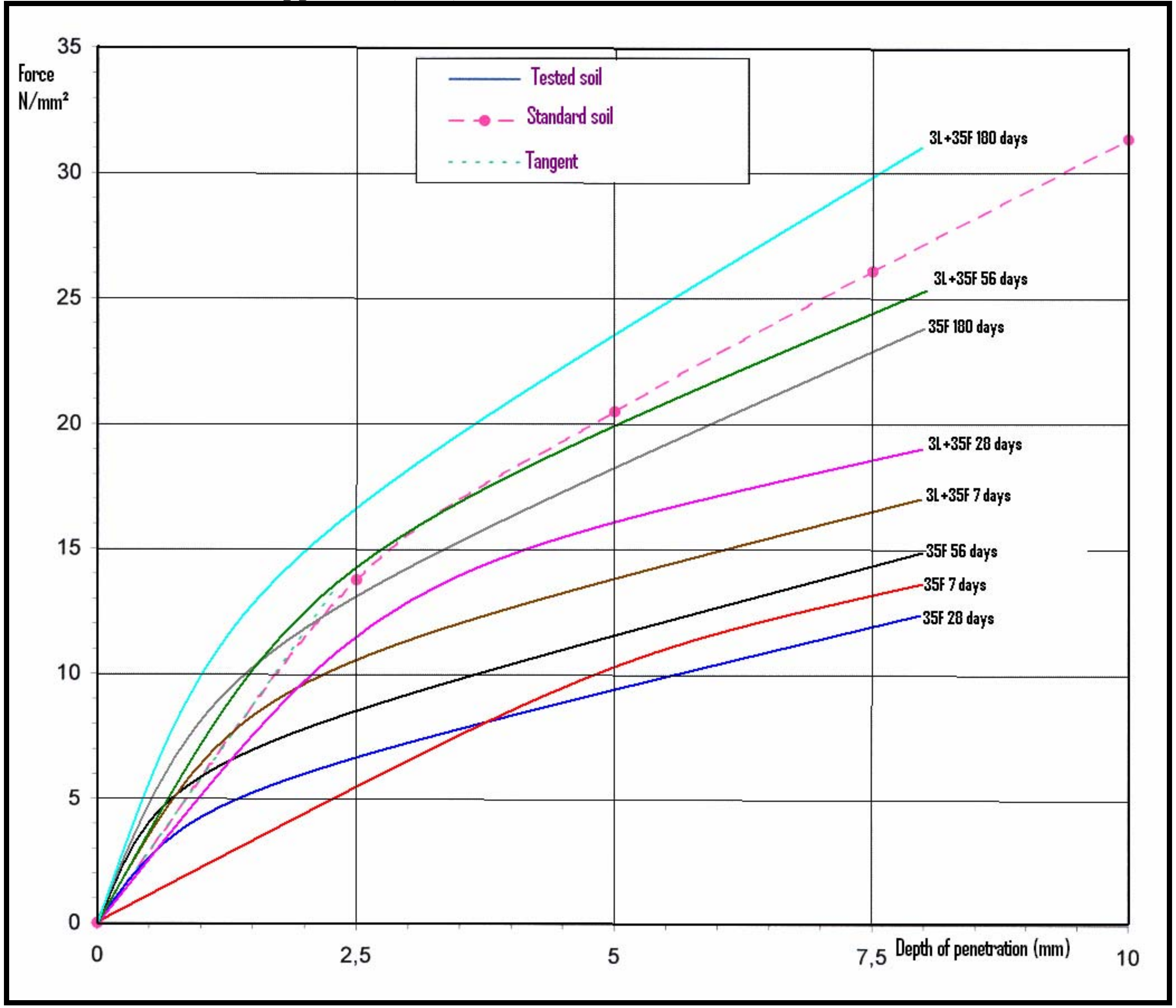
Appendix (17, b) follow: CBR-curves of organic silt



Appendix (18, a) CBR-curves of weathered soil



Appendix (18, b) follow: CBR-curves of weathered soil



Appendix (19) Calculation of tensile strength of tertiary clay

Admixture mixing		Curing time	L	D	P	σt = 2P/π.D.L = 0.636P/D.L	σt
L (%)	FA (%)	Days	(mm)	(mm)	(N)	(MPa)	(KN/m^2)
4.5*	0	7	100	100	1717.8	0.10925	109.25
6.5	0	7	100	100	2519.6	0.16025	160.25
8.5	0	7	100	100	2886.8	0.18360	183.60
0	8	7	100	100	687.1	0.4370	43.70
0	12	7	100	100	1146.2	0.7290	72.90
0	16*	7	100	100	1603.8	0.10200	102.00
		28	100	100	2921.4	0.18580	185.80
		56	100	100	3321.7	0.21126	211.26
		180	100	100	4352.5	0.27682	276.82
0	20	7	100	100	2405.7	0.15300	153.00
0	25	7	100	100	2404.9	0.15295	152.95
2.5*	8*	7	100	100	1947.3	0.12385	123.85
1.5	16	7	100	100	2061.1	0.13115	131.15
2.5	16	7	100	100	2233.6	0.14206	142.06
		28	100	100	3779.9	0.2404	240.4
		56	100	100	5268.9	0.33510	335.10
		180	100	100	6299.7	0.40066	400.66
4.5	16	7	100	100	2176.3	0.13841	138.41
6.5	16	7	100	100	2000.6	0.12749	127.49

L = Thickness of the tested specimen (mm)
D = Diameter of the tested specimen (mm)
P = Load at failure (N)
σt = Tensile strength (MPa)

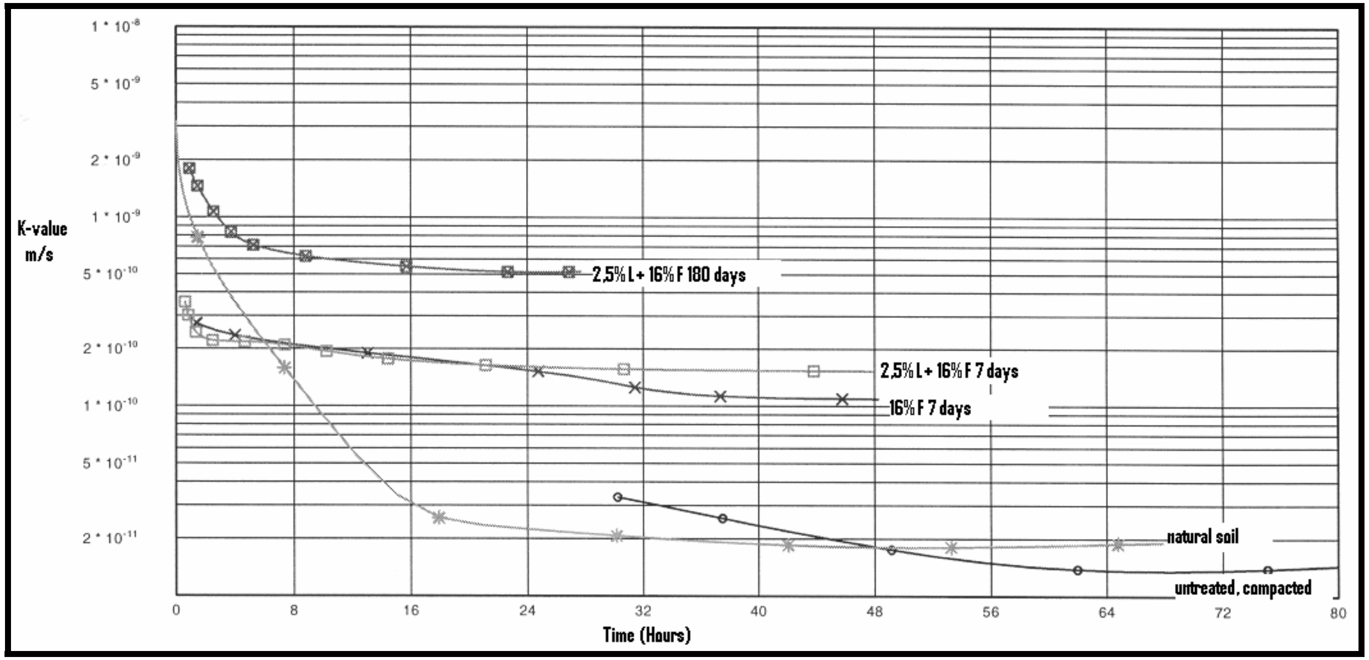
Appendix (20) Calculation of tensile strength of organic silt

Admixture mixing		Curing time	L	D	P	σt = 2P/π.D.L = 0.636P/D.L	σt
L (%)	FA (%)	Days	(mm)	(mm)	(N)	(MPa)	(KN/m²)
3*	0	7	100	100	1374.2	0.08740	87.40
5	0	7	100	100	1718.6	0.10930	109.30
7	0	7	100	100	1603.0	0.10195	101.95
0	8	7	100	100	595.9	0.03790	37.90
0	12	7	100	100	916.7	0.05830	58.30
0	16	7	100	100	1145.4	0.07285	72.85
0	20*	7	100	100	1202.7	0.07649	76.49
		28	100	100	2405.7	0.15300	153.00
		56	100	100	2634.4	0.16755	167.55
		180	100	100	4009.0	0.25497	254.97
0	25	7	100	100	1718.1	0.10927	109.27
2*	12 *	7	100	100	1030.7	0.06555	65.55
2	20	7	100	100	1259.9	0.08013	80.13
		28	100	100	2405.3	0.15298	152.98
		56	100	100	2978.1	0.18941	189.41
		180	100	100	4066.2	0.25861	258.61
3	20	7	100	100	1317.3	0.08378	83.78
5	20	7	100	100	1146.2	0.07290	72.90

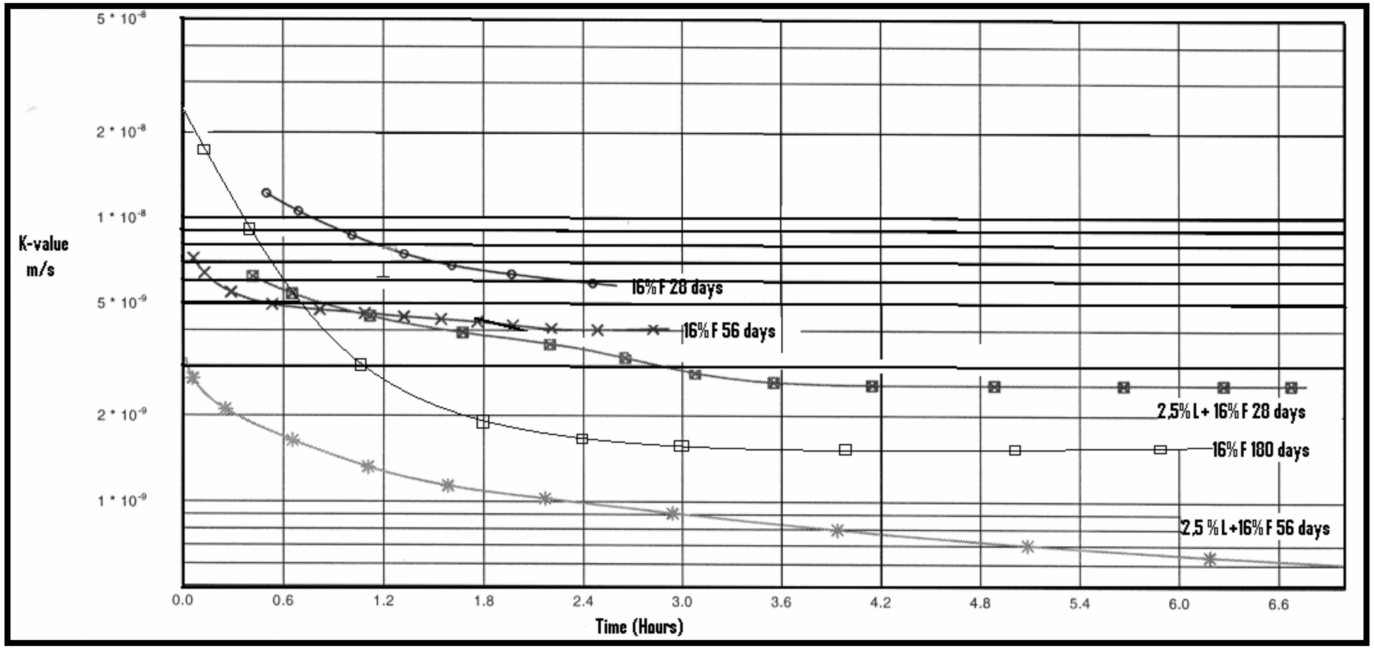
Appendix (21) Calculation of tensile strength of weathered soil

Admixture mixing		Curing time	L	D	P	$\sigma t = 2P/\pi.D.L = 0.636P/D.L$	σt
L (%)	FA (%)	Days	(mm)	(mm)	(N)	(MPa)	(KN/m^2)
5*	0	7	100	100	744.5	0.04735	47.35
7	0	7	100	100	801.1	0.05095	50.95
9	0	7	100	100	629.7	0.04005	40.05
0	8	7	100	100	572.3	0.03640	36.40
0	12	7	100	100	629.6	0.04004	40.04
0	16	7	100	100	801.1	0.05095	50.95
0	20	7	100	100	1088.1	0.06920	69.20
0	25	7	100	100	1146.2	0.07290	72.90
0	35*	7	100	100	1741.4	0.11075	110.75
		28	100	100	2176.3	0.13841	138.41
		56	100	100	2405.3	0.15298	152.98
		180	100	100	2863.5	0.18212	182.12
3*	20 *	7	100	100	1718.6	0.10930	109.30
3	35	7	100	100	1832.7	0.11656	116.56
		28	100	100	2290.9	0.14570	145.70
		56	100	100	2520.0	0.16027	160.27
		180	100	100	3723.3	0.23680	236.80
5	35	7	100	100	2863.2	0.18210	182.10
7	35	7	100	100	2748.4	0.17480	174.80
8	35	7	100	100	2348.1	0.14934	149.34

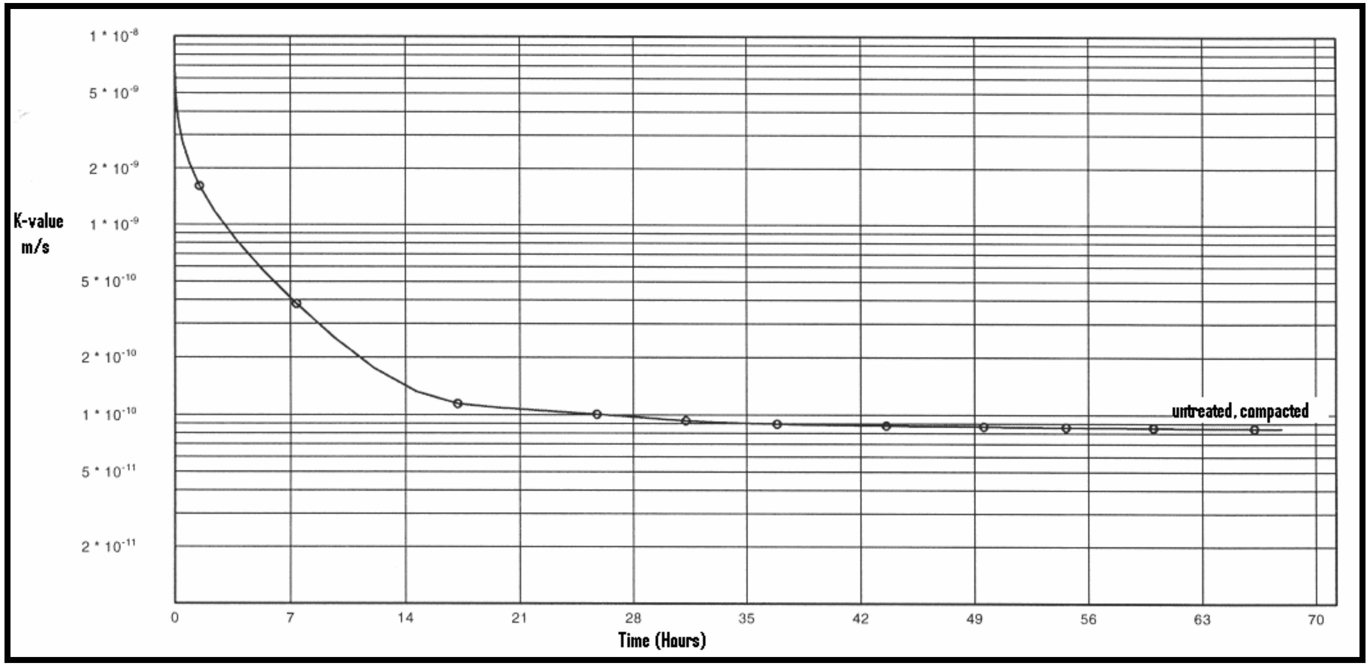
Appendix (22, a) K-value curves of tertiary clay.



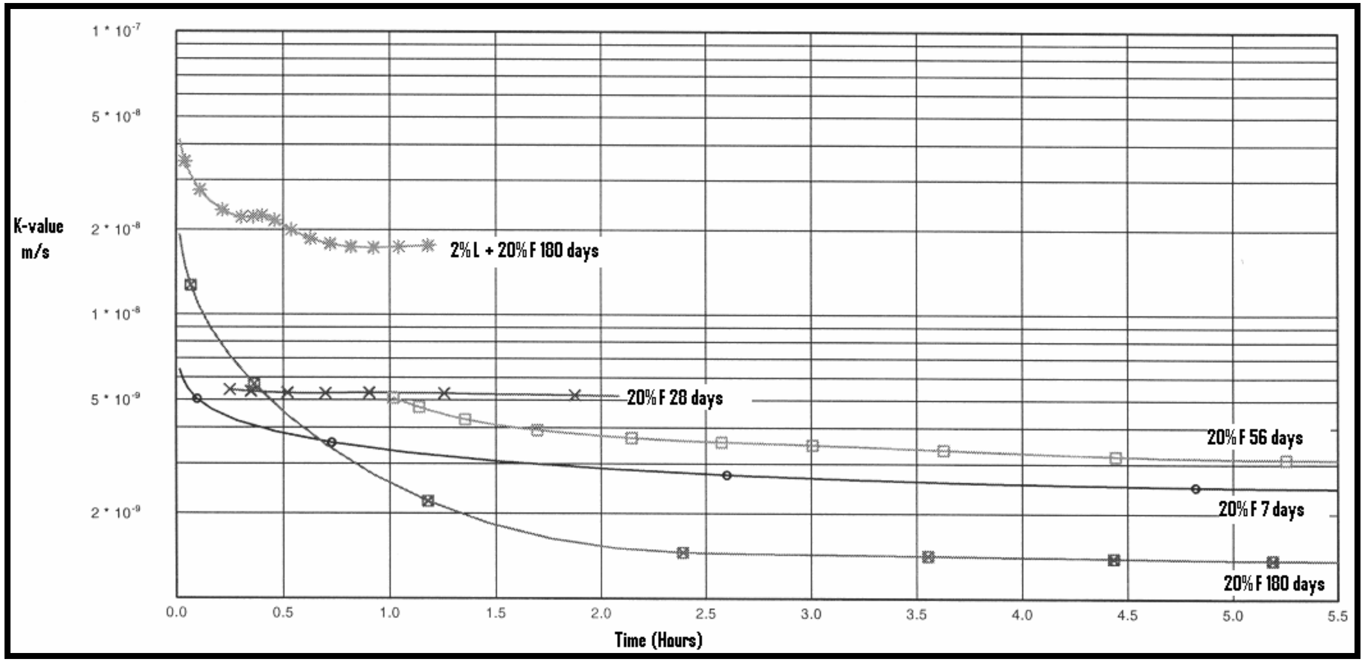
Appendix (22, b) follow: K-value curves of tertiary clay.



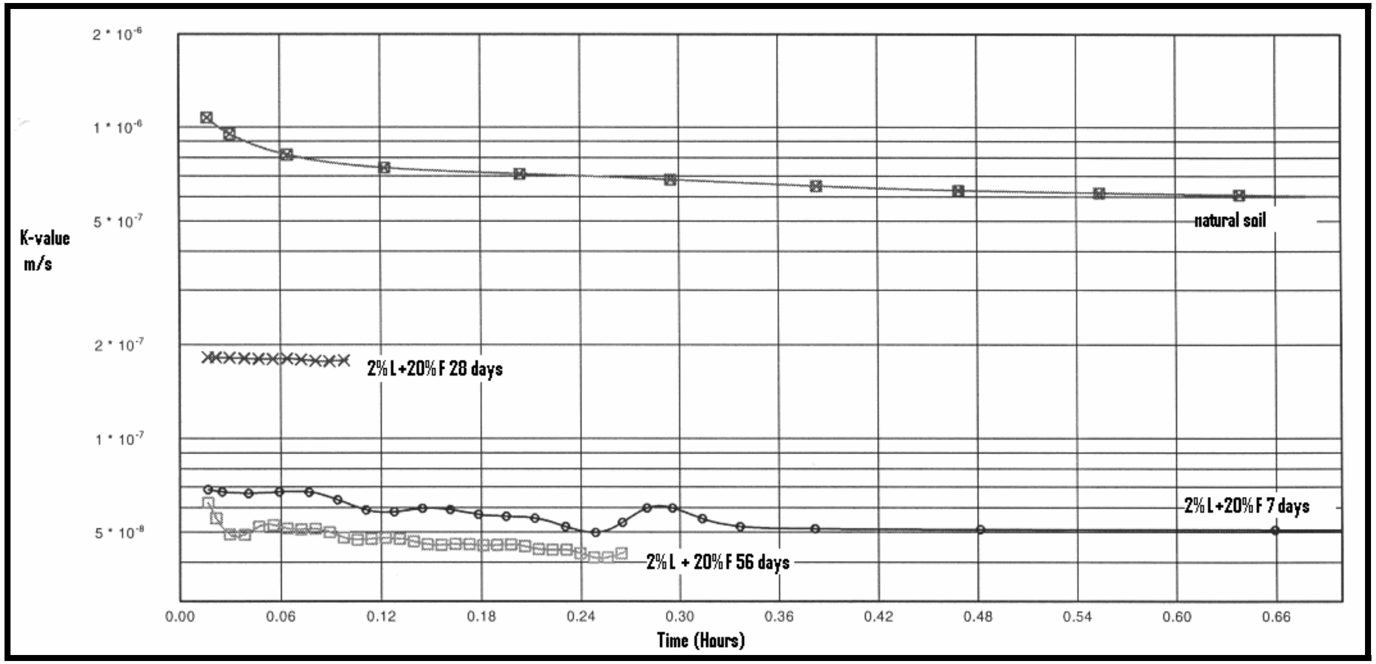
Appendix (23, a) K-value curves of organic silt.



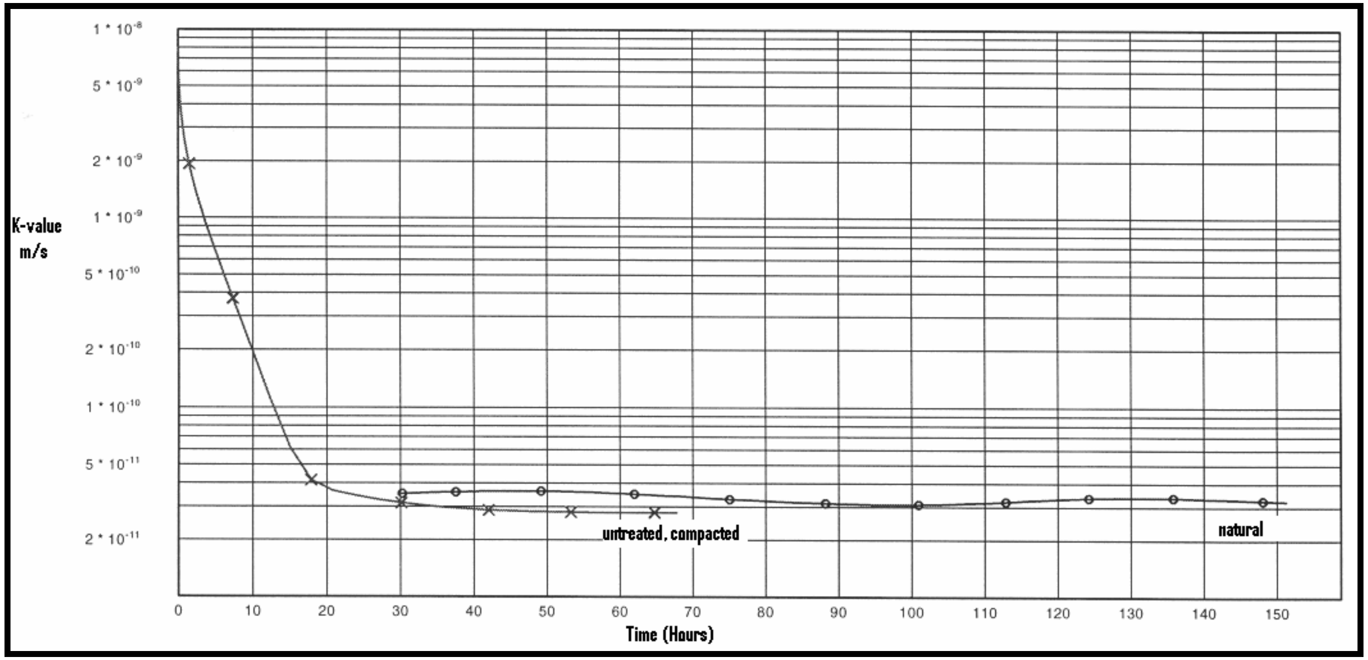
Appendix (23, b) follow: K-value curves of organic silt.



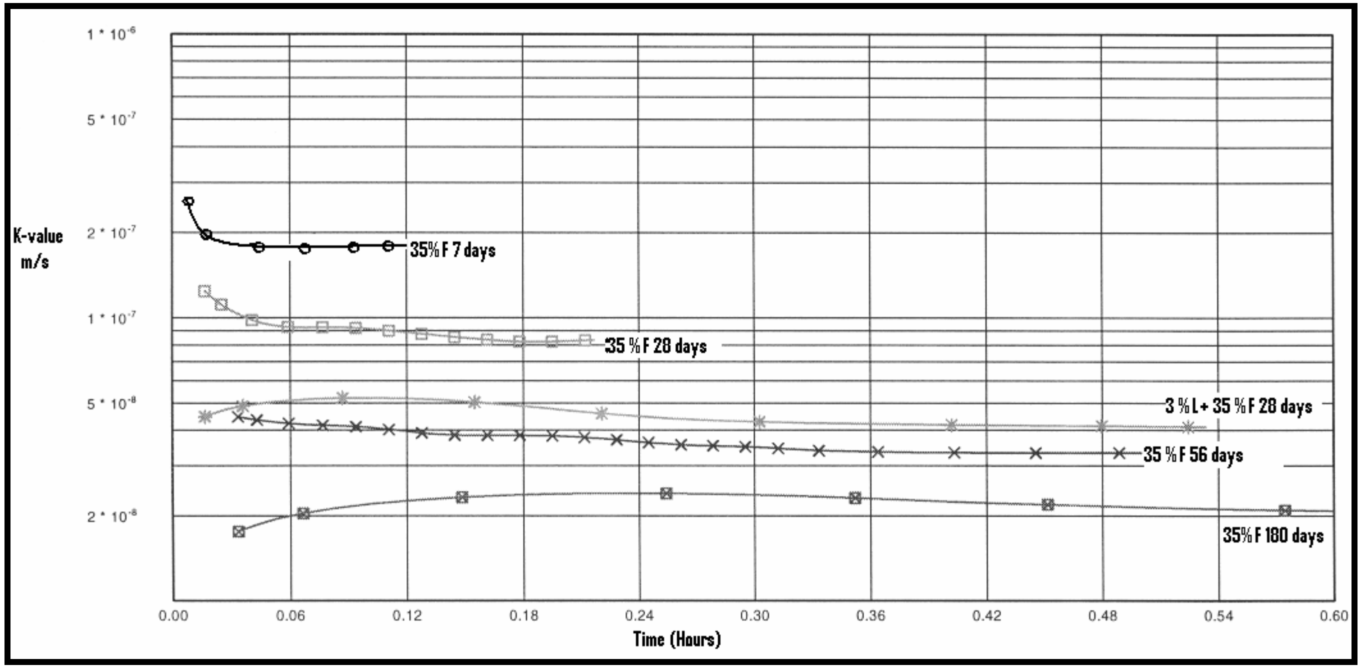
Appendix (23, c) follow: K-value curves of organic silt.



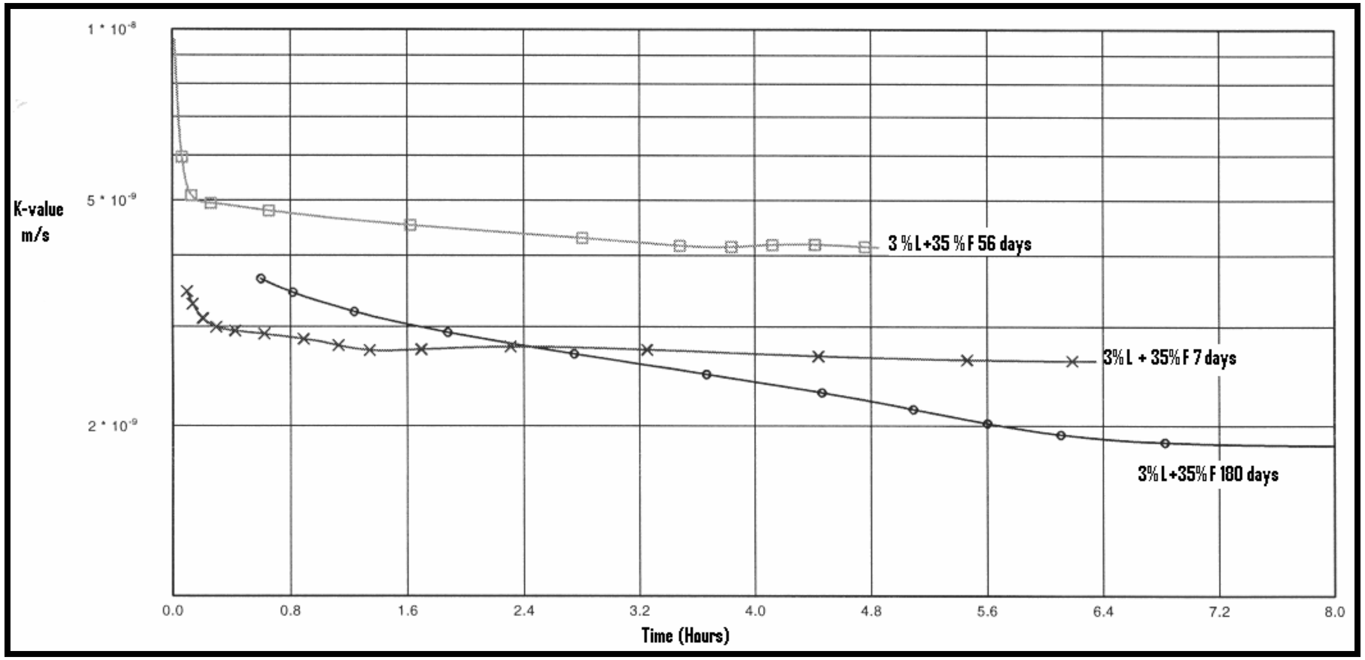
Appendix (24, a) K-value curves of weathered soil



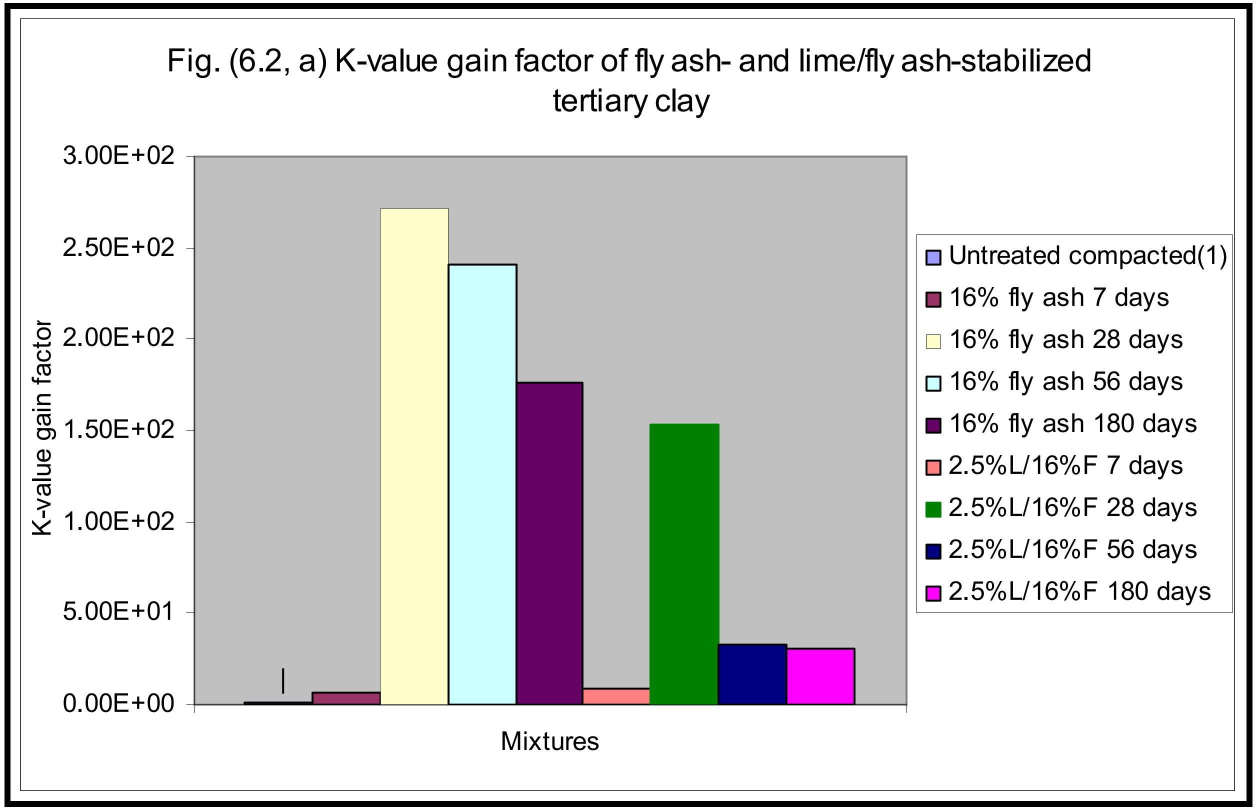
Appendix (24, b) follow: K-value curves of weathered soil.



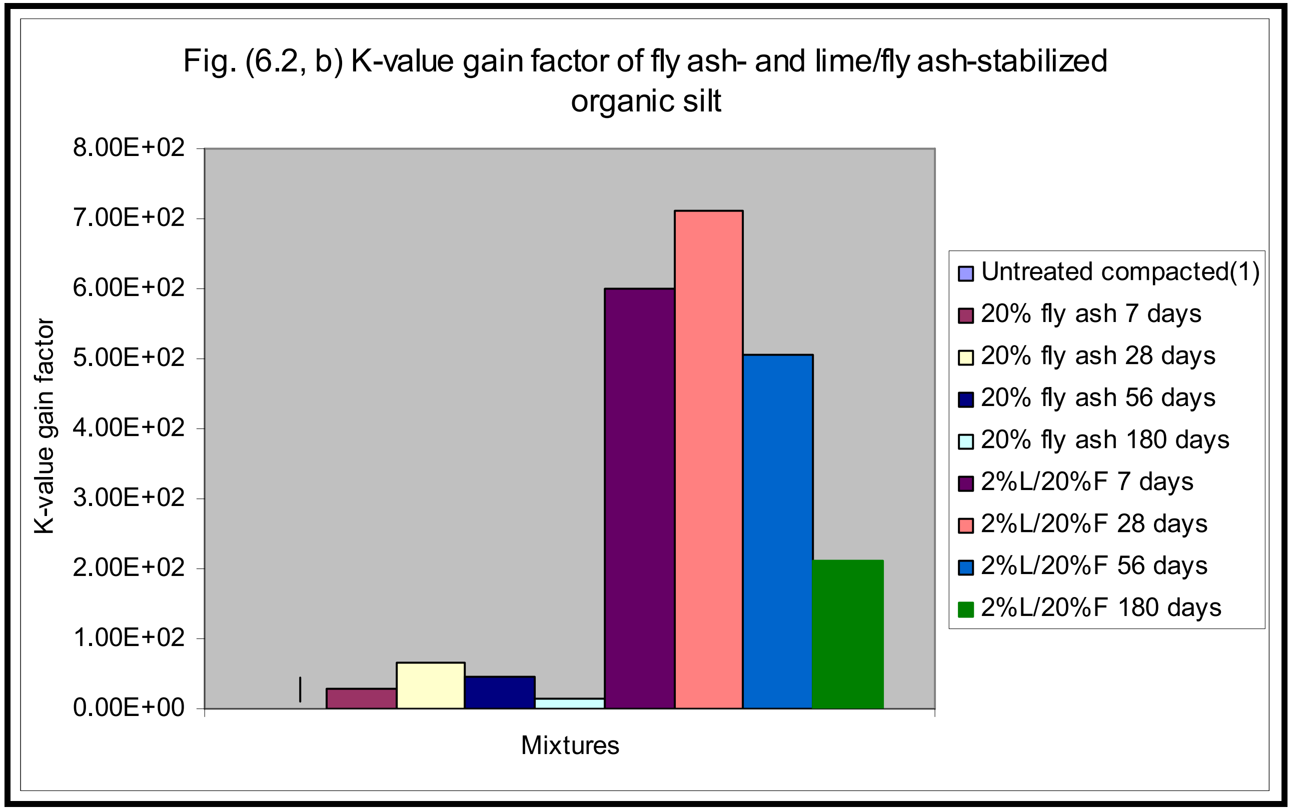
Appendix (24, c) follow: K-value curves of weathered soil.



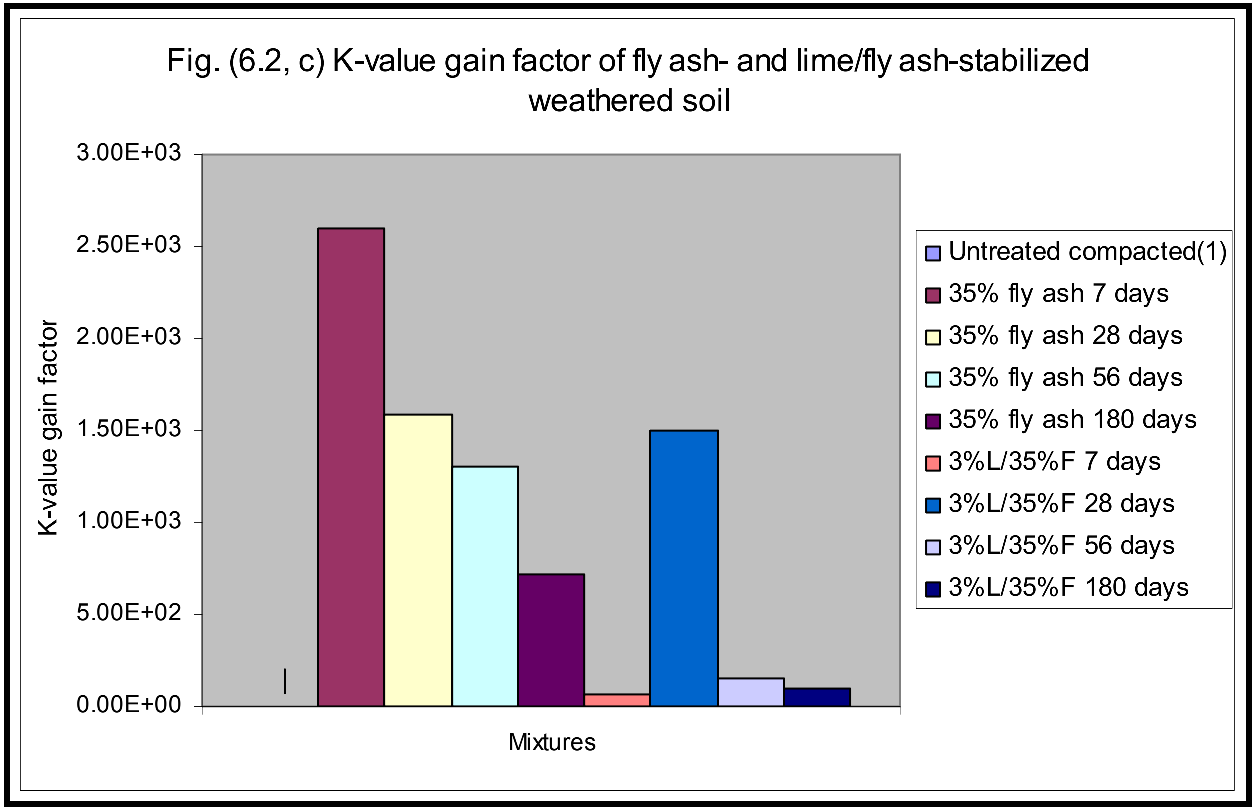
Appendix (25) K-value gain factors of stabilized tertiary clay.



Appendix (26) K-value gain factors of stabilized organic silt.



Appendix (27) K-value gain factors of stabilized weathered soil.



Appendix (28) Calculation of Vp-value for tertiary clay

Admixture mixing		Curing time	X	Average of ta	Vp-value
L (%)	**FA (%)**	**Days**	**(m)**	**(µs)**	**(m/sec)**
0	0	Natural soil	0.12	186.6	643
0	0	Untreated compacted	0.12	179.9	667
4.5*	0	7	0.12	84.6	1418
6.5	0	7	0.12	76.5	1568
8.5	0	7	0.12	75.0	1600
0	8	7	0.12	115.5	1039
0	12	7	0.12	98.8	1215
0	16*	7	0.12	91.4	1309
		28	0.12	79.3	1513
		56	0.12	78.1	1536
		180	0.12	77.2	1600
0	20	7	0.12	78.1	1537
0	25	7	0.12	80.8	1485
2.5*	8*	7	0.12	91.3	1314
1.5	16	7	0.12	80.5	1491
2.5	16	7	0.12	80.2	1496
		28	0.12	72.5	1655
		56	0.12	67.2	1786
		180	0.12	64.0	1875
4.5	16	7	0.12	83.6	1435
6.5	16	7	0.12	87.5	1371

X = Height (thickness) of the specimen (m)
ta = Average (4-Measurments) of arrival (travel)-time (µs)
Vp = Velocity of ultrasonic p-wave (m/sec)

Appendix (29) Calculation of Vp-value for organic silt

Admixture mixing		Curing time	X	Average of ta	Vp-value
L (%)	FA (%)	Days	(m)	(μs)	(m/sec)
0	0	Natural soil	0.12	283.0	424
0	0	Untreated compacted	0.12	258.2	465
3*	0	7	0.12	140.8	852
5	0	7	0.12	124.9	961
7	0	7	0.12	130.8	917
0	8	7	0.12	153.6	781
0	12	7	0.12	137.0	876
0	16	7	0.12	125.9	953
0	20*	7	0.12	118.7	1011
		28	0.12	97.7	1228
		56	0.12	92.5	1297
		180	0.12	84.5	1420
0	25	7	0.12	109.3	1098
2*	12*	7	0.12	129.2	929
2	20	7	0.12	121.1	991
		28	0.12	101.9	1178
		56	0.12	89.4	1342
		180	0.12	83.7	1434
3	20	7	0.12	118.5	1013
5	20	7	0.12	126.5	949

Appendix (30) Calculation of Vp-value for weathered soil

Admixture mixing		Curing time	X	Average of ta	Vp-value
L (%)	FA (%)	Days	(m)	(μs)	(m/sec)
0	0	Natural soil	0.12	171.4	700
0	0	Untreated compacted	0.12	166.4	721
5*	0	7	0.12	136.0	882
7	0	7	0.12	116.0	1034
9	0	7	0.12	120.0	1000
0	8	7	0.12	122.6	979
0	12	7	0.12	117.9	1018
0	16	7	0.12	117.6	1020
0	20	7	0.12	117.4	1021
0	25	7	0.12	104.6	1147
0	35*	7	0.12	97.1	1236
		28	0.12	87.8	1367
		56	0.12	87.5	1371
		180	0.12	83.4	1439
3*	20*	7	0.12	92.5	1297
3	35	7	0.12	87.7	1368
		28	0.12	85.6	1402
		56	0.12	82.6	1453
		180	0.12	74.0	1622
5	35	7	0.12	86.2	1392
7	35	7	0.12	86.8	1382
8	35	7	0.12	87.6	1370

Curriculum vitae

December 12, 1969	Born at Qena city, **Egypt**.
1987	General **Secondary Education** (Science Section) Certificate from Abu Bakr Al-sedik Secondary School, Qena city, Egypt, (very good)
1991	**Degree of Science-Bachelor in Geology** (very good) from Qena Faculty of Science, Assiut University, Egypt.
1992- 1996	Work as **Demonstrator** at Geology Department, South Valley University, Egypt.
1994	**Postgraduate Courses** for the Master Degree in Geology (Distinct), Qena, Egypt.
1996	Compacted Courses for Preparation of the **University-Teacher**, at Qena Educational Faculty, South Valley University, Egypt.
1997	**Master Degree in Geology**, Qena, Egypt.
1997- 2001	Work as **Assistance Lecture** in Geology Department, at Qena Faculty of Science, South Valley University, Egypt.
2001	**Egyptian scholarship** (4-5 years) for obtaining doctor degree in Engineering Geology, at Martin Luther University Halle-Wittenberg, Germany.
2001- 6.2002	**Developing Study** “Aufbau Studium” 1.5 years including Success in the Study-Stages of the **German Diploma** in the field of **Engineering Geology/Geotechnology** (Courses in Engineering Geology I & II, Soil and Rock Mechanics, Applications about the methods of geotechnical calculations, the Courses include “Lectures/Exercises/Practises in the field and in the Laboratory/**GGU-software-programs**”, two compacted Courses in the **Finite Elements Analysis** “Lectures/Exercises/Software-programs”, and finally, Seminar with project in Engineering Geology and Geotechnology).
6.2002- 2003	**Training** in the Laboratory of the **Soil Mechanics** on the modern generations of the computerized-instruments.
2003- Present	Pursuing **doctor degree in Engineering Geology**, Institute of Geology, Martin Luther University Halle-Wittenberg, Germany.
Marital status	**Single**.

Halle (Saale), 8.02.2006

Hesham Ahmed Hussin Ismaiel

Announcement

Herewith, I affirm that, I have written this dissertation independently. This one was not taken from strange sources directly or indirectly. I have characterized the thoughts as such. This dissertation was not submitted at any other university. I have never applied for a doctorate before.

Halle (Saale), 8.02.2006 *Hesham Ahmed Hussin Ismaiel*