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SHEAR STRENGTH IMPROVEMENT AT THE INTERFACE OF LIME-TREATED SOIL AND CONCRETE STRUCTURES

by

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ABSTRACT

Lime stabilization can be used to increase the shaft resistance capacity of friction piles in clayey-soil. The concentration of the shear stress decreases gradually from the pile shaft outwards. By means of lime stabilization, the shear resistance can be increased in the zone close to the pile shaft where failure is most probable to occur. When the pile type is selected as cast-in-situ concrete pile, the free lime in fresh concrete may be considered to be the lime source for stabilization. However, the lime content in fresh concrete does not exceed one to two per cent. To provide extra lime, a slurry can be prepared in the bore-hole prior to concrete casting. Thus, the lime stabilization may occur through diffusion.

In this study, direct shear tests were performed to observe the increase in shear strength at the interface of lime stabilized clayey-soil and concrete. By the lime stabilization of the clayey-soil, an increase of a great extent up to 100 per cent is observed in shear strength at the soil-concrete interface.

The method proposed for increasing the shaft resistance capacity of cast-in-situ concrete piles by the lime stabilization is very easy and convenient for the field applications. Therefore, the necessary load carrying capacity of piles can be obtained economically without increasing the pile dimensions.

Keywords: Cast-in-situ concrete piles, skin friction, lime stabilization, diffusion.

ÖZET

Kireç stabilizasyonu killi zeminlerde kazıkların yanal taşıma kapasitelerini arttırmak için kullanılabilir. Kesme kuvvetinin şiddeti kazık yanal yüzeyinden uzaklaştıkça azalır. Kireç stabilizasyonu ile, çökmenin en olası olduğu kazık yanal yüzeyinin hemen yakınındaki bölgenin kesme dayanımı arttırılabilir. Kazık tipi yerinde dökme beton kazık olarak seçildiğinde, taze betonun içindeki serbest kirecin, stabilizasyon işlemi için kaynak olabileceği düşünülebilir. Fakat taze betonun içinde serbest halde bulunan kireç oranı yüzde bir ya da ikiyi geçmez. Daha fazla kireç sağlamak için, beton dökülmesinden önce, kazık çukurunda kireç çamuru oluşturulabilir. Bu sayede, kireç stabilizasyonu difüzyon yolu ile gerçekleşebilir.

Bu çalışmada, kireçle stabilize edilmiş killi zemin ile beton arasındaki yüzeyde kesme dayanımı artışını görebilmek amacı ile kesme deneyleri yapılmıştır. Deney sonuçlarına göre, killi toprakların kireç stabilizasyonu ile kesme dayanımında yüzde 100' e varan büyük artışlar olduğu saptanmıştır.

Kazıkların yanal taşıma kapasitelerinin arttırılması için önerilen metod çok kolay ve saha uygulamaları için oldukça uygundur. Bu metodla, kazıkların gerekli yük taşıma kapasiteleri, kazık boyutlarını arttırmaksızın, ekonomik şekilde elde edilebilir.

Önemli Sözcükler: yerinde dökme beton kazıklar, yanal sürtünme, kireç stabilizasyonu, difüzyon.

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A	a mineralogical parameter including mineralogy, specific surface, grain size and cation exchange capacity
A_p	the area of the pile tip
A_w	the initial pore-water parameter
a	empirical constant related to testing conditions
C	cement content by weight in 1 m^3 of concrete
c	the concentration of the additive (in **Equation 2.1**)
c	cohesion of soil
D_o	the "free solution" diffusion coefficient
f_s	skin resistance of piles
J	the mass flux
K	empirical constant related to cement quality (in **Equation 3.2**)
K	coefficient of lateral earth pressure
m	fineness modulus of the aggregate mixture
N	statistical average SPT blow count
N^*_c	bearing capacity factor at the pile tip related to cohesion
N^*_q	bearing capacity factor at the pile tip related to surcharge
n	the total soil porosity
Q_p	load carried at the pile point
Q_s	load carried by skin friction
Q_u	ultimate pile load

q	the sorbed concentration of solute per mass of soil
q_c	cone-penetration resistance
q'	the sorbed concentration of the chemical species (in **Equation 2.3**)
q'	the effective vertical stress
$\bar{q}$	effective vertical stress on element ΔL of the soil stratum
R	compressive strength of concrete specimens at 28 days of age
S_r	the degree of saturation of soil
S_u	the undrained shear strength of soil
SF_s	the safety factor of shaft resistance of piles
SF_p	the safety factor of point resistance of piles
t	the time for curing
t_a	the time of mellowing (the time between mixing and compaction of soil)
W	water content by weight in 1 m^3 of concrete
W_0	the molding water content
x	the direction of transport
α	fluidity factor (in **Equation 2.3**)
α	empirical adhesion factor (in **Equation 2.15**)
α	coeff. depending on consistency and type of aggregates (in **Equation 3.1**)
γ	anion exclusion factor (in **Equation 2.3**)
γ'	effective unit weight of soil
δ	angle of friction between soil and pile
δ_d	the dry density of soil

ϕ	angle of internal friction
θ	volumetric water content
σ	normal stress
τ	tortuosity factor (in **Equation 2.3**)
τ	shear stress

USCS	Unified Soil Classification System
AASHTO	American Association of State Highway Transportation Officials
ASTM	American Society for Testing and Materials

1. INTRODUCTION

If the subsoil consists of clay too weak or too compressible to support footings or raft, piles or piers are used to transfer the loads to a firm stratum by end bearing or to the surrounding soil by skin friction. When the firm stratum is not close to the surface, skin friction is the dominant mechanism for the carrying capacity. Since skin resistance is a function of shear strength parameters of the surrounding soil, lime stabilization can be applied to increase these parameters.

Lime stabilization is widely used to improve the engineering properties of clayey-soil. In most of the applications, lime is mixed with soil prior to compaction. However, for the lime stabilization of the clayey-soil around the pile shaft, diffusion mechanism can be utilized. When cast-in-situ concrete piles are used, the free lime in fresh concrete can be considered the lime source for this stabilization. Nevertheless, the free lime content in fresh concrete does not exceed 1-2 per cent [1]. For a more effective stabilization, more lime is necessary. For this purpose, a slurry of lime can be prepared in the bore-hole and left for a period that is sufficient for lime to diffuse through the shear zone close to the pile shaft.

In this study, the effect of the lime stabilization on the clayey-soil and concrete interaction was determined. For this determination, direct shear tests were used. The interaction between the lime stabilized soil and concrete was compared to the interaction between unstabilized soil and concrete. The results have shown that the shear strength increases markedly by the lime stabilization of clayey-soil through diffusion at the concrete interface.

In the literature, no work related to the attempt of applying pile or pier structures together with the treatment of surrounding soil with lime is faced. The mechanism of lime treatment of soil, the diffusion properties of lime into soil and the skin resistance properties at the interface of soil and pile are presented in the succeeding section.

2. LITERATURE REVIEW

2.1. Lime Stabilization

Since World War II much attention has been directed to the use of stabilizing agents to improve the engineering properties of cohesive soils. Lime, one of the many chemicals tried, has proven to be an effective and an economical additive for improving properties of clayey soils. On the other hand, the use of lime as a stabilizing agent has remained limited mainly to road constructions. However, because of its binding property and reactivity, it may be considered to be used for the stabilization of soils which are in contact with concrete structures, such as cast-in-situ concrete piles or drilled piers, specially in cases where skin friction are important for the carrying capacity.

2.1.1. Theory and Mechanism

The beneficial effects of lime are generally attributed to the interaction of lime and the clay minerals in the soil. As it is known clay minerals are very tiny crystalline materials formed primarily from chemical weathering of certain rock forming minerals. Chemically, clay minerals are hydrous aluminosilicates plus other metallic ions [2]. All clay minerals are minute, colloidal-sized crystals, and they can only be seen with an electron microscope. The distinct crystals look like tiny plates that consist of a number of crystal sheets having repeated atomic structure. The two fundamental sheets are *tetrahedral* or *silica*, and *octahedral* or *alumina* sheets. The way which these sheets are stacked together and the different bonding and different metallic ions in the crystal lattice, constitute the different clay minerals.

Lime reacts with clays in four ways:

(a) *cation exchange* (Diamond and Kinter 1965 [3]) - Cations like Na^+, K^+, Ca^{++}, Mg^{++}, Al^{+++} has an order of replacebility generally from monovalent to the multivalent cations. With the addition of lime, excess Ca^{++} ions are provided to the soil and Ca^{++} will replace dissimilar cations from the soil complex. However as many soil scientists agree, natural soils are largely calcium saturated. Therefore, the factor of cation exchange has not been regarded as a very significant effect of lime on clayey soils.

(b) *flocculation and particle aggregation* (Herzog and Mitchell 1963 [4]) - The addition of lime to a fine grained-soil causes flocculation and agglomeration of clay fraction. These reactions result in an apparent change in texture because of the clay particles joining together into larger sized aggregates.

(c) *lime carbonation* (Eades *et al.* 1962 [5]; Le Roux 1969 [6]; Pretty and Rich 1971 [7]) - Lime reacts with carbon dioxide from the atmosphere to form relatively weak cementing agents, calcium carbonate, depending on the type of lime used. Nevertheless, since the long-term reactions of uncarbonated lime with soil itself would far exceed the contribution of calcium carbonate, carbonation is said to be a deleterious rather than a remedial phenomenon in soil stabilization [3].

(d) *pozzolanic reactions between lime, silica and alumina* - The pozzolanic reactions occur between soil, silica and/or alumina and lime to form various types of cementing agents. These cementing agents are generally regarded as the major source of strength increases noted in lime-soil mixtures.

Reactions (a) and (b) lead to immediate improvement in soil plasticity, workability, uncured strength, and load deformation properties (US Transportation Research Board 1987 [8]). Reactions (c) and (d) lead to the formation of cementing products (Eades and Grim 1960 [9]; Hilt and Davidson 1960 [10]; Glen and Handy 1963 [11]; Diamond *et al.* 1964 [12]; Sloane 1965 [13]; Ormsby and Kinter 1973 [14]) . Pozzolanic reactions give rise to long-term increase in soil strength causing little or no change in water content, even many months after mixing and compaction [15].

In order to evaluate the efficiency of lime stabilization and measure the gain in shear strength at a given time, Regina [16] has derived the formula:

$$S_u = f(\, S_{u0},\, A,\, A_w,\, W_0,\, c,\, t_a,\, t\,) \tag{2.1}$$

where S_u (kPa) is the undrained shear strength immediately after compaction; A is a mineralogical parameter including mineralogy, specific surface, grain size and cation exchange capacity; A_w is an initial pore-water parameter; W_0 (%) is the molding water content; c (%) is the concentration of the additive; t_a and t (days) are the time of mellowing and curing, respectively. The mellowing time is the time between mixing and compaction and curing time is the time since compaction. It must also be realized that the nature of the soil is not constant because the chemical attack on minerals results in changes on parameters such as mineralogy and grain size distribution [15]. This precedes to modifications of index properties such as liquid and plastic limits. Regarding these considerations, it is explicit why only empirical approaches are known in the field of soil stabilization.

Locat *at al.* [15] has shown by the laboratory investigations that the long term strength obtained, for a given soil, is directly relevant to molding water content and quicklime concentration. They found that the undrained shear strength S_u increases with increasing quicklime concentration and decreasing molding water content of the soil. Choquette (1988) [17] has shown that the initial controlling reaction parameters are grain size and specific surface area. However, with the development of pozzolanic reactions, the mineralogy becomes the only parameter that is positively related to strength development [15].

It has been debated that during the pozzolanic reactions lime in solution is used up and more must be dissolved so as to prevent the solution equilibrium. Therefore any increase in quicklime concentration is favorable for the strength development even though 0.1% lime is sufficient to saturate pore-water solution [15]. Moreover, besides lime content, the dispersion of solid lime in excess seems to have a strong effect on stabilization. Tests reported by Bèrubè and Locat in 1987 [18] emphasized that mixing lime with more energy produces higher strength at equivalent lime concentration and time, being more evident for higher plasticity soils.

A schematic model describing the physicochemical process of lime stabilization which was first presented by Ingles and Metcalf in 1973 [19] and later modified by Choquette in 1988 [17] adding results for high water content soils is illustrated in **Figure 2.1** . As can be seen in this model, reaction products diffuse within the soil particles and create bridges or cover between or on soil particles. This cementitious process results in an increase in strength acting primarily on the cohesion factor of the shear strength parameters of the soil [15]. It can be inferred from these scheme that a high water content soil may show a better performance than a soil with low water content because of the easy movement of solutes in porous space.

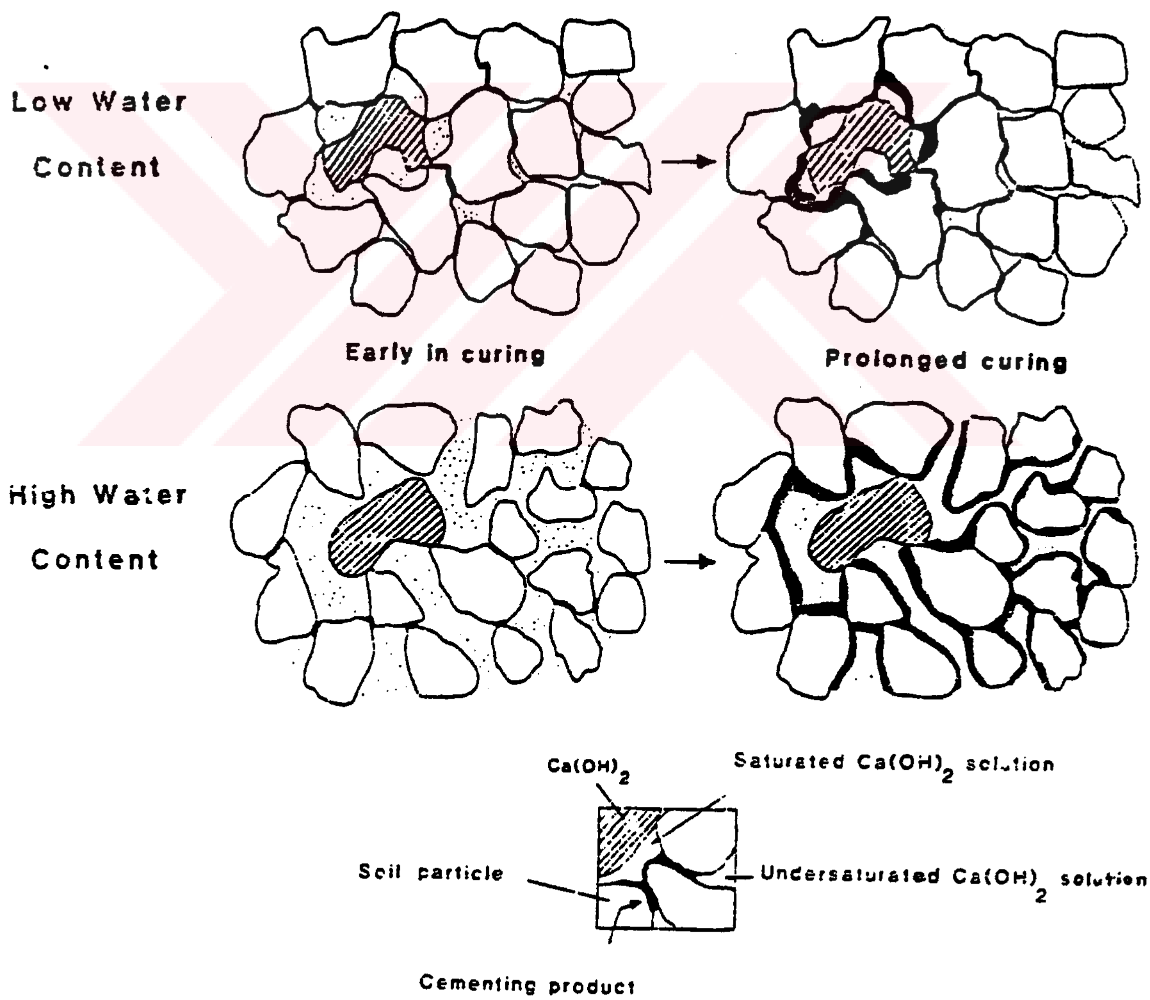


FIGURE 2.1. Physical conceptual model proposed for lime stabilization of sensitive clays (after Ingles and Metcalf 1973 [19], from Locat *et al.* [15])

Considering the schematic model illustrated in **Figure 2.1** and making use of results from their laboratory investigations, Locat *at al.* [15] has presented a mechanical model describing the strength development with time for lime-stabilized clayey soils, as can be seen in **Figure 2.2** . In this figure two soil-lime mixes are considered: (1) low water content that can be compacted and (2) high water content that cannot be compacted. The model that Perret proposed in 1977 [20] for silty soils is also illustrated in **Figure 2.2** . Because of the successful compaction of the samples, low water content mixes show a significant initial shear strength. In this model, Locat *et al.* [15] has assumed that the reason for the strength increase is mainly the particle bridging by the pozzolonic reaction products as long as reactants are available and he has stated that since the precipitates have finite dimensions, the greater the initial void ratio (or water content), the more time needed to create significant bridges or (contacts) between soil particles.

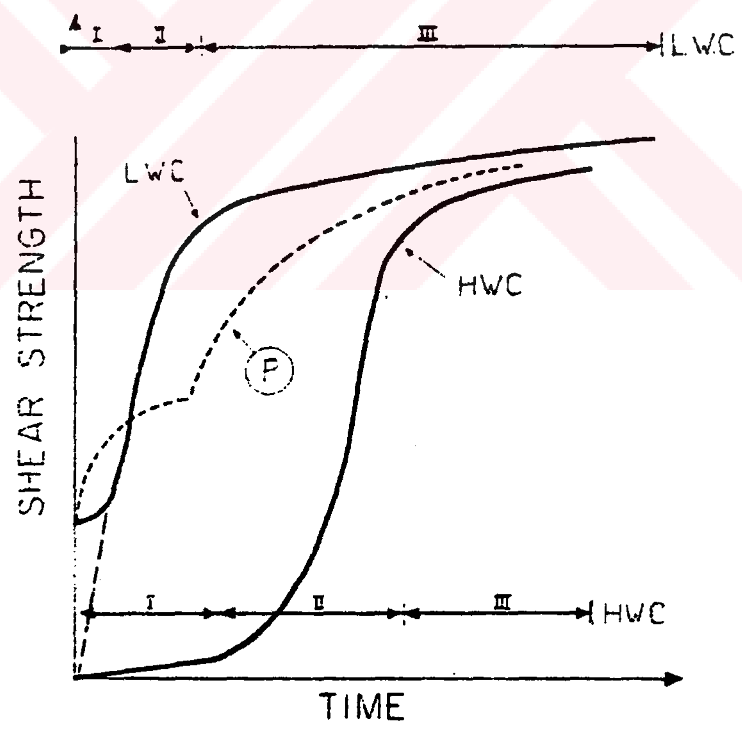


FIGURE 2.2. Mechanical conceptual model of shear strength development with time for high water content and low water content lime-stabilized clayey-soils (P refers to the model of Perret [20]) [15]

Strength development can be divided into three phases with time, as can be seen in **Figure 2.2** [15] . *Phase I* is the is the beginning period when bridging is not mechanically felt, even if the chemical reactions are highly active and cements are formed. This is mostly faced for high water content soils. *Phase II* is a period when bridging development is efficient during the pozzolanic reactions which are mechanically felt as a sharp increase in strength. *Phase III* is distinguished by a decline in the rate of increase of shear strength or even a leveling and this may be attributed to: (a) completion of pozzolanic reactions because of the exhaustion of lime, (2) although reactions are still continuing, it becomes more difficult for solutes to diffuse within the soil-cement matrix, (3) reaction products are not so effective on strength as in *Phase II* even though they are still produced because the soil has a more rigid state in this phase.

For planning a stabilization project, Locat *et al.* [15] proposed a laboratory experimental chart, as illustrated in **Figure 2.3** . In this chart field conditions are given as minimum and maximum. At any time, a chart like the one given in **Figure 2.3** can be built, and then, for a given field water content (point A in **Figure 2.3**), a target strength (point C in **Figure 2.3**) can be chosen for that time, and the lime content can be obtained (point B in **Figure 2.3**).

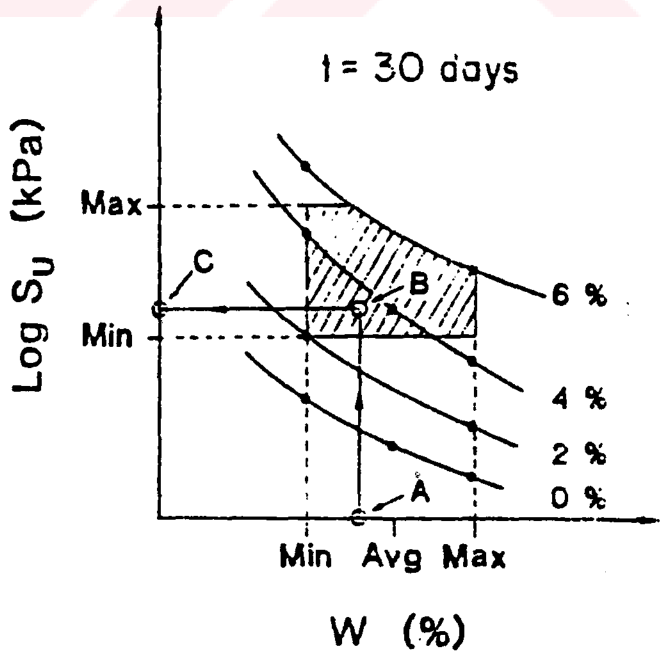


FIGURE 2.3. Laboratory chart proposed for planning lime stabilization projects[15]

2.1.2. Microstructural Development

As mentioned above, four types of reactions occur with the addition of lime into clay: Cation exchange, agglomeration and particle flocculation, lime carbonation and pozzolanic reactions. The last of these, pozzolanic reactions, are attributed to be the main cause of the strength increase in soil-lime mixtures [21].

During pozzolanic reactions, various cementing agents are formed. There are two mechanisms that explain the formation of cement minerals in lime-stabilized soils: (a) Solution-precipitation, (b) Formation of cement mineral directly over the clay minerals. The former mechanism bases the concept that lime takes silica and alumina into solution by creating a highly alkaline matrix and cement minerals precipitate when the solution attains a certain density [22]. On the other hand, the idea behind the latter mechanism is that; the Ca^{++} ions from lime, fractures the clay layers by attacking over the clay leaves and cement minerals form during the diffusion of these ions [23]. Almost all researchers agree that Ca^{++} ions begin attacking the clay minerals from the edges [24].

Baykal [24] has observed the morphologic formation of lime-stabilized soils by the X-ray diffraction study on the samples of compacted lime-bentonite mixture which were exposed to triaxial tests and clarified the above interpretations. In that study, it has been found that the morphologic structure of samples change at the end of 28 days and aggregation borders become more definite as it is the case typically for brittle materials. Baykal [24] has summarized the formation of cement minerals that Ca^{++} ions diffuse into bentonite by cation exchange and this process continue as long as Ca^{++} ions are present in the clay-lime matrix. Thus the structural layers of clay undergo morphologic alteration and cement fibers form.

<u>2.1.2.1. Addition of Fly Ash.</u>

Fly ash is the fine residue results from the combustion of coal. It is primarily composed of silica and alumina and is used for years in stabilization projects of clayey soils together with lime. Baykal [24] has stated that, when fly ash is added with lime into samples mentioned above, the same formation mechanism of cement minerals directly over clay minerals are observed together with solution precipitation mechanism. Therefore when lime is used with fly ash, cement minerals develop even if the soil is not highly reactive. In a similar study, Baykal, Arman and Ferrel [25] has obtained the results illustrated in **Figure**

2.4 . As can be seen in this figure, the addition of fly ash into bentonite together with lime result in sharp increase of the unconfined compressive strength of the samples.

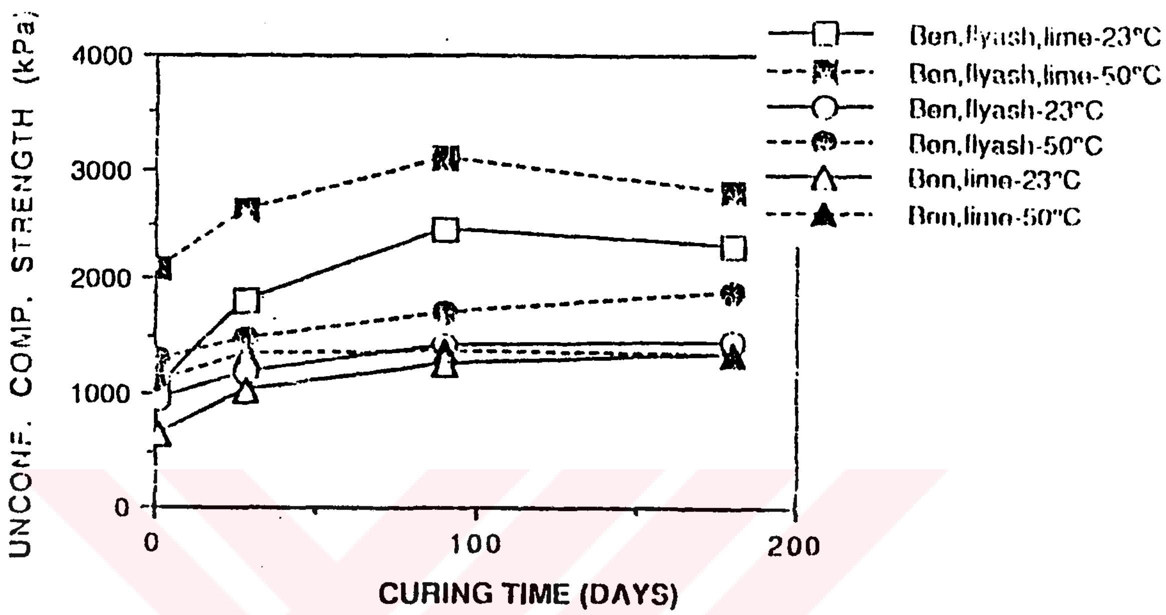


FIGURE 2.4. Average unconfined compressive strength of bentonite-fly ash-lime, bentonite-fly ash, bentonite-lime mixtures versus curing time [25]

2.1.2.2. Effect of Temperature Curing.

The effect of temperature curing on lime-stabilization is determined by several researchers. In **Figure 2.4** it is also shown that by increasing only the curing temperature from 23 °C to 50 °C for the samples of the same mix proportions, a marked increase in unconfined compressive strength is observed.

Arabi and Wild (1986) [26] have found that clay component (illite) of a particular soil, reacts with hydrated lime under favorable curing conditions to produce a cementitious phase and observed by the microstructural examination of fracture surfaces that the formation of cementitious products are limited at low curing temperatures while it is massive at high curing temperatures. In **Figure 2.5**, the variation of unconfined compressive strength with lime content for 24 week cured samples of the soil which is composed primarily of quartz, feldspar and illite is illustrated by Arabi and Wild [26].

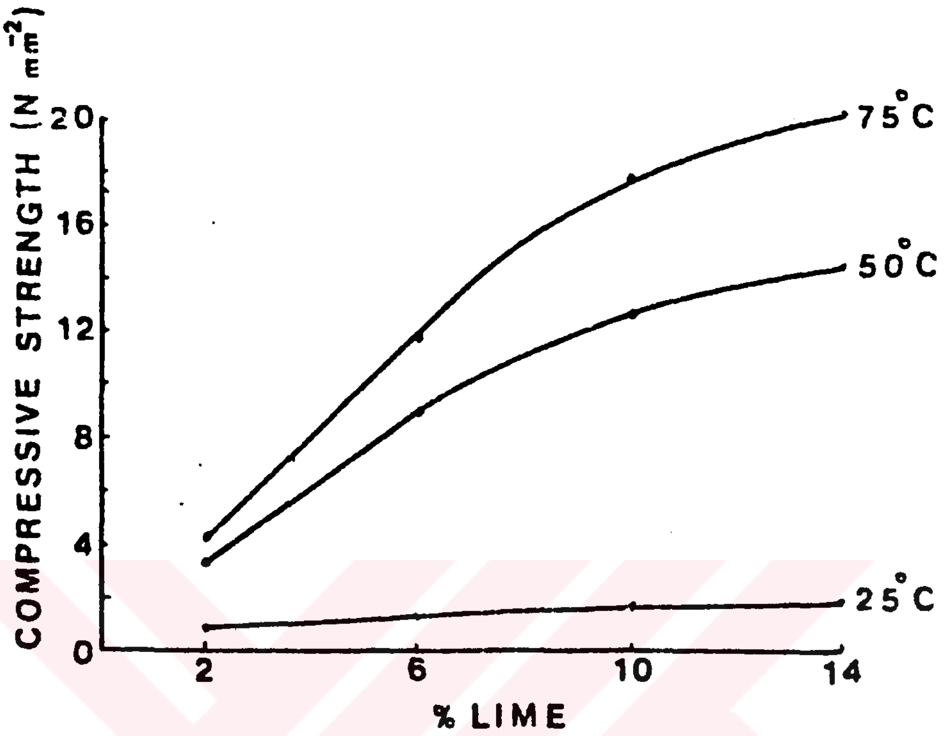


FIGURE 2.5. Variation of compressive strength with lime content for 24 week moist-cured specimens at different temperatures [26]

2.2. Diffusion Property of Lime into Clayey Soils

When cast-in-situ concrete piles or drilled piers are constructed, the soil around these structures may be stabilized by the help of lime slurries prepared in the bore-holes prior to casting the concrete, and thus, the dimensions of the cast-in-situ concrete piles or drilled piers can be decreased during the design. The diffusion property of lime into clayey soils is of great importance for verifying this proposal.

2.2.1. Diffusion in soil: Background

Diffusion of a chemical or chemical species in solution typically is assumed to take place in response to a concentration gradient in accordance with Fick's First Law which may be written for one dimension as:

$$J = -D_o \frac{\partial c}{\partial x} \tag{2.2}$$

where J = the mass flux, c = the concentration of the solute in the liquid phase, x = the direction of transport, and D_o = the "free solution" diffusion coefficient [27].

However, diffusion in soil cannot simply be explained by the above expression. In soil, solutes diffuse in slower rate than in free solution because the pathways for migration are more tortuous and the diffusive mass flux are less in soil than in free solution as solid particles in soil occupy some of the cross-sectional area [27]. These effects can be seen on schematic illustration of **Figure 2.6** .

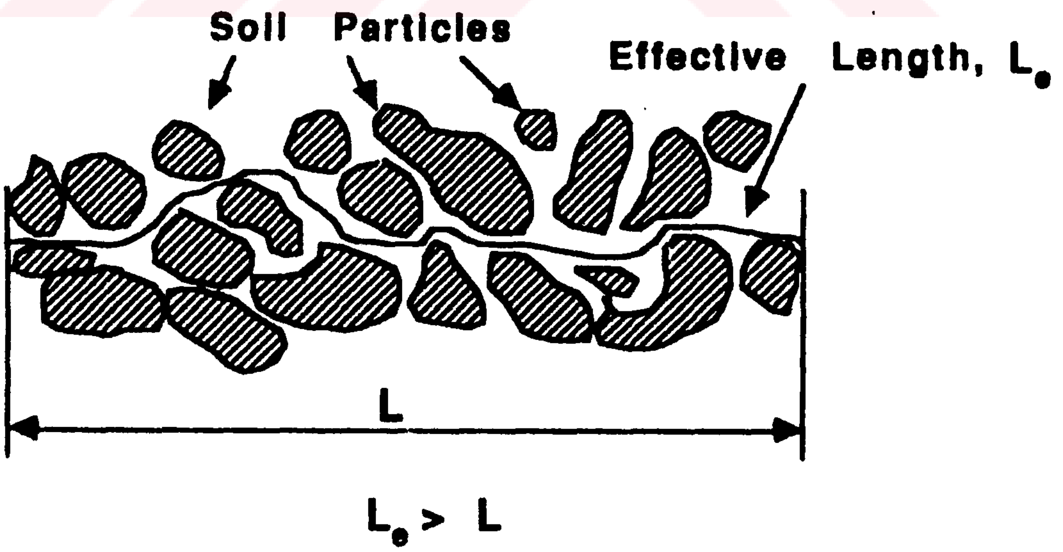


FIGURE 2.6. Concept of effective length in transportation through soil [27]

Since there are many additional factors affecting the diffusion phenomena into soil such as reduction of cross-sectional area, tortuous pathway, fluidity, porosity or degree of saturation, Fick's First Law has been modified by many researchers such as Gilham *et al.* (1984), Li and Gregory (1974), and, Olsen and Kemper (1968). The resulting formula that taking into account these additional effects has been given as;

$$J = D_o \tau \alpha \gamma \theta \frac{\partial c}{\partial x} \tag{2.3}$$

where τ = tortuosity factor, α = fluidity factor, γ = anion exclusion factor, θ = volumetric water content defined as follows;

$$\theta = nS_r \tag{2.4}$$

where n = the total soil porosity and S_r = the degree of saturation of the soil, expressed as a decimal.

Shakelford and Daniel [27] have defined an "apparent tortuosity factor," τ_a, that includes all other factors which may be inherent in its measurement and expressed Fick's First Law for the diffusion of a chemical species into soil as follows:

$$J = -D_o \tau_a \theta \frac{\partial c}{\partial x} \tag{2.5}$$

Since the volumetric water content, θ, is an independently determined variable, Shakelford and Daniel [27] have not included θ in the definition of τ_a Using the definition;

$$D^* = D_o \tau_a \tag{2.6}$$

Shakelford and Daniel [27] have formulated the Fick's First Law as:

$$J = -D^* \theta \frac{\partial c}{\partial x} \tag{2.7}$$

Fick's First Law describes the steady state diffusion flux of solutes but for time dependent transport of nonreactive solute in soil, Ficks second law is assumed to apply as follows:

$$\frac{\partial c}{\partial t} = -D^* \frac{\partial^2 c}{\partial^2 x} \tag{2.8}$$

$$\frac{\partial c}{\partial t} = -D^{*}\frac{\partial^{2} c}{\partial^{2} x} \tag{2.8}$$

On the other hand, for reactive solutes in soil, as it is observed in diffusion phenomenon of lime into clayey soils, Equation 2.8 is modified by as follows [27]:

$$\frac{\partial c}{\partial t} = -D^{*}\frac{\partial^{2} c}{\partial^{2} x} - \frac{\partial q'}{\partial t} \tag{2.9}$$

where q' = the sorbed concentration of the chemical species expressed in terms of the mass of sorbed species per unit volume of void and is formulated as :

$$q' = \frac{\delta_d}{\theta} q \tag{2.10}$$

where q = the sorbed concentration of solute per mass of soil and δ_d = the dry density of the soil. Effective diffusion coefficient can be measured in cell which compacted soil samples are first presoaked so as to prevent mass transport via advection and then exposed to a reservoir of leachate and later sectioned to determine the distribution of the soils at the end of the test [27]. Effective diffusion coefficients can be determined either by measuring the reduction rate of solute concentration in the reservoir or analyzing the final profile of the solute in the soil.

2.2.2. Diffusion of Lime into Clayey Soils

Stocker [28] has made quantitative measurement of the diffusion of lime into unpulverized clay lumps in lime and cement stabilized mixtures of a heavy montmorillonitic clay soil and found that even 1/2 per cent lime could alter the physical properties of this soil. It is also concluded that 1/2 per cent diffused lime in lime or cement stabilized lumps was sufficient to eliminate swelling on wetting from as-cured state, 2 per cent lime to increase as cured strength ten-fold, and 3 per cent lime to produce very low permeability [28].

One of the remarkable results that Stoker [28] has obtained is very little montmorillonite was consumed even when striking changes in physical properties had been produced and a total of 96 per cent of initial montmorillonite remained unreacted in soil made ten times stronger.

The rate of diffusion of lime into soil is also a matter of great importance. Stocker [28] has observed the calcium uptake into lime stabilized clay cores as illustrated in **Figure 2.7**. It is stated that about 7 per cent lime is acquired in the first day in the outer 1 mm of the lime stabilized cores [28] . It is also expressed that although lime in the first 1 mm rapidly rose to 7 per cent, this lime had not been consumed by the usual clay/lime reactions until much later; alternatively if it was incorporated in early products, these had peculiarly high $CaO:SiO_2$ and $CaO:Al_2O_3$ ratios, and further reaction involved desorption of lime with no further uptakes .When the absorbed lime is consumed by reacting with clay it is not replaced even though diffusion still occurs through the outer layers to deeper parts [28] . At greater depth in a lump, Stocker [28] observed that the outer part of the lump itself is the lime source but the potential of this source is not high enough to affect this adsorbtion and reaction of clay uses lime as fast as it is supplied.

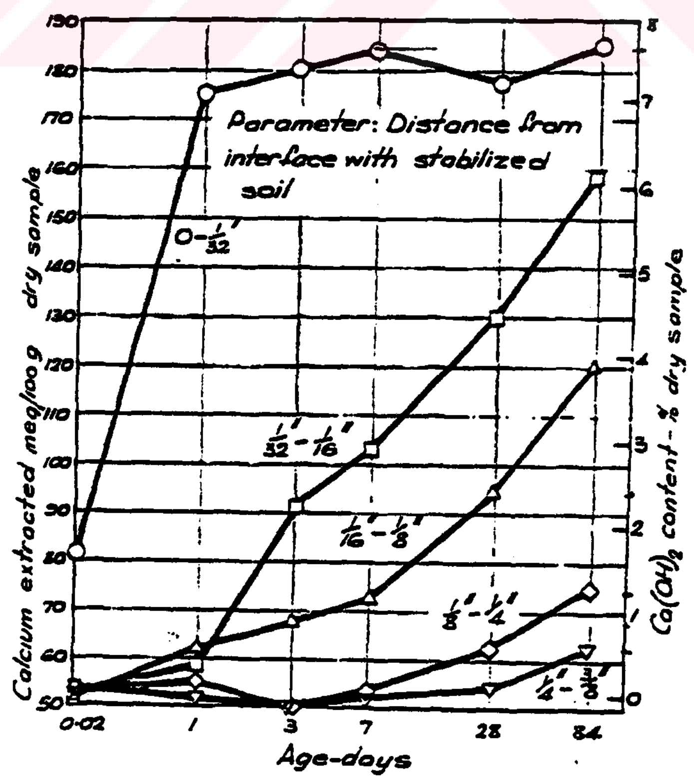


FIGURE 2.7. Diffusion of calcium into cores of lime stabilized (15 per cent) clayey lumps [28]

After 1 day curing, 1/2 per cent lime which is enough to alter the physical properties of clayey soils has reached to 3 mm from outer parts of lumps and after the end of 3rd day, concentration at 3 mm has increased to 1 per cent. The diffusion of calcium into cores of lime stabilized clayey lumps is illustrated on **Figure 2.7**.

2.3. Fundamental Aspects of Piles and Drilled Piers

2.3.1. General

Piles and drilled piers are structural members which are used to transmit surface loads to lower levels onto soil mass when the surface soils are not capable of supporting the applied loads. This mechanism may occur by vertical distribution of the load along their shafts or a direct application to a lower stratum by their end points.

There is no sharp distinction drawn between piles and piers, therefore, terminology in this respect differs considerably in different localities [29]. In literature, specially cast-in-situ concrete piles and drilled piers have almost appeared to be the same even though they are presented exclusively. Bowles [30] has termed drilled piers also as bored piles which are usually restricted to be larger than 760 mm in diameter. Dunn *et al.* [31] has differentiated piles and drilled piers according to their installation methods and named the ones which are driven by impact or vibratory driving hammer as piles and which are backfilled with concrete after drilling or excavating a cylindrical hole as drilled piers.

Although there are different criteria for the categorization of piles and piers, the mechanism for deriving support is essentially the same for piles and drilled piers [30] . Therefore the remaining discussion about pile and drilled piers is devoted to pile foundations. However, much of the discussion in preceding parts applies also to pier foundations.

2.3.2. Soil-Pile Interaction: Load Transfer

Analysis of the bearing capacity of a pile simply depends on the assumption that pile point and every point along the pile shaft move sufficiently relative to the adjacent soil to develop ultimate point and skin resistance of a pile at the same time. However, the displacement needed to mobilize skin resistance is small not exceeding 10 mm regardless of soil and pile type or dimensions, while it is relatively large for mobilizing point resistance, particularly for very large piles [32]. The displacement necessary for mobilizing the point resistance is about 8 per cent of a pile-point diameter for driven piles and as much as 30 per cent for bored piles [32]. Hence, even if the pile-point displacement is not much less than the displacement of pile head as in the case of very rigid piles, the ultimate skin friction is mobilized much earlier than the point resistance, and the fraction of pile total load carried by the pile-point is much less at working loads than at ultimate load [32].

Although the load transfer mechanism between the pile and the adjacent soil is a very complex phenomena involving the stress-strain-time and failure characteristics of all elements of the pile-soil system, numerical assessment of load transfer characteristics of a pile-soil system is required for settlement calculations and for reasonable design of pile foundations. The following introduces the analytical approach presently used.

As stated above, the maximum load capacity of a pile has two components and is calculated by the formula:

$$Q_u = Q_p + Q_s \tag{2.11}$$

where, Q_u = ultimate pile load, Q_p = load carried at the pile point, Q_s = load carried by skin friction. Let the load on a pile be gradually increased from zero to $Q_{(z=0)}$ at the ground surface for a pile of length L, as shown in **Figure 2.8a** . Part of this load is resisted by the side friction (Q_1), while part of it is carried by the soil below the tip of the pile (Q_2), and the nature of the variation of load carried by the pile shaft ($Q_{(z)}$) will be as given in **Curve 1** of **Figure 2.8b** . For a given depth z and pile of perimeter *p, the frictional resistance per unit area,* ($f_{(z)}$) is formulated as below and the nature of variation of that with depth is shown in **Figure 2.8c** [33].

$$(f_{(z)}) = \frac{\Delta Q_{(z)}}{(p)(\Delta z)} \tag{2.12}$$

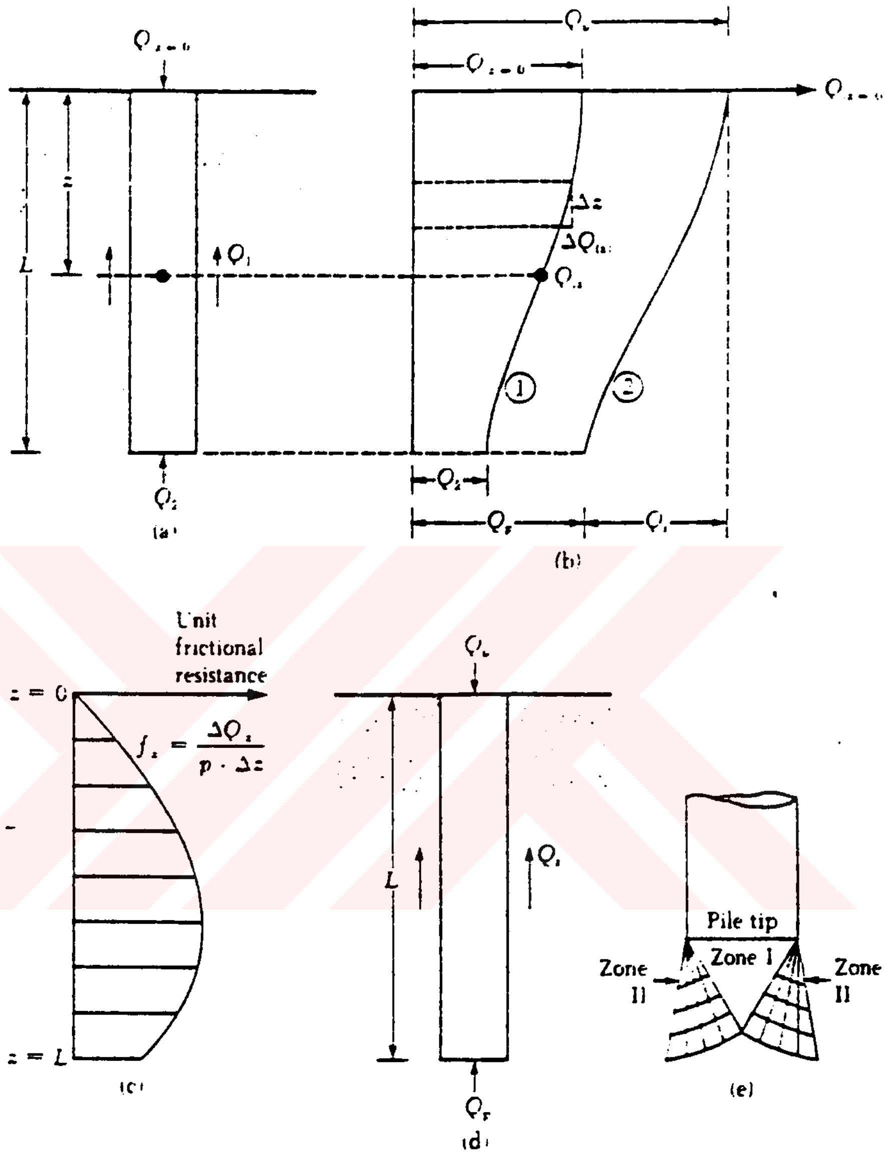


FIGURE 2.8. Load transfer Mechanism for Piles [33]

In case of ultimate load like that in **Figure 2.8d** and **Curve 2** in **Figure 2.8b**, $Q_{(z=0)} = Q_u$, $Q_1 = Q_s$ and $Q_2 = Q_p$. However, as it is mentioned above, skin friction, Q_s is developed at a much smaller displacement compared to the point resistance, Q_p [33].

Finally, since the failure surface in the soil at the pile point is like shown in **Figure 2.8e**, the failure type of the soil is almost in punching mode, therefore, a ***triangular zone***, **I**, is developed at the pile tip, which is pushed downward with producing no other apparent slip surface [33].

2.3.3. Resistance of Piles

The ultimate load capacity of a pile has two components as given in **Equation 2.11**; the point resistance and the frictional resistance.

The unit point resistance of a pile is calculated in a similar way to ultimate bearing capacity of shallow foundations. The difference is the bearing capacity factors related to shape and depth. Also, since the pile diameter D is relatively small, the term coming from friction can be neglected for the calculation of point bearing capacity of piles. In general, the unit point bearing capacity (q_p)of piles can be expressed as;

$$q_p = c\,N^*_c + q'\,N^*_q \tag{2.12}$$

where c is the cohesion of the soil supporting the pile tip, q' is the effective vertical stress at the level of the pile tip, and N^*_c and N^*_q are the bearing capacity factors which can be determined by several methods including Meyerhof's, Vesic's, Hansen's or Janbu's Method. Hence, the point bearing capacity (Q_p) of piles can be calculated as;

$$Q_p = A_p\,q_p = A_p\,(c\,N^*_c + q'\,N^*_q) \tag{2.13}$$

where A_p is the area of the pile tip.

The second component of the ultimate load capacity of a pile is *the frictional* or *skin resistance* of which the increase with the addition of lime into soil is the main subject of interest in this study The skin resistance capacity of a pile can be written as;

$$Q_s = \Sigma\,p\,\Delta L\,f_s \tag{2.14}$$

where p = perimeter, ΔL = embedment increment, f_s = skin resistance, and Σ = summation of contributions from several strata or pile segments.

There are several methods for calculating the skin resistance f_s including the α method, β method, and λ method.

The α Method was proposed by Tomlinson (1971) [34], and fundamentally the skin resistance is computed as

$$f_s = \alpha\,c + \bar{q}\,K\,tan\delta \tag{2.15}$$

where α = empirical adhesion factor, c = average cohesion (or s_u) for the soil stratum of interest, $\bar{q}$ = effective vertical stress on element ΔL of the soil stratum, K = coefficient of lateral earth pressure ranging from K_o to 1.75, depending on volume displacement, initial soil density, etc., and δ = effective friction angle between soil and pile material. For clayey type of soils, the variation of α with undrained cohesion, c_u, is shown **Figure 2.9**.

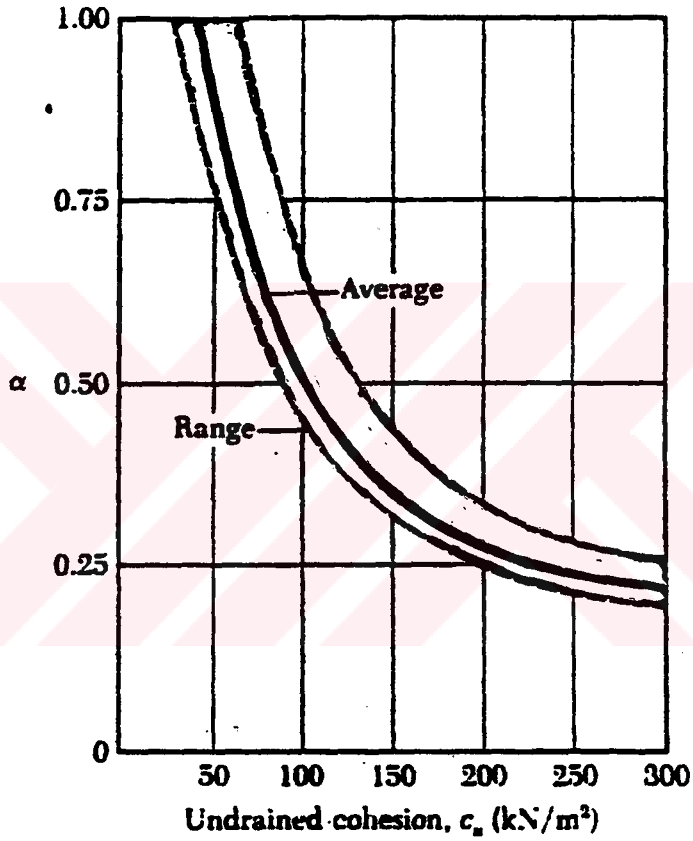


FIGURE 2.9. Variation of α with undrained cohesion of clay [34]

Vijayvergiya and Focht (1972) [35] presented a method called **The λ Method** for obtaining the skin resistance of a pile in clay as

$$f_s = \lambda(\bar{q} + 2\,s_u) \tag{2.16}$$

where $\bar{q}$, s_u = values defined in **Equation 2.15**, and λ = coefficient which can be obtained from **Figure 2.10**.

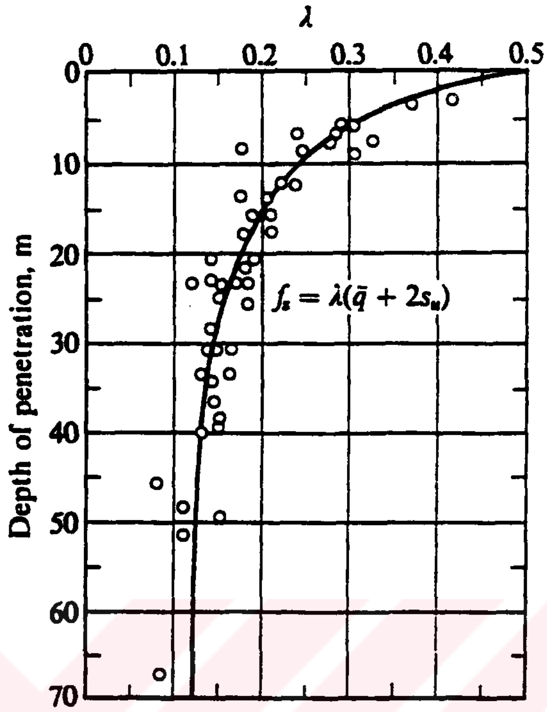


FIGURE 2.10. λ coefficients depending on pile penetration [35]

Another method which is known as **"The β Method"** was suggested by Burland (1973) [36] by making the assumptions; first, remolded soil adjacent to the pile has a cohesion of zero during the pile driving, second, the effective stress acting on the pile surface is equal to the initial effective stress K_o of the soil after the dissipation of excess pore pressure, and third, the major shear distortion occur around the pile shaft and the drainage of this zone occur rapidly after loading or in delay between driving and loading. With these assumptions Burland [36] has expressed the skin resistance as

$$f_s = K\,\bar{q}\,tan\delta \qquad (2.17a)$$

Taking $\beta = K\,tan\delta$, and modifying for a surcharge q_s, equation for skin resistance can be rewritten as

$$f_s = \beta\,(\,\bar{q} + q_s\,) \qquad (2.17b)$$

where as previously used, $\bar{q}$ = average effective vertical stress for the i th element of length ΔL.

There are some other methods for estimating skin resistance. For SPT data, Meyerhof (1956) [37] suggested obtaining f_s as

$$f_s = X_m N \qquad \text{(kPa)} \tag{2.18}$$

where X_m = 2.0 for piles with large volume displacement piles and 1.0 for small volume piles, and N = statistical average SPT blow count in the stratum.

For cone-penetration data, Meyerhof (1956) [37] and Thorburn and Mac Vicar [38] suggest

$$f_s = 0.005\, q_c \qquad \text{(kPa)} \tag{2.19}$$

where q_c = cone-penetration resistance in kPa.

2.3.4. Allowable Pile Capacity

The allowable pile capacity Q_{all} is obtained from applying a suitable safety factor on both components of ultimate resistance as

$$Q_{all} = \frac{Q_p}{\mathrm{SF_p}} + \frac{Q_s}{\mathrm{SF_s}} \tag{2.20a}$$

or using a single value SF to obtain

$$Q_{all} = \frac{Q_u}{\mathrm{SF}} \tag{2.20b}$$

where SF_i represents the safety factor generally ranging from 2.0 to 4 or more, depending on the designer uncertainties. There are mixed opinion whether safety factor should be based on both resisting mechanism or be a single value. In general, the safety factor used for piles are larger than for spread foundations because of the uncertainties in soil-pile interaction.

2.3.5. Cast-in-situ Concrete Piles or Drilled Piers

2.3.5.1. Construction Method

Cast-in-situ concrete piles or drilled piers are constructed by three different methods today: dry method, casing method and slurry method.

In *dry method*, as can be seen in **Figure 2.11**, first the shaft is drilled, and then partially filled with concrete with the rebar cage. This method requires the site soil to be cohesive and water table to be below the base or the permeability to be low so that water does not fill the bore-hole before concrete casting and affect concrete strength.

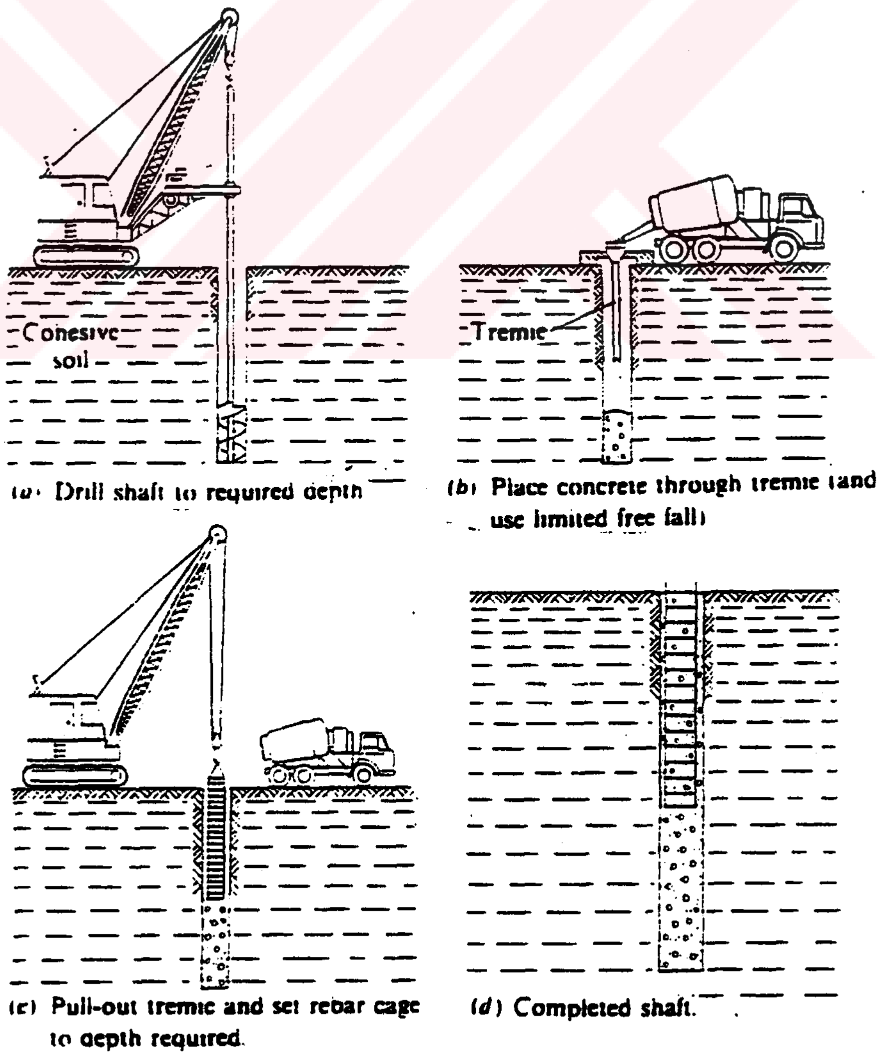


FIGURE 2.11. Dry method of drilled pier construction [30]

Casing method which is outlined in **Figure 2.12** , is used at sites where caving or extreme lateral deformation toward the shaft cavity can occur. Also it is used to seal the hole against ground water entry but this requires an impermeable stratum below the caving zone which the casing will be socketed. Until the casing is placed, the shaft may be filled with a slurry and this slurry is bailed out while the casing is being seated to the required depth. The casing may be left in place or pulled. If it will be left in place, the slurry inside is replaced with pressure-injected grout of cement mixture. Optionally, if the casing will be pulled, great care is necessary to ensure that the concrete inside the casing is still in a fluid state and concrete "head" is always sufficiently greater than the slurry "head" that concrete displaces slurry and not vice versa.

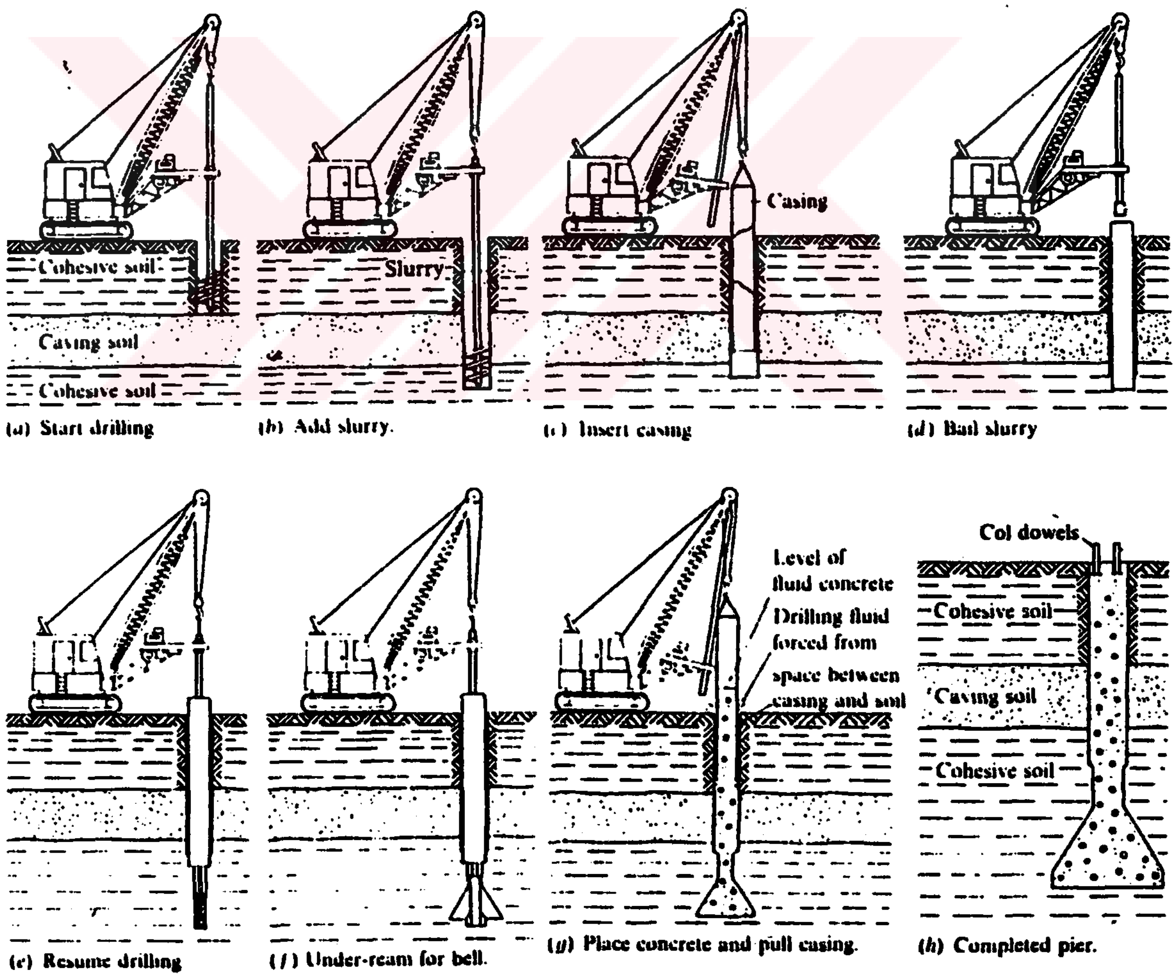


FIGURE 2.12. Casing method of drilled pier construction [30]

Slurry Method is applicable for any situation requiring casing specially where casing is not adequate for keeping the ground water out of the cavity. The important point in this method is that there is sufficient slurry "head" inside the shaft than that of the ground water table or the tendency of the soil to cave. The most commonly used slurry is "bentonite slurry". Approximately 4 to 6 per cent bentonite by weight is thoroughly mixed with water and this mixture forms a filter cake on the shaft wall and carry the smaller excavated particles in suspension. With the slurry method, it is generally desirable not to have slurry in the shaft too long, since it may cause the filter cake difficult to be displaced with concrete. Furthermore, it is also necessary to pump the slurry by screening out the larger particles in suspension with the "conditioned" slurry returned to the shaft just prior to concreting. Also, it is important not to cause collapse of part of the shaft because of pulling a large fragment during excavation. The outline of the several steps of slurry method can be seen in **Figure 2.13** .

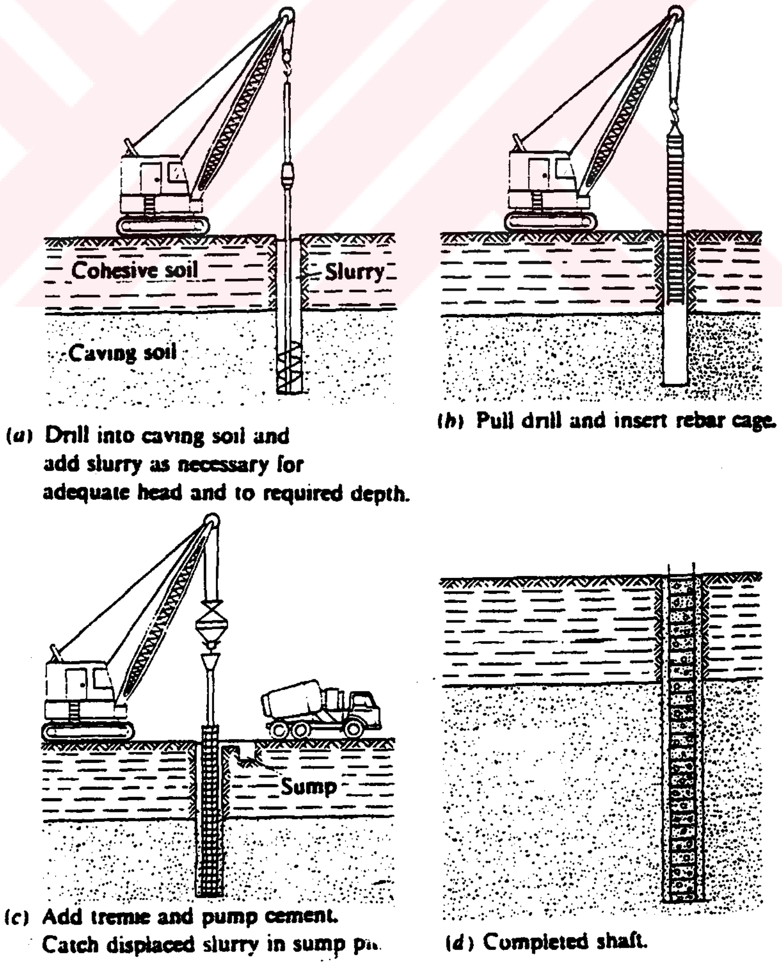


FIGURE 2.13. Slurry method of drilled pier construction [30]

2.3.5.2. Important Aspects of Shaft Resistance for Bored Cast-in-situ Piles in Clayey-soil

Field observations on bored piles [39] and microfabric studies on laboratory specimens [40] have shown that shearing occurs in a thin zone within the clay close to the pile shaft. Also Meyerhof (1983) [41] stated that the side effects are unlikely to influence the shaft adhesion in bored piles. This means, the diameter or length of the bored pile does not have an effect on shaft adhesion. Another aspect about the shaft adhesion is that; during augering the soil is sheared and therefore after the casting of concrete, small settlements mobilize full shaft adhesion.

Furthermore, it is stated that if there is a delay between the processes of boring and concreting, friction angle at the pile-soil interface decreases [42]. Although the friction angle between soil and concrete is greater than the internal friction angle initially due to the rough surface of concrete, it becomes the same or smaller with a one day delay of concreting. In this case, it is observed that shearing takes place on the smoother pile-soil interface rather than in the soil [42].

Skempton [43] has stated that a reduction of adhesion occurs essentially because of the softening of clay immediately adjacent to the contact surface, and this softening increasing by an increase in water content around the pile shaft. In a field case, Meyerhof and Murdock (1953) [44] have measured an increase in water content approximately by 4 per cent at the contact surface although at a distance of nearly 7.5 cm from the shaft, the water content of the clay had not been altered. Skempton [43] has found that even if there is perfect adhesion between the clay and the concrete, and when the increase in water content is zero, the ratio of the adhesion to the original strength is 0.8 . Moreover, a variation of only 1 per cent in the water content is responsible for a 20 per cent change in adhesion factor α [43].

There may be four reasons which is responsible for the increase in water content around the pile shaft [43]:

(a) water flowing out of the clay during the process of boring.

(b) migration of the water from clay through the less highly stressed zone around the pile to the bore-hole.

(c) water tipped into the boring for the operation of cutting tool.

(d) Water derived from the concrete.

Of these causes (a) and (c) can be eliminated or decreased by good drilling techniques or carrying out the drilling and concreting operations rapidly but (b) and (d) would seem inevitable [43].

If the maximum increase in water content is taken to be typically in the region of 5-6 per cent, then the softened strength will be about 30 per cent of the original strength [43]. Also, Meyerhof and Murdock [44] has found the lowest value of α as 0.3 . Skempton [39], has shown the relation between shear strength and water content for a particular clay as in **Figure 2.14** .

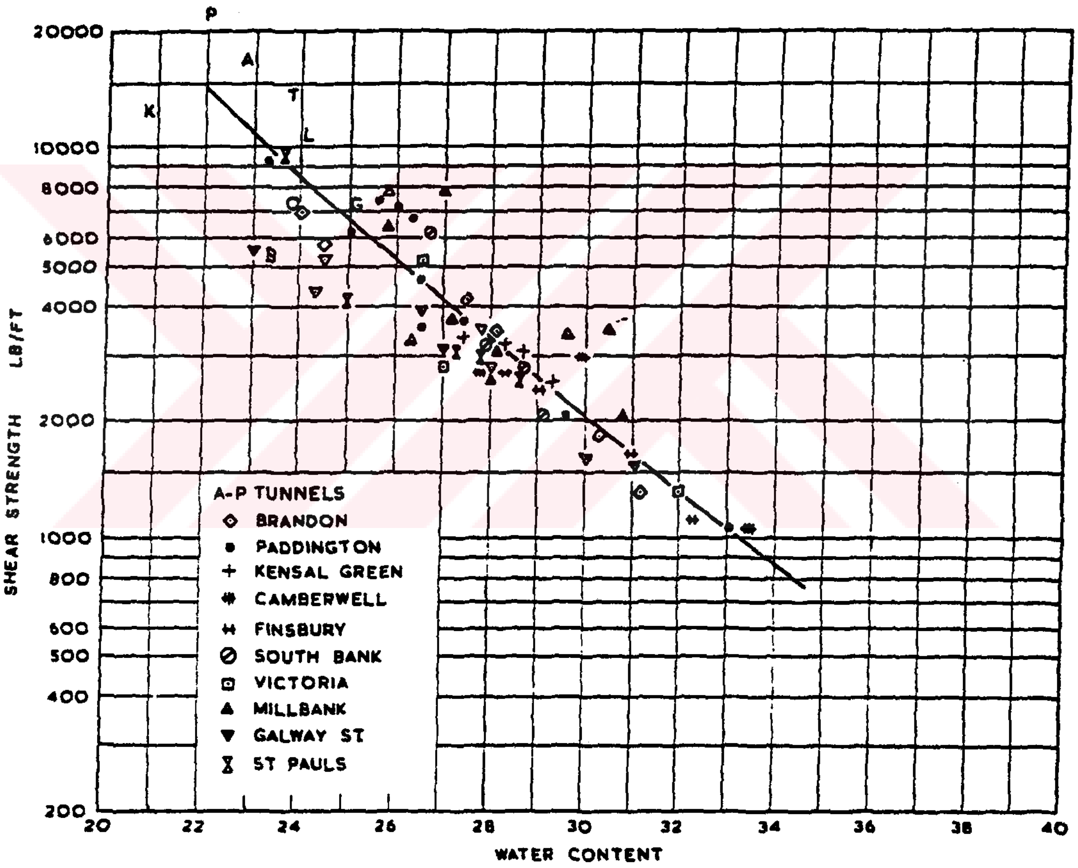


FIGURE 2.14. Relation between shear strength and water content for London Clay (liquid limit, 70-85) [43]

Another factor affecting the shaft adhesion between concrete piles and clayey- soils might be free lime in cement diffused from concrete during setting. Although the adhesion factor is found to be as small as 0.3 per cent, in practice it is generally higher. In many instances, it is found twice as expected [43]. This might be a result of stabilizing effect of diffused free lime from the cement in concrete on the adjacent layer of clay.

In cement pastes, mortars and concrete, tricalcium silicate hydrolyzes, that is, breaks it down to a calcium silicate or lower basicity, to form $2CaO.SiO_2$ and release the excess lime as calcium hydroxide, $Ca(OH)_2$ [1]. Thus, the $Ca(OH)_2$ precipitates out as crystals from the supersaturated $Ca(OH)_2$ solution produced by the rapid rate of hydrolysis of tricalcium silicate. The total free lime amount in "Normal Portland Cement" is detected as about 1-2 per cent by weight [1]. Being partly hydrated $Ca(OH)_2$ and partly unhydrated (CaO), the unreacted lime in cement is not related much to the strength developed [45] in concrete.

3. PRESENT STUDY

In this study, the effect of the treatment of clayey soils with lime at the interface of concrete structures like cast-in-situ concrete piles or drilled piers is investigated in the laboratory. For making comparison of the shear strength parameters at the interface of concrete between lime-stabilized and non-stabilized soils, *direct shear tests* are performed. Later on, the results obtained from shearing the layered samples of soil and concrete, are compared with the results obtained from shearing the soil alone.

Since the method in this study is proposed for clayey soils, bentonite, being very active and giving high response to the stabilizing effect of lime, is chosen as the clay type. However, in the field, generally clay is not present in pure state. So as to provide similar characteristics with soils commonly faced in the field, bentonite is mixed with different amounts of sand from zero to 50 per cent by weight and some of these mixtures are stabilized with different amounts of lime.

3.1. Materials

As mentioned above, three types of materials are used for the preparation of the test samples: bentonite, sand, and lime. The physical and mechanical properties of these materials are given below.

3.1.1. Clay

The clay used in this study is a type of sodium bentonite which is known as "Çanakkale Mito Bentonite" in Turkiye. The typical mineralogy and index properties and the geotechnical properties of the bentonite which is used in the experiments are given in **Table 3.1** and **Table 3.2** respectively. The grain size distribution of this bentonite can be seen in the first line of **Table 3.6** as soil type 1.

TABLE 3.1. Typical mineralogy and index properties of bentonite (After Grim and Güven) [46]

Chemical Analysis	(%)
SiO_2	69.01
Al_2O_3	22.36
Fe_2O_3	3.55
MgO	4.99
CaO	0.02
Na_2O	0.07
Rational Analysis (roughly in general)	(%)
smectite (mostly montmorillonite)	80
micas, feldspar, quartz, impurities	20
Index Properties (for Na-Bentonite)	
liquid limit (%)	344-700
plastic limit (%)	89-97
unconfined compressive str. (kPa)	37.92
specific gravity	2.64
activity	3.14-7.09

TABLE 3.2. Geotechnical properties of Bentonite used in this study

	Bentonite
Liquid Limit (%)	302.5
Plastic Limit (%)	41.3
Plasticity Index (%)	261.5
Optimum Water Content (%)	41
Maximum Dry Density kN/m^3	10.25
Finer than 2μ (%)	59.4
Activity	4.4

3.1.2. Sand

In the preparation of both concrete and soil sections of the samples, "Sakarya River Sand" is used. Since the dimensions of the samples are small, the sand particles which cannot pass from sieve no.20 (0.85 mm) are not taken. Also, for avoiding unpredictable effect of small size particles in reactions, the sand particles which pass from sieve no.200 (0.075 mm) are not used. The particle size distribution of this sand is shown in **Figure 3.1** . From calculations, the coefficient of uniformity, C_u, is found as 2.5 and the coefficient of concavity, C_c, as 1.74. As can be seen from **Figure 3.1** and can be inferred from the value of 2.5 as the coefficient of uniformity, the sand is "poorly graded". The USCS classification of sand is SP.

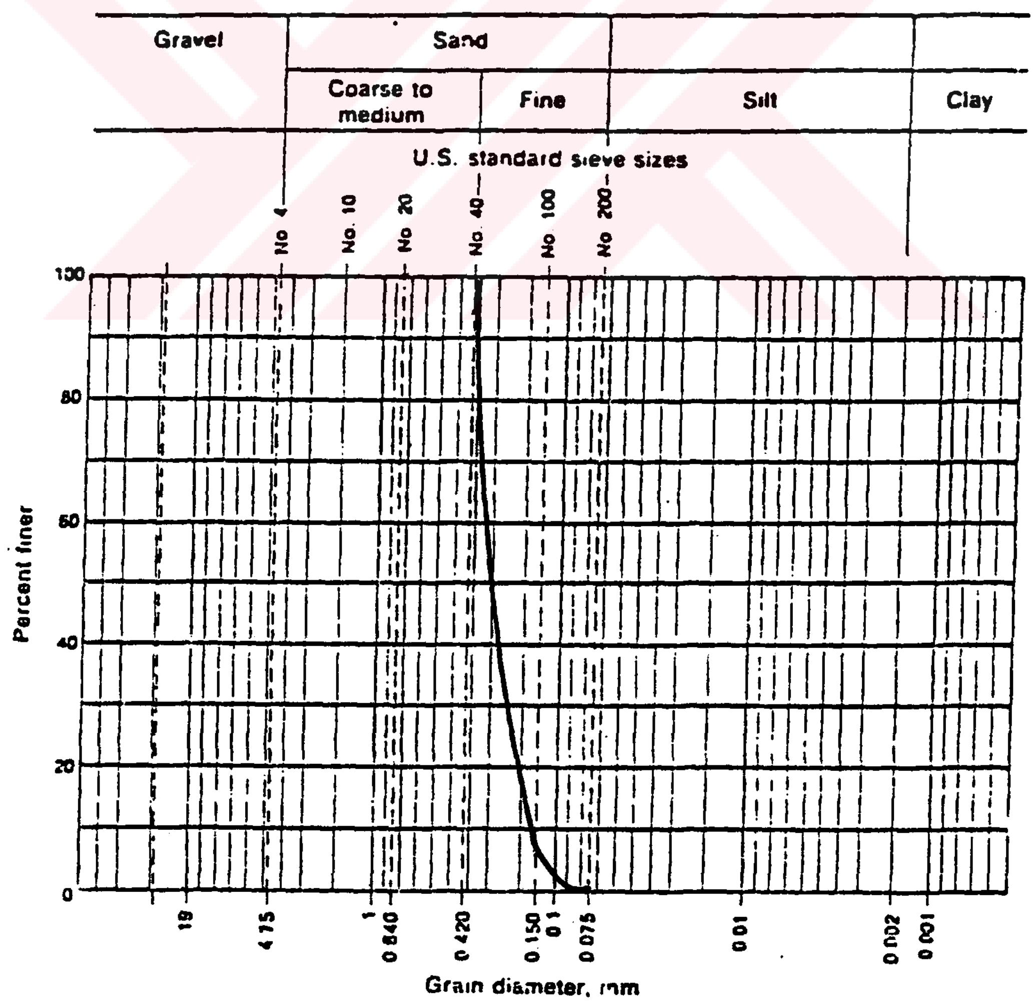


FIGURE 3.1. Grain size distribution of the sand used in the experiments

3.1.2. Lime

The lime used in this study is a slaked lime which is commercially available. Neutralization process by acid titration has revealed that the weight percentages of $Ca(OH)_2$ and CaO in this commercial lime are 35.93 and 27.10 respectively. The sieve analysis of this lime is shown in **Table 3.3** .

In general, the properties and composition of lime show variations from place to place. In **Table 3.4**, the properties of commercial limes which was published by "National Lime Association (USA), (1973)" is shown.

TABLE 3.3. Sieve analysis of the lime used in the experiments

Sieve Size	Per cent Passing
No.10 (2.00 mm)	99.30
No.20 (0.85 mm)	97.64
No.40 (0.425 mm)	62.34
No.60 (0.25 mm)	25.97
No.100 (0.15 mm)	3.30
No.200 (0.075 mm)	0.72

TABLE 3.4. Properties of commercial limes [47]

(a) Quicklime

Constituent (per cent)	High Calcium	Dolomitic
CaO	92.25 - 98.00	55.70 - 57.50
MgO	0.30 - 2.50	37.60 - 40.80
CO_2	0.40 - 1.50	0.40 - 1.50
SiO_2	0.20 - 1.50	0.10 - 1.50
Fe_2O_3	0.10 - 0.40	0.05 - 0.40
Al_2O_3	0.10 - 0.50	0.05 - 0.50
H_2O	0.10 - 0.90	0.10 - 0.90
Specific Gravity	3.2 - 3.4	3.2 - 3.4

(b) Hydrates

	High Calcium	Monohyd. Dolomitic	Dihyd. Dolomitic
Principle Constituent	$Ca(OH)_2$	$Ca(OH)_2$+MgO	$Ca(OH)_2$+$Mg(OH)_2$
Specific Gravity	2.3 - 2.4	2.7 - 2.9	2.4 - 2.6

3.2. Sample Preparation

First, control mixes of soil including no lime, 3 per cent lime or 7 per cent lime by weight, were prepared and cured for 28 days and were tested. Second, to determine the shear strength improvement at the interface of clayey-soils and concrete structures with the addition of lime into soil, the direct shear test samples were prepared as two layers of soil and concrete, and the results were compared with results of direct shear tests performed on control samples.

For the preparation of the layered samples, after compacting the soil mixtures at the optimum water content, samples were taken by a standard sample cutter and cut by half. Then, the soil sections of the samples having the half height of a standard direct shear test sample were placed into special glass molds which were previously prepared. Finally, concrete sections of the samples were kindly placed over the soil sections. In the mix design of concrete, since the heights of the concrete sections of the samples were small like the soil sections, only aggregates which all passing from sieve no.20 (0.85 mm) were used and most of these aggregates were fines which pass from sieve no.40 (0.425 mm).

After the preparation of the layered samples, they were cured for 28 days in molds like only soil samples. The reason for choosing 28 days for curing period is first, concrete reaches to its design strength and second, lime has enough time to change the formation of the bentonite in a great extent. Also, it may be assumed that lime in the soil sections of the samples and free lime formed in the concrete diffuse into soil and alter the properties of soil.

The height of each half of the samples is 0.855 cm and the dimensions of the base are 5.08 cm x 5.08 cm. The cross-section of a layered sample is shown in **Figure 3.2**.

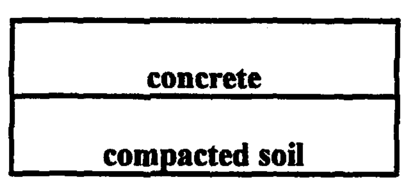


FIGURE 3.2. Schematic profile of the layered direct shear test samples

Control (pure soil) samples or soil sections of the layered samples were prepared by compacting bentonite, sand, and lime mixture of various proportions at the optimum moisture content. Bentonite, being a very active clay is attacked by lime. Its microstructure and mechanical properties change after reacting with lime. Therefore, improving effects of lime are observed better on bentonite. Various amounts of sand was also mixed with bentonite so as to observe the effect of lime on soil which have lower plasticity index (PI) and liquid limit (LL) as can be faced occasionally in real applications. The grain size distribution of sand is given in **Figure 3.1**. The mix proportions of soil types are given in **Table 3.5**.

As can be seen in **Table 3.5**, four types of soils were taken first which based on bentonite and/or sand. The classification of these soils were ranging from CH to SC in "USCS" classification system and A-7-5 according to the classification system of "AASHTO". By adding 3 per cent by weight lime into each of these soils, the second group of four soils were obtained. Finally, 7 per cent lime was added to the original four samples and the last set of four soils was obtained. Thus, 12 type of soils were obtained, which first four of them were not stabilized, second four were stabilized with 3 per cent lime, and the last four were stabilized with 7 per cent lime, and were used for the comparative studies of shear strength determination. The classification of each soil prepared, may be seen in **Table 3.6** together with grain size distribution, liquid limit (LL), plastic limit (PL), and plasticity index (PI) values.

By changing the mix proportions of soil samples (or soil sections of the layered samples), different mechanical properties were obtained as expected. In **Figure 3.3** and **Figure 3.4** LL and PI values versus sand amount in bentonite-sand mixture is shown respectively for different amounts of lime added. On the other hand, **Figure 3.5** and **Figure 3.6** shows those values versus lime content in bentonite or bentonite-sand mixtures for different amount of sand in mixture. In **Table 3.7**, clay fraction and activity values of soil part of the samples are given. The twelve types of soil **(Table 3.8,)** were compacted at optimum moisture contents. For obtaining optimum moisture contents of the soil types, a number of compaction tests were performed. Maximum dry unit weights and optimum moisture contents of the soil types can be seen on **Table 3.8 .**

TABLE 3.5. The mix proportions of the soil types

Soil Type	Proportions of the Clay and Sand (%)		Added Lime(%)	The Mix Proportions of the Composite Soil Types (%)		
	Bentonite	Sand		Bentonite	Sand	Lime
1	100	0	0	100	0	0
2	90	10	0	90	10	0
3	70	30	0	70	30	0
4	50	50	0	50	50	0
5	100	0	3	97.09	0	2.91
6	90	10	3	87.38	9.71	2.91
7	70	30	3	67.96	29.13	2.91
8	50	50	3	48.54	48.54	2.91
9	100	0	7	93.46	0	6.54
10	90	10	7	84.11	9.35	6.54
11	70	30	7	65.42	28.04	6.54
12	50	50	7	46.73	46.73	6.54

TABLE 3.6. Classification of soil samples

Soil Type	Percent Passing						LL	PI	Classification	
	No.10	No.20	No.40	No.60	No100	No200			USCS	AASHTO
1	100	100	100	100	97.0	80.9	302.8	261.5	CH	A-7-5
2	100	100	99.5	93.5	88.1	72.7	268.4	231.7	CH	A-7-5
3	100	100	98.4	80.6	70.2	56.6	215.2	180.1	CH	A-7-5
4	100	100	97.3	67.6	52.3	40.4	155.6	128.1	SC	A-7-6
5	100	99.9	98.9	97.8	94.3	78.5	153.9	106.4	CH	A-7-5
6	100	99.9	98.4	91.6	85.6	70.7	144.8	93.4	CH	A-7-5
7	100	99.9	97.3	79.0	68.2	55.0	103.9	57.6	MH	A-7-5
8	100	99.9	96.3	66.4	50.8	39.3	66.7	30.8	SM	A-7-5
9	100	99.9	97.5	95.2	90.9	75.6	124.9	69.9	MH	A-7-5
10	100	99.9	97.0	89.1	82.5	68.1	119.5	66.1	MH	A-7-5
11	100	99.9	96.0	77.0	65.8	52.9	90.7	43.5	SM-MH	A-7-5
12	100	99.9	95.0	64.9	49.0	37.8	64.8	28.7	SM	A-7-5

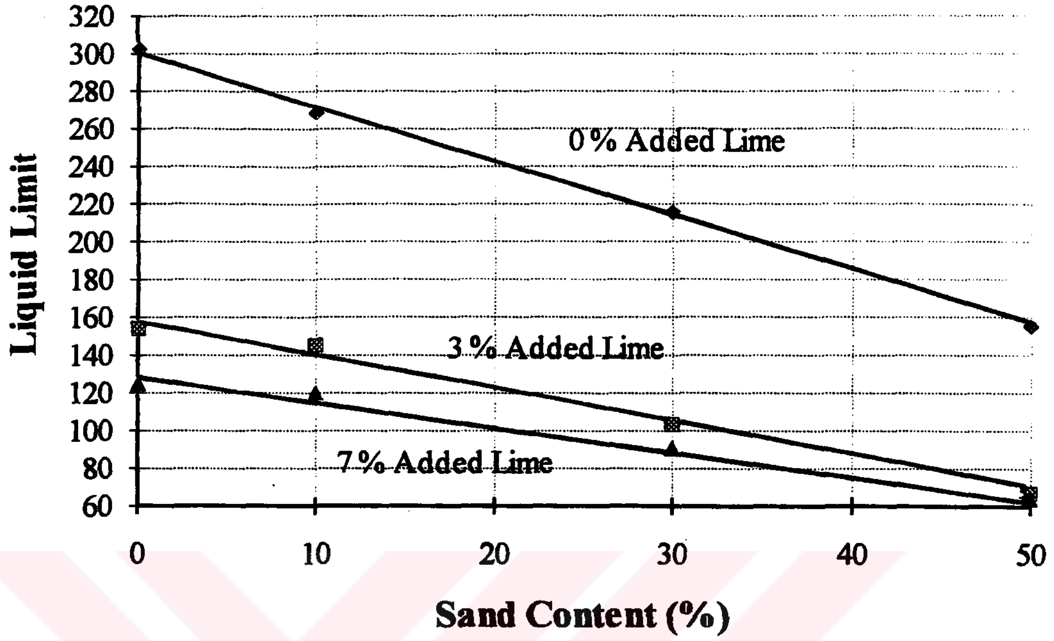


FIGURE 3.3. Liquid limit versus the sand amount in bentonite-sand mixture

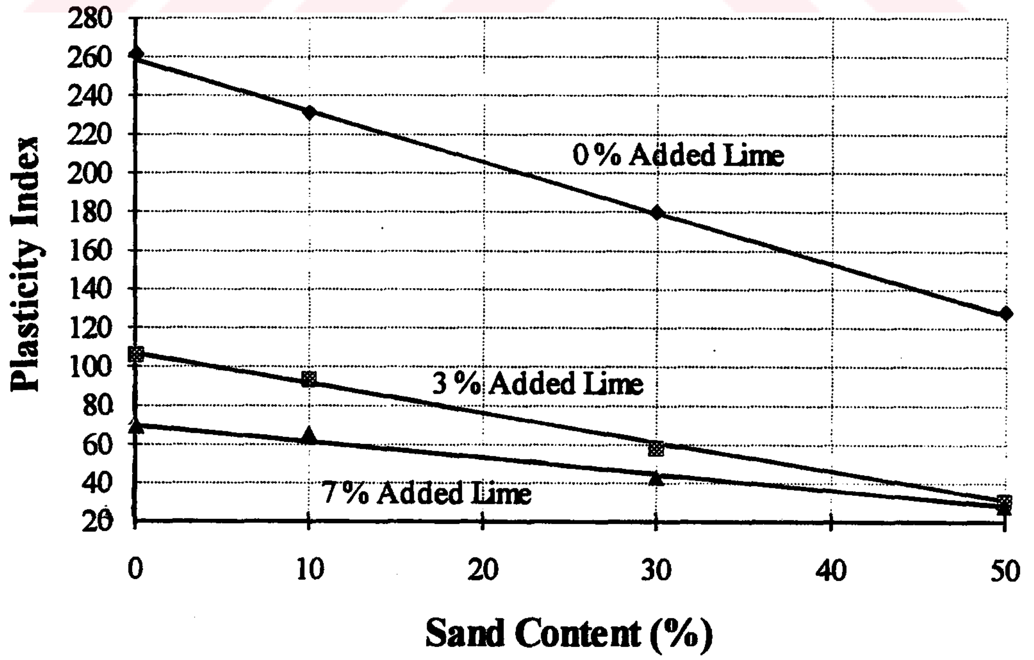


FIGURE 3.4. The plasticity index versus sand amount in bentonite-sand mixture

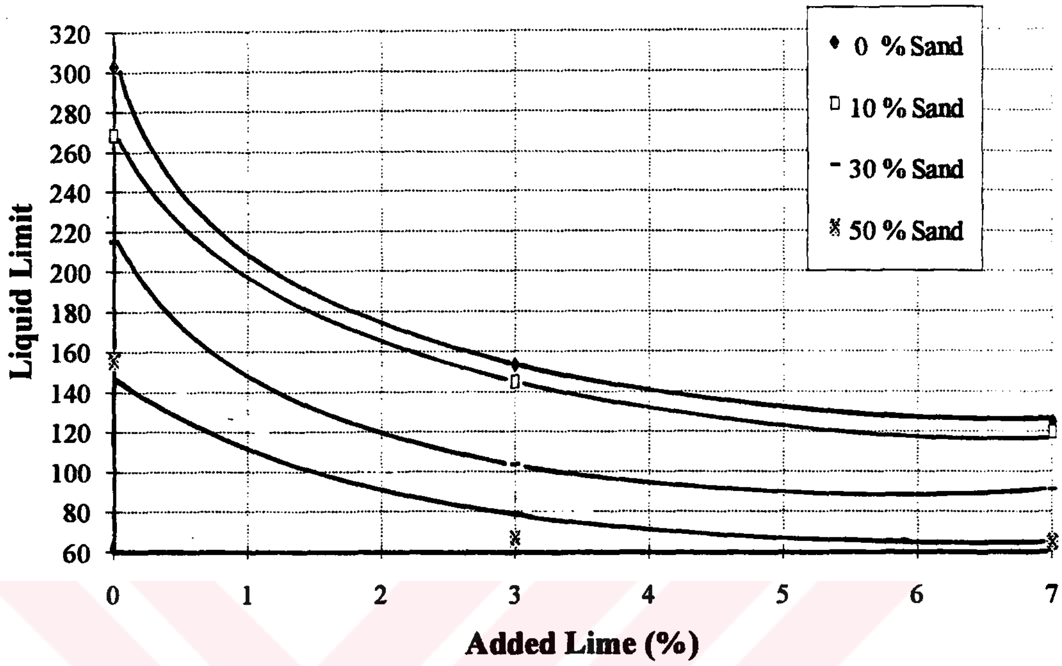


FIGURE 3.5. Liquid limit versus added lime for different sand amounts in samples

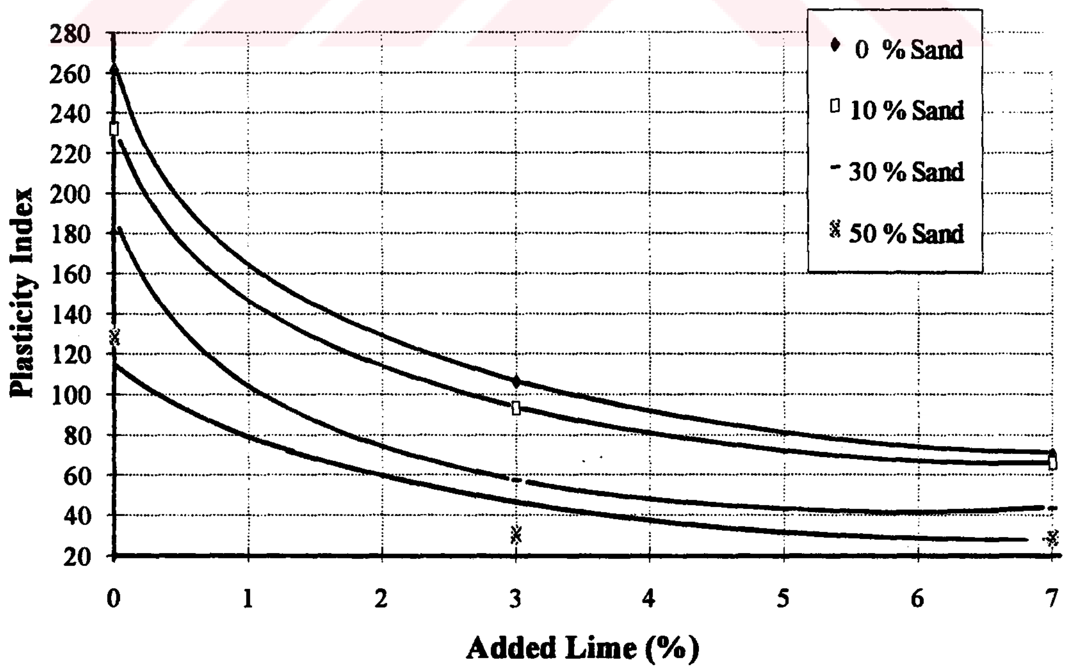


FIGURE 3.6. Plasticity index versus added lime for different sand amounts in samples

TABLE 3.7. The clay fraction and activity values of soil types used in the direct shear tests

Soil Type	Clay and Sand (%)		Added Lime(%)	PI	Clay Fraction*	Activity**
	Bentonite	Sand				
1	100	0	0	261.5	59.4	4.4
2	90	10	0	234.3	53.5	4.4
3	70	30	0	180.2	41.6	4.3
4	50	50	0	128.1	29.7	4.3
5	100	0	3	106.4	59.4	1.8
6	90	10	3	93.4	53.5	1.7
7	70	30	3	57.6	41.6	1.4
8	50	50	3	34.6	29.7	1.2
9	100	0	7	69.9	59.4	1.2
10	90	10	7	66.1	53.5	1.2
11	70	30	7	43.5	41.6	1.0
12	50	50	7	28.5	29.7	1.0

* Per cent of soil particles which diameters are smaller than 0.002 mm . From hydrometer tests it has been found that clay fraction of bentonite is 59.4 while it is nearly zero for lime.

** Ratio of the plasticity index to clay fraction as defined by Skempton (1953).

TABLE 3.8. Compaction Tests Results of the Soil Samples

Soil Type	Optimum Moisture Content (%)	Maximum Dry Density (kN/m^3)
1	41	10.25
2	40	10.40
3	37	11.38
4	29	12.85
5	36	11.70
6	35	11.95
7	29	13.03
8	24	14.28
9	35	11.75
10	35	11.90
11	28	13.20
12	24	14.30

For the concrete sections of the layered samples, *Normal Portland Cement (type I)* was used. In preliminary mix design calculations, approximate amounts were calculated as 220 kg water, 377 kg cement, and 1691 kg sand in 1 m^3 concrete. The calculations are shown below :

$$W = \alpha(10 - m) \tag{3.1}$$

and

$$\frac{W}{C} = [K/(aR)]^{1/2} \tag{3.2}$$

where

m = fineness modulus of the aggregate mixture

α = coefficient depending on consistency and type of aggregates

a = empirical constant related to testing conditions (average value is 6)

K = empirical constant related to cement quality

R = compressive strength of concrete specimens at 28 days of age

W = water content by weight in 1 m^3 of concrete

C = cement content by weight in 1 m^3 of concrete

Here in this case, m can be taken as 3.33 for fine aggregates, α as 33 for stiff consistency natural sand, K as 41 for normal portland cement, a as 6 being an average value and R as 20 MPa as desired compressive strength. With these considerations, the mix design proportions of concrete is calculated as

weight of water , W = 33 x (10-3.33) = 220.1 kg/m^3

water cement ratio, $\frac{W}{C}$ = [41/(6x20)]$^{1/2}$ = 0.584

weight of cement (C) = 0.581 x 220.1 = 376.9 kg/m^3

volume of cement = $376.9 \div 3.15$ = 119.7 lt/m^3 (dens. of cement=3.15 kg/m^3)

volume of air voids = 10 lt/m^3 (assumed)

volume of sand = 1000 - (220.1+119.7+10) = 650.2 lt/m^3

weight of sand = 2.60 x 650.2 = 1690.6 kg/m^3 (dens. of sand=2.60 kg/m^3)

3.3. The Direct Shear Tests

The reason for choosing the direct shear test method for this study is its practicality and its flexibility for preparing composite samples. Triaxial tests which are relatively more difficult and time consuming to perform, is certainly more versatile than direct shear tests. They have many advantages over the direct shear tests such as drainage control, measuring the pore pressure, determination of the principle stresses and failure plain, and obtaining the deformation at failure. In many cases, it might be more helpful to perform triaxial tests for the design problem under consideration. However, for soil-structure interface studies, triaxial test is not as suitable as direct shear test. Also, Bowles has stated that the soil parameters obtained by the direct shear tests, is as reliable as triaxial values [48].

Another reason for choosing the direct shear test method for this study is that samples can be prepared as two layers and shear stress parameters of the soil or at the interface of soil and concrete can easily be determined. As it is known, one of the drawbacks of the direct shear test method is the assumption that the actual failure surface is plane. However, this is not a drawback for a layered sample since the matter of investigation is the strength parameters just at the plane surface of the soil-concrete interface.

The principal purpose of the direct shear test is to examine the strength of the soil samples; it is not suitable for measuring the deformations. However the shearing deformations at failure is adequate to have an idea about the deformation for mobilization of full shearing force.

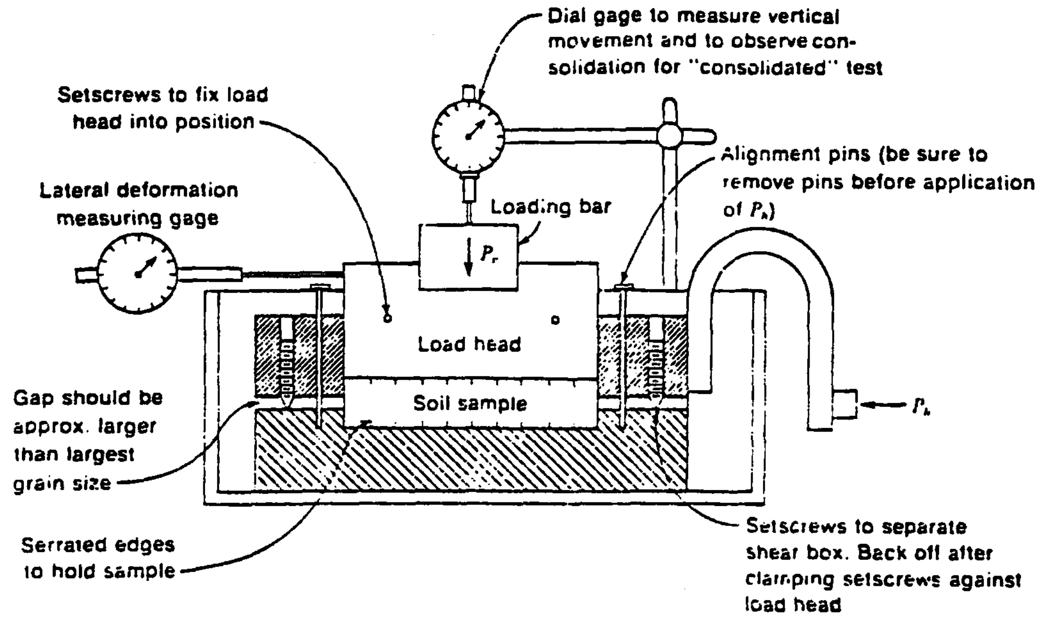


FIGURE 3.7. Schematic representation of the direct shear box and related equipment [48]

The essential futures of the direct shear box apparatus are shown in **Figure 3.7**. The stresses on the sample are, almost, those illustrated in **Figure 3.8.** and, because of the constraints of the box, the soil is forced to shear along the horizontal plane AB. For the detailed explanation of the test method, ASTM D3080 may be seen.

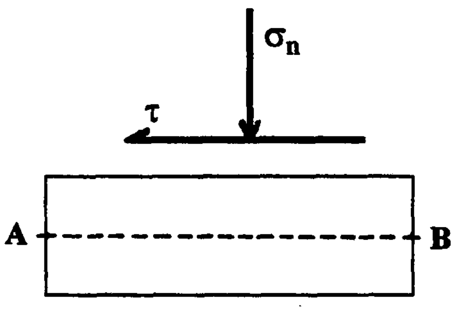


FIGURE 3.8. Schematic presentation of the stresses on the boundary of a direct shear sample

In this study, direct shear device of "Soiltest Inc." is used. Three sample for each set, is sheared by applying 25, 50, and 75 kgf normal loads which corresponds to approximately 95, 190, and 285 kPa normal stresses respectively, initially before slip deformations. The loading rate is chosen to be low, transversing horizontal deformation at a rate of 0.03 mm/s which causes failure in between 1 to 3 minutes approximately. The type of the direct shear test in the study can be considered to be analogous to *unconsolidated undrained triaxial test* because shear is initiated before the sample consolidates and excess pore pressure may develop during the test, due to the lack of the drainage control.

4. RESULTS AND DISCUSSION

4.1. Direct Shear Test Results

To investigate the change of shear strength parameters at the interface of lime-treated soil and concrete, direct shears tests were performed on the samples which are defined in Part 3 in detail. The comparison of the shear strength parameters at the interface of the concrete between lime-stabilized and non-stabilized soils, will reveal the contribution of lime-stabilization to the increase in pile friction.

Each individual shearing test was plotted as relative horizontal displacements of top and bottom platens versus corresponding shearing stresses. The readings were taken at every 0.2 mm horizontal displacement and the displacement rate was 0.03 mm/s.

For a better interpretation, results are grouped in three for the same mix proportions of bentonite and sand having 0, 3, and 7 per cent lime respectively. By this means, the improving effect of lime on the shear strength parameters at the slip surfaces of samples can be observed better.

4.1.1. Soil Samples

Each type of soil itself was sheared in the direct shear box for the determination of shear strength parameters of those soils and the effect of lime was observed. Three samples of each soil which are numbered and classified as in **Table 3.1** and **Table 3.2** were sheared under three different normal pressures for the representation of the failure envelops and thus, for the determination of the shear strength parameters c and ϕ of soils. The summary of results may be seen in **Table 4.1** . Besides, the deformations at failure for the samples of different soil types are illustrated 'n **Table 4.2** .

TABLE 4.1. Shear strength parameters of the soil samples obtained from the direct shear tests

Soil Type	Clay-Sand Proportion Bent. (%)	Sand (%)	Lime (%)	(Vert.Load=25kg) Normal Stress (kPa)	Max.Shear Stress (kPa)	(Vert.Load=50kg) Normal Stress (kPa)	Max.Shear Stress (kPa)	(Vert.Load=75kg) Normal Stress (kPa)	Max.Shear Stress (kPa)	c (kPa)	ϕ (Deg
1	100	0	0	101.4	74.8	204.5	78.5	308.1	86.3	74.1	3.2
2	90	10	0	99.3	68.5	198.6	73.9	299.1	82.0	68.0	3.9
3	70	30	0	100.5	63.8	201.9	78.2	300.4	82.4	56.1	5.3
4	50	50	0	98.5	60.8	197.8	76.0	295.5	86.4	48.6	7.4
5	100	0	3	101.0	154.5	203.6	193.3	305.5	214.3	128.1	16.3
6	90	10	3	100.5	149.0	202.8	181.5	306.8	212.7	118.7	17.2
7	70	30	3	102.3	150.3	203.6	186.6	305.5	221.7	115.0	19.4
8	50	50	3	99.7	130.9	203.6	198.3	308.7	252.7	74.4	30.2
9	100	0	7	102.3	193.5	207.6	236.0	310.7	264.3	161.2	18.8
10	90	10	7	101.4	185.2	203.6	226.0	306.8	270.2	143.3	22.5
11	70	30	7	102.7	183.8	207.1	253.0	312.0	296.3	133.2	28.2
12	50	50	7	101.4	184.0	202.8	242.2	304.2	329.4	106.4	35.6

TABLE 4.2. Slip Deformations of the soil samples at failure

Soil Type	First Sample (under 25kg Vert.load) (mm)	Second Sample (under 50kg vert.load) (mm)	Third Sample (under 75kg vert.load) (mm)
1	3.2	3.6	3.8
2	2.2	2.2	2.4
3	2.8	3.0	2.6
4	1.8	2.0	1.8
5	3.0	3.4	3.4
6	2.8	3.2	3.6
7	3.6	3.4	3.4
8	2.4	3.4	3.9
9	3.6	4.3	4.2
10	3.2	3.4	3.6
11	3.8	4.2	4.4
12	3.2	3.2	3.2

In **Figures** from **A.1** to **A.4** of the **Appendix,** shear strength versus deformation behavior of soil samples having different proportions of bentonite and sand and lime are illustrated.

In **Figure A.1**, shear stress versus deformation behavior of samples having 100 per cent bentonite and no sand in their mix proportions may be seen. When there is no lime in bentonite ultimate shear stresses at failure on the shearing plane are in the range of 70 and 90 kPa for all vertical stresses. When 3 per cent lime is added into bentonite, shearing stresses at failure are 155, 193, and 214 kPa for approximately 100, 200, and 300 kPa vertical stresses applied respectively. On the other hand, these stresses increase to 194, 236, and 264 kPa respectively when 7 per cent of lime is added to bentonite. There is no noticeable change in slip deformations at failure when 3 per cent lime is added into bentonite. The measurement of the slip deformations are on the order of 2.4 to 3.6 mm being generally greater for greater vertical stresses applied. However, the slip deformations are shifted to the range from 3.6 to 4.3 mm when 7 per cent lime is added addition into bentonite. This means greater deformations occur for the full mobilization of greater shearing stresses.

As shown in **Figure A.2**, when 10 per cent of sand is added to bentonite, there is no remarkable change in the characteristics of shear stress versus deformation graphs from the ones shown in **Figure A.1**. In general, very little decrease in the maximum shear stresses are observed for all samples having 0, 3, and 7 per cent lime under about 300 kPa vertical stress compared to the direct shear results of samples having only bentonite. Moreover, a slight decrease in slip deformations at failure as compared to samples of pure bentonite having 0, 3, and 7 per cent lime is also observed in shear stress versus deformation plots of these samples. Replacement of 10 per cent bentonite with sand does not cause remarkable changes in general behavior in response to the phenomenon of the lime treatment.

The shearing stress versus slip deformation behavior of 70 per cent bentonite and 30 per cent sand mixture with 0, 3, and 7 per cent added lime is given in **Figure A.3.** It may be inferred that the effect of lime on shear stresses are still the same as that of samples containing no sand and 10 per cent sand in their mix proportions. However, because of the increased amount of sand in the composition of the samples, the effect of applied vertical stress onto the increase of ultimate shearing stress can be seen more markedly, specially for 3 and 7 per cent of added lime as being greater for the latter. There is again slight shifting in ultimate relative slip deformations at the full mobilization of ultimate shearing stresses of samples with the addition of lime and this shifting increases when added lime amount is increased.

When the amount of sand is the same as bentonite, lime can still increase the ultimate shearing stresses of samples in a great extent as can be seen in **Figure A.4**. It is observed that the effect of vertical stress on the ultimate shear resistance is the most remarkable for these samples. Although there is not much difference in slip deformations of these samples than the deformations of the samples having 30 per cent sand in their composition which is discussed previously. More sudden decrease is observed in shearing stresses from ultimate to a residual value, for especially samples of 7 per cent added lime. Higher sand content (50 per cent) in bentonite sand mix proportion is interpreted as the cause of this observation.

In **Figures** from **A.5** to **A.8** of the **Appendix,** the failure envelops of soil samples obtained from the direct shear tests are illustrated. By fitting three points obtained from shearing three samples for each soil type under the pressure of about 100, 200, and 300 kPa to a line, shear strength parameters are obtained for each soil type.

Figure A.5 shows that when there is no sand added, in pure state without lime, shear resistance of bentonite depends primarily on the cohesion component of shear strength. The obtained value of cohesion in the samples of zero per cent sand and zero per cent lime is 74.1 while the ultimate strength under about 300 kPa pressure is only 86.3 kPa. The friction angle is found to be 3.2 for this case. When 3 per cent lime is added into bentonite, huge increases are faced in shearing strength of samples under all 100, 200, and 300 kPa vertical pressures. The friction angle increases to 16.3 and cohesion to 128.1. When the added amount of lime is increased to 7 per cent, shear strength increases more than 2 times of the initial strength when there is no lime. With 7 per cent lime, friction angle, ϕ, increases to 18.8 degrees and cohesion to 161.2 kPa.

When 10 per cent sand is added to bentonite, the same observations are made as shearing tests of samples without sand as illustrated in **Figure A.6**. Again shearing strength increases about 1.5 times with 3 per cent lime and more than about 2 times with 7 per cent lime as being greater under higher vertical pressures. In this case, the friction angle and cohesion increase from initial values of 3.8 degrees and 68 kPa to 17.2 degrees and 118.7 kPa with 3 per cent lime and to 22.5 and 143.3 kPa with 7 per cent lime. As expected, friction angle values are slightly larger and cohesions are smaller than samples of having no sand in their compositions.

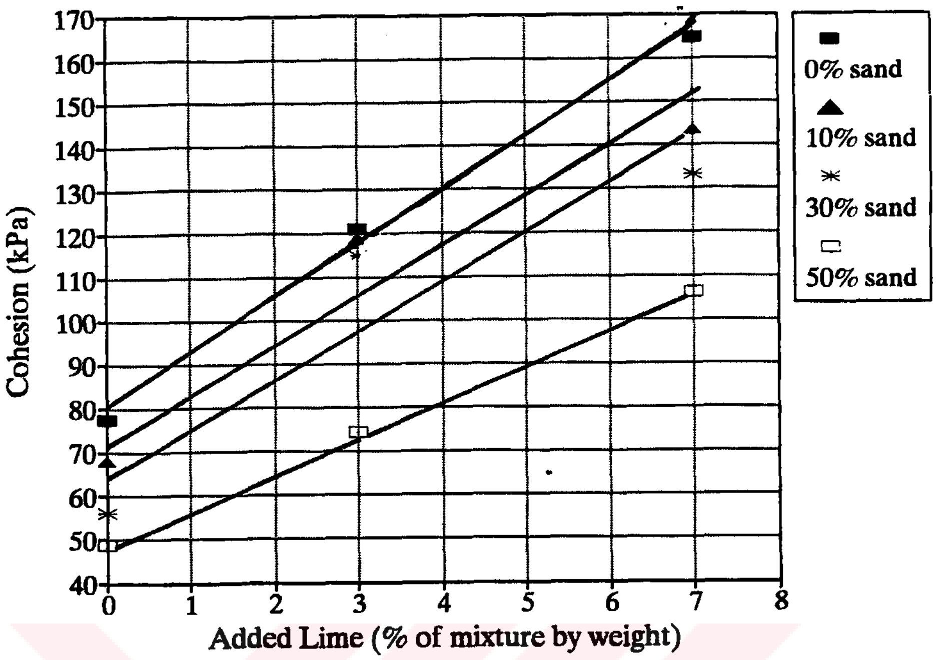
When the sand percentage is increased to 30, the effect of lime is the same on samples as the samples having 10 per cent sand. Friction angle and cohesion still increase with increasing amount of lime in bentonite sand mixture as can be seen in **Figure A.7** . When they are initially 5.3 degrees and 56.1 kPa, friction angle and cohesion increases to 19.4 degree and 115 kPa with 3 per cent lime and to 28.2 degrees and 106.4 kPa with 7 per cent lime respectively.

When 50 per cent sand is mixed with 50 per cent bentonite, lime still increases the shearing strengths excessively as shown in **Figure A.8** . However, in this case the shearing resistance depend more on friction component than on cohesion as expected. From failure envelops, friction angle and cohesion values are found to be 7.4 degrees and 48.6 kPa initially, and increased to 30.2 degrees and 74.4 kPa with 3 per cent lime, and to 35.6 degrees and 106.4 kPa with 7 per cent lime.

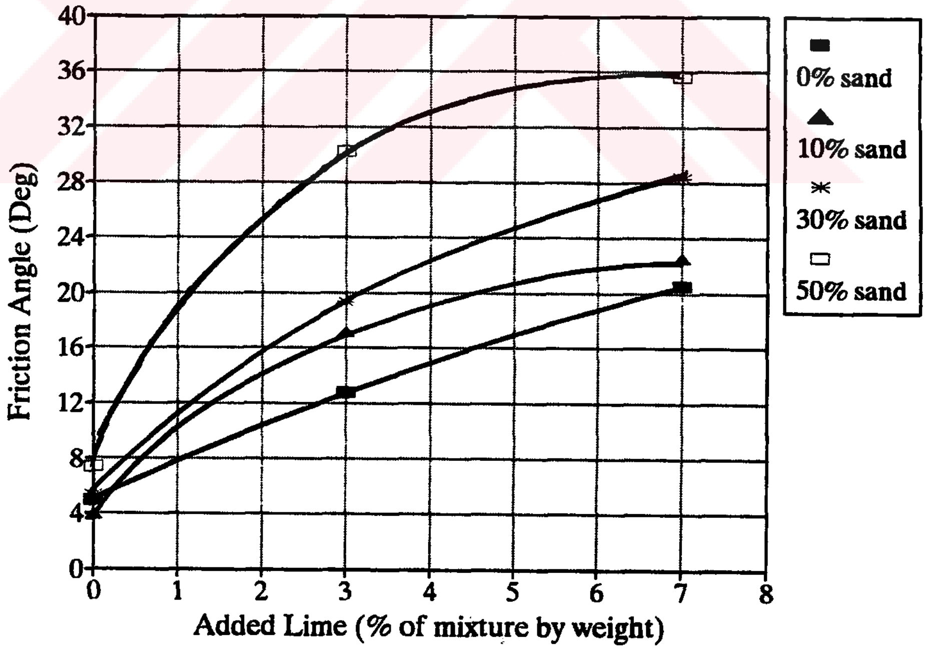
Figure 4.1a shows the relation between cohesion and amount of added lime for the soil samples containing different amounts of sand in bentonite-sand mixtures. It is observed from the figure that besides the cohesion values itself, the rates of growth also increases slightly with decreasing sand amounts. The ultimate cohesion occurs as 161.2 kPa when pure bentonite is treated with 7 per cent lime.

In **Figure 4.1b**, the graph of friction angle versus amount of added lime is illustrated. Unlike on cohesion values, the effect of lime on friction angles is increases with the increasing sand content in bentonite-sand mixture. When bentonite is treated with lime, friction component dominates in slip resistance for especially samples including greater sand content. This might be interpreted as the synergetic effect of treated bentonite and sand occurred after addition of lime. Furthermore, it is observed that, with an increase in lime content, the friction angle does not increase much after 3 per cent lime addition. The ultimate friction angle occurs as 35.6 degrees in samples of 50 per cent bentonite and 50 per cent sand mixtures when 7 per cent lime is added.

In **Figure 4.2a**, the change of cohesion values with changing sand content in bentonite-sand mixture is shown for soil samples for different added lime contents. As expected for each lime content, cohesion decreases with increasing sand content in compacted bentonite-sand mixture. On the other hand, as shown in **Figure 4.2b**, friction angles increase with increasing sand percentages. The rate of this increase is greater for higher lime percentages.

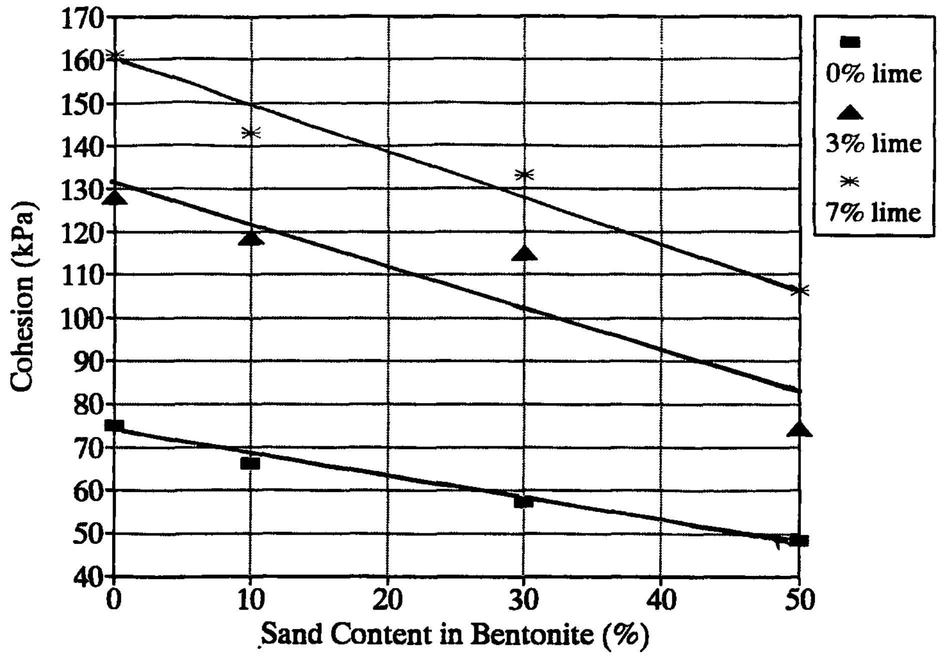


(a)

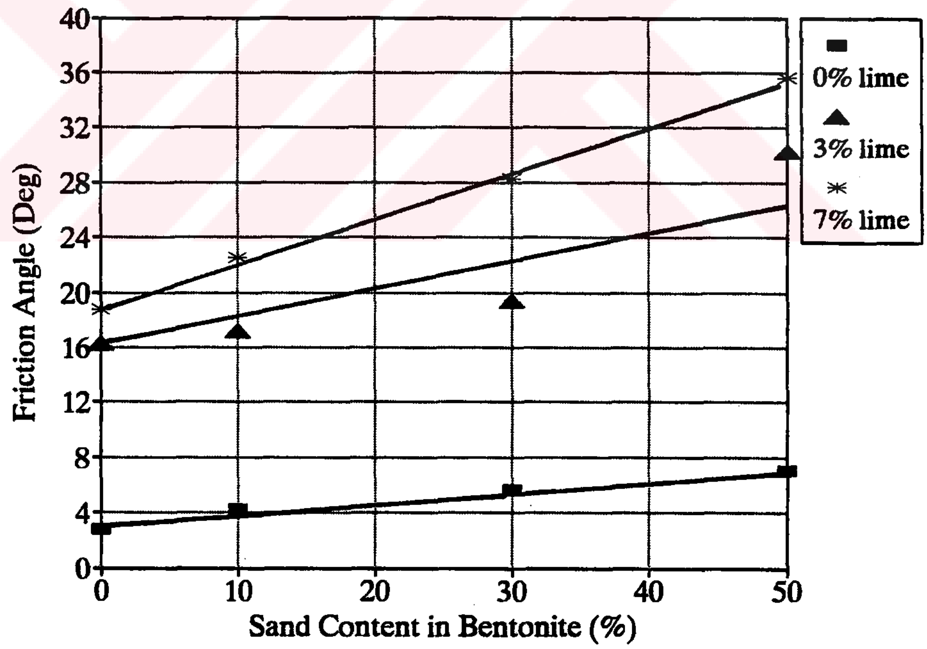


(b)

FIGURE 4.1. The change of shear strength parameters of soil samples with lime content a) cohesion vs added lime b) friction angle vs added lime



(a)



(b)

FIGURE 4.2. The change of shear strength parameters of soil samples with sand content a) cohesion vs sand content b) friction angle vs sand content

4.1.2. Layered Samples of Soil and Concrete

After the determination of the shear strength parameters of the soil sections of samples, the laboratory work is directed to the investigation of the shear strength parameters and their improvement with the addition of lime into soil sections at the interface of soil and concrete. Therefore, the direct shear tests on layered samples of soil and concrete were conducted. In order to decrease the effect of possible errors which might occur from the trial of shearing the layered samples just at the contact surface, the direct shear tests were performed as two sets for each type of soil part. Each set was based upon three direct shear tests under three different normal pressures as it was in the tests of soil sections alone and represented individual failure envelops of its own. The soil parameters obtained from two distinct sets are averaged for the final considerations of them. The shear strength parameters at the soil-concrete interface and the deformations at failure can be seen on **Table 4.2** and **Table 4.3** respectively.

In **Figures** from **A.9** to **A.16** of the **Appendix,** shear stress versus deformation behavior are shown for layered samples of concrete and bentonite-sand mixture into which 0, 3, and 7 per cent lime are added respectively. In both figures representing the results of samples having the same mix proportions, it is observed that lime can increase the shearing resistance at the soil concrete interface in a remarkable extent. Although there are small differences in corresponding plots of both direct shear sets of the same soil types, the overall shapes are very similar. The small differences might have resulted from forcing samples just at the surface of interface at the middle height.

When there is no lime in bentonite, shearing resistance is ranging from about 50 to a maximum of 80 kPa with the increasing vertical stresses applied as shown in **Figure A.9** and **A.10** . These stresses range from 70 to 120 kPa with 3 per cent added lime and increase to the range of 100 and 170 kPa with the addition of 7 per cent lime. Also, the slip deformations at the ultimate shear stresses increase for the occurrence of full mobilization of the shear resistance.

When 10 per cent sand is added to bentonite in the soil sections of the samples, the shear resistance and deformation properties does not alter much from the samples of 100 per cent bentonite. As shown in **Figure A.11** and **A.12,** the only noticeable change is that when 7 per cent lime is added higher shear resistance is obtained with larger corresponding slip deformations.

TABLE 4.3. Shear strength parameters of the layered samples of soil and concrete obtained from the direct shear tests

	Clay- Sand Proportion			(Vert. Load=25kg)		(Vert.Load=50kg)		(Vert.Load=75kg)					
				Normal	Max.Shear	Normal	ax.Shear	Normal	Max.Shear		**Average**		**Average**
Soil	Bent.	Sand	Lim	Stress	Stress	Stress	Stress	Stress	Stress	c_n	c_n	ϕ	ϕ
Type	(%)	(%)	(%)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kpa)	**(kPa)**	Deg	**Deg**
1	100	0	0	98.5	53.6	197.8	65.2	295.5	79.2	53.2	**53.6**	7.4	**5.6**
1	100	0	0	98.1	52.2	196.2	64.1	294.3	65.3	54.0		3.8	
2	90	10	0	97.7	53.2	195.4	64.4	293.1	73.3	43.4	**42.6**	5.9	**6.4**
2	90	10	0	98.5	53.6	197.8	65.8	295.5	77.4	41.8		6.9	
3	70	30	0	99.3	55.9	197.0	70.9	295.5	88.8	39.0	**40.3**	9.5	**10.2**
3	70	30	0	98.9	61.9	198.6	76.9	296.7	99.9	41.7		10.9	
4	50	50	0	99.7	59.1	200.3	84.2	299.1	107.6	35.2	**37.3**	13.7	**13.6**
4	50	50	0	99.7	63.6	199.4	86.2	299.1	111.6	39.4		13.5	
5	100	0	3	100.1	73.9	201.5	92.0	299.8	108.2	57.1	**53.2**	9.7	**11.3**
5	100	0	3	99.3	73.3	202.8	92.0	301.6	119.2	49.2		12.8	
6	90	10	3	101.0	73.6	200.3	89.0	304.2	120.8	47.6	**47.0**	13.1	**14.7**
6	90	10	3	99.7	76.0	199.4	104.3	302.9	135.6	46.4		16.3	
7	70	30	3	99.7	80.2	199.4	114.6	302.9	153.3	43.2	**43.7**	19.8	**20.5**
7	70	30	3	98.1	82.6	201.1	123.3	302.9	162.0	44.3		21.2	
8	50	50	3	99.7	85.6	201.9	133.7	302.9	175.2	42.7	**42.5**	23.8	**24.1**
8	50	50	3	99.7	87.7	201.9	130.1	302.9	179.5	42.2		24.3	
9	100	0	7	101.0	97.1	201.1	137.4	302.9	174.0	59.7	**61.7**	20.9	**19.8**
9	100	0	7	101.0	99.2	200.3	127.2	300.4	166.5	63.7		18.6	
10	90	10	7	102.7	108.1	204.5	152.1	308.1	203.7	59.8	**59.5**	25.0	**25.4**
10	90	10	7	102.3	108.2	205.4	156.8	306.8	206.8	59.3		25.7	
11	70	30	7	100.5	111.0	202.8	171.1	308.1	229.1	52.8	**53.2**	29.7	**30.0**
11	70	30	7	101.4	114.7	202.8	169.2	302.9	232.3	53.5		30.3	
12	50	50	7	101.0	127.0	205.4	207.4	308.1	285.3	50.5	**48.5**	37.4	**35.9**
12	50	50	7	101.4	116.8	203.6	184.7	306.8	257.6	46.5		34.4	

TABLE 4.4. Slip Deformations of the layered samples of soil and concrete at failure

Soil Type	First Sample (under 25kg vert.load) (mm)	Second Sample (under 50kg vert.load) (mm)	Third Sample (under 75kg vert.load) (mm)
1	1.8	2.0	1.8
1	1.6	1.6	1.6
2	1.4	1.4	1.4
2	1.8	2.0	1.8
3	2.2	1.8	1.8
3	2.0	2.2	2.0
4	2.4	2.6	2.4
4	2.4	2.4	2.4
5	2.6	2.9	2.5
5	2.2	3.2	2.8
6	3.0	2.6	3.2
6	2.4	2.4	3.0
7	2.4	2.4	3.0
7	1.6	2.8	3.0
8	2.4	3.0	3.0
8	2.4	3.0	3.0
9	3.0	2.8	3.0
9	3.0	2.6	2.6
10	3.8	3.6	3.8
10	3.6	3.8	3.6
11	2.8	3.2	3.8
11	3.2	3.2	3.0
12	3.0	3.8	3.8
12	3.2	3.4	3.6

When the sand content is increased to 30 per cent, the stress versus deformation curves become steeper and ultimate shearing stresses increase with increasing lime content added to the soil sections of the samples. As can be seen in **Figure A.13** and **A.14**, the ultimate shearing stresses for vertical pressures of about 100, 200, 300 kPa increase from initial values of about 60, 75, and 90 kPa to about 80, 120, and 160 with the addition of 3 per cent lime and rise to 110, 170, and 230 kPa with the addition of 7 per cent lime into soil sections. Increments in shear strength values are larger for larger vertical pressures. The deformations at the ultimate shearing stresses increase with the increasing lime contents.

When 50 per cent lime by weight is mixed with the same amount of bentonite, maximum shear resistance obtained for all samples with maximum corresponding slip deformations as illustrated in **Figures A.15** and **A.16.** The curves become steeper when 7 per cent lime is added into bentonite-sand mixture.

For layered samples of soil and concrete, the failure envelops obtained from the two sets of direct shear tests are plotted on the same graphs for a better interpretation. In **Figures** from **A.17** to **A.20** of the **Appendix** these graphs can be seen.

In **Figure A.17**, the failure envelops of two sets of samples including only bentonite with 0, 3, and 7 per cent lime are shown. It is observed that the initial value of friction angle is about 5.6 degrees average and adhesion 53.6 kPa. When 3 per cent lime is added, friction angle is doubled but adhesion does not change much. When the amount of lime is increased to 7 per cent, average friction angle reaches more than triple of its initial value. Besides, the adhesion value increases 15 per cent with the addition of 7 per cent lime comparing to the initial values when there is no lime.

From the failure envelops obtained from the direct shear test results of layered samples which soil sections have 10 per cent lime, it is observed that the effect of lime on shear strength parameters are similar to that on layered sample of concrete and only bentonite. However, the increase in both friction angle and adhesion is larger for these samples as shown in **Figure A.18** .While the friction angle and adhesion are 6.4 degrees and 42.6 kPa initially, they increase to 14.7 degrees and 47.0 kPa with the addition of 3 per cent lime and to 25.4 degrees and 59.5 kPa with the addition of 7 per cent lime into soil sections of the samples.

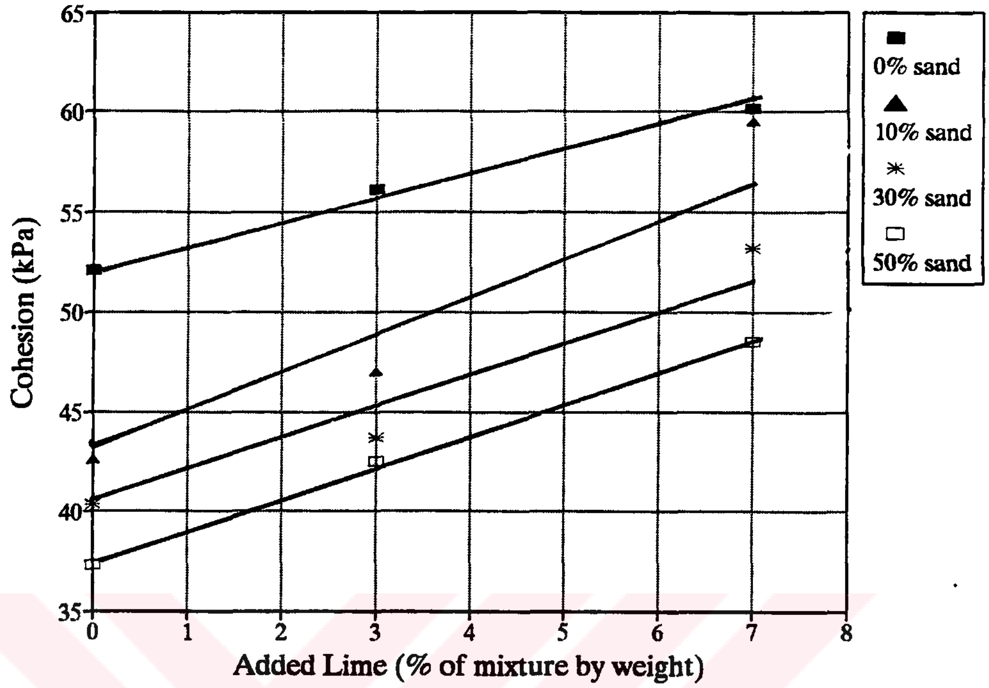
When the layered samples of concrete and soil having 70 per cent bentonite and 30 per cent sand in their composition of soil sections are sheared, it is observed that 3 per cent lime causes an increase of friction angle from 10.2 degrees to 20.5. When added lime is 7 per cent this value increases to 30.0 degrees. The effect of lime on adhesion in between concrete and soil is not so marked for these samples. The value of adhesion increases from 40.3 kPa to 43.7 kPa with 3 per cent lime and to 53.2 with 7 per cent lime. The failure envelops of these samples are illustrated in **Figure A.19**.

When there is 50 per cent sand in the soil sections of the samples, although there is not much increase in adhesion values of the soil concrete interface, the friction angle, δ, increases in a great extent. Resistance to slip primarily depend upon friction component of the shear strength. Therefore, strength is much greater under higher vertical pressures. The values of the average friction angles for 0, 3, and 7 per cent lime are 13.6, 24.1, and 35.9 degrees and the values of adhesion are 37.3, 42.5, and 48.5 respectively. The failure envelops of these samples may be seen in **Figure A.20.**

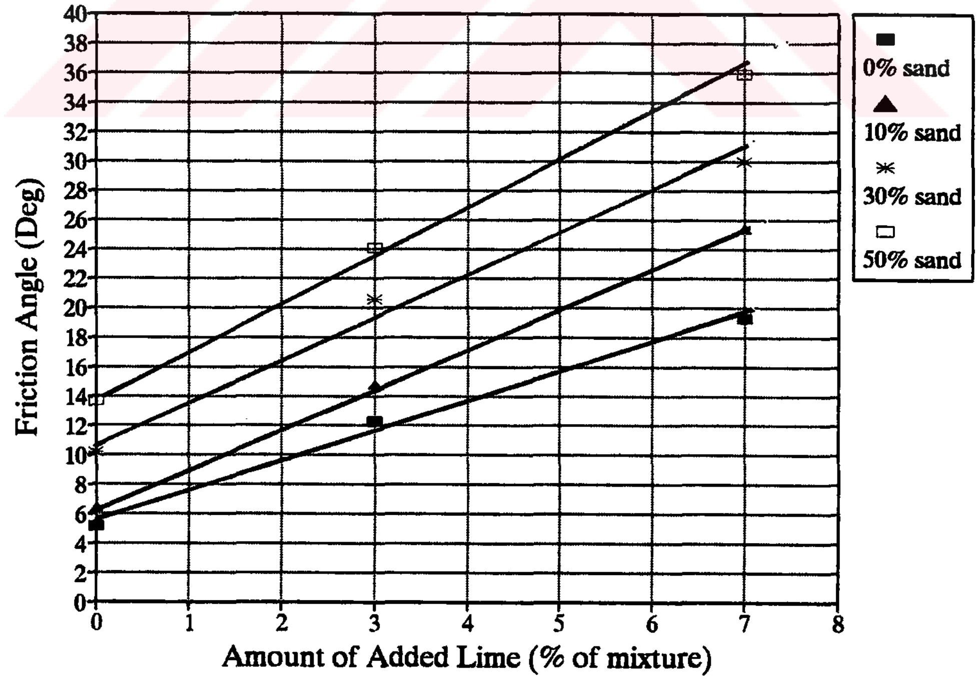
Figure 4.3a shows the change in adhesion at the interface of soil and concrete with changing amounts of lime added to the soil sections of the samples. The values are greater for greater bentonite contents in bentonite-sand mixtures. The maximum value is 61.7 kPa and obtained when there is 7 per cent lime in pure bentonite.

In **Figure 4.3b,** the behavior of friction angle versus amount of added lime is illustrated for different sand contents in bentonite-sand mixtures. As expected, friction angle values are higher when sand content is higher in bentonite-sand mixtures. The increment rate of friction angles does not alter much for different sand contents in bentonite. The maximum value is obtained when there is 7 per cent lime in soil sections of the samples which are composed of 50 per cent bentonite and 50 per cent sand..

In **Figures 4.4a** and **4.4b**, the change of adhesion and friction angle values are shown at the soil-concrete interface respectively. Adhesion values get larger and friction angles get smaller when the sand amount is decreased in soil sections.

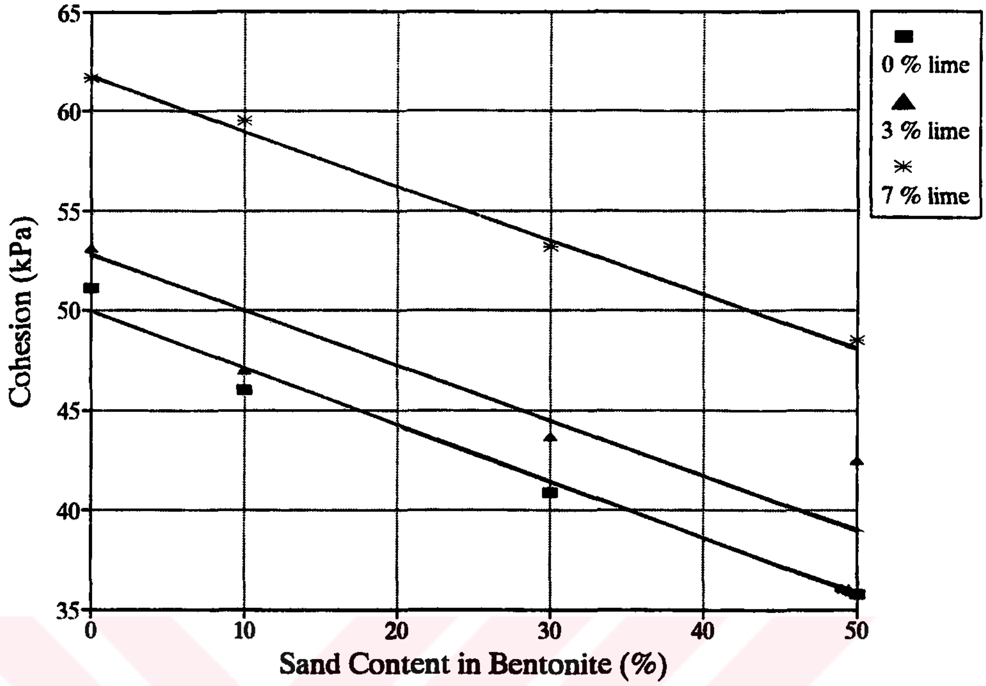


(a)

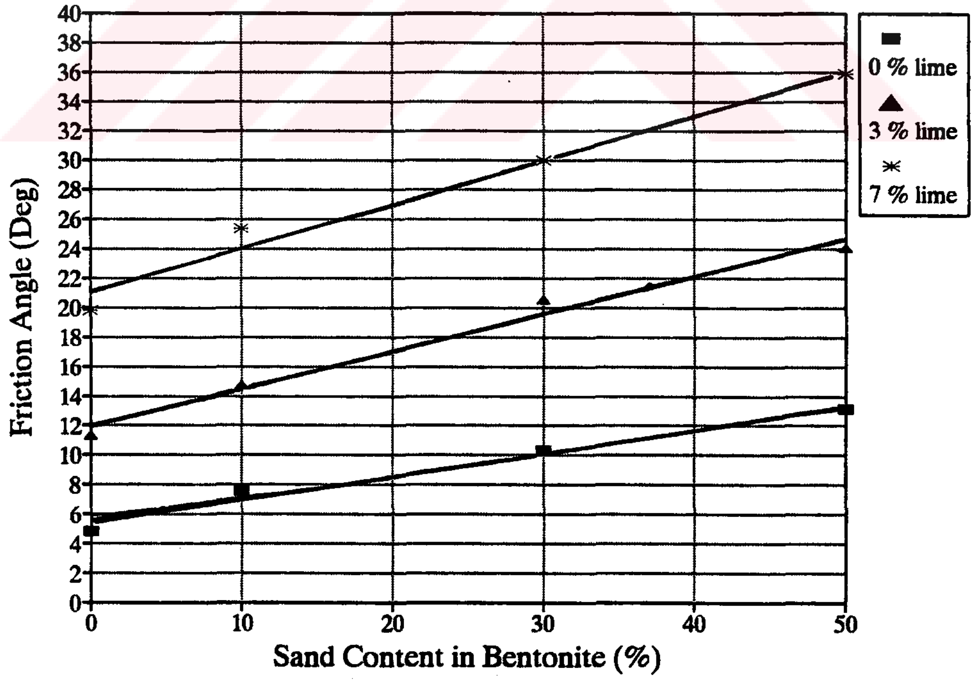


(b)

FIGURE 4.3. The change of shear strength parameters of layered samples with lime content a) cohesion vs added lime b) friction angle vs added lime



(a)



(b)

FIGURE 4.4. The change of shear strength parameters of layered samples with sand content a) cohesion vs sand content b) friction angle vs sand content

4.2. General Remarks

The results of direct shear tests have shown that lime increases the shear strength at the interface of clayey-soil and concrete structures considerably. However, the full mobilization of maximum shear resistance requires large slip deformations. When the deformations are considered as strains, it is noticed that slip deformations of about 3.8 mm at the ultimate shear stresses correspond to large values up to 7.5 per cent strains. Nevertheless, there are excessive boundary effects and large stress concentrations at the shearing surface of relatively small sized samples. Therefore, the strains should be disregarded and the deformation measurements themselves should be taken into consideration.

Vesic (1977) [32] has stated that the displacement needed to mobilize skin resistance is small, not exceeding 10 mm, regardless of the soil and pile type and pile dimensions. Some investigators (Whitake and Cooke, 1966 ; AISI, 1975) have also expressed that the slip deformation to develop maximum skin resistance is on the order of 5 to 10 mm and relatively independent of shaft diameter and embedment length but may depend upon soil parameters. In addition, Skempton (1959) [43] did also state that full shaft adhesion was mobilized after a settlement of 10 mm in clay. From these expressions it can be inferred that for any alteration of soil properties and for any increase in shearing resistance, the full mobilization of the shaft resistance does not exceed 10 mm. When deformations are regarded, it is seen that the ultimate value does not exceed 3.8 mm, being reasonable with respect to above statements.

Another remark on the direct shear test results is that the friction angle, δ, between soil and concrete is found as greater than the friction angle of soil, ϕ, when there is no lime. This indicates that the rough surface of concrete contributes much to the increase in friction angle. However, many investigators take δ maximum as the value of ϕ. In this regard, Anderson *et al.* (1985) [42] have stated that when there is no delay between boring and concrete casting, δ is greater than ϕ but when there is one day delay they are about the same because of the stress relaxation of the soil. They state that failure may occur at the smoother soil-pile interface instead of a thin zone near to pile shaft when there is long delay between boring and concrete casting.

When lime is added, the shear strength parameters of soil samples increases more than that of layered samples. The reduction coefficient, α, of adhesion decreases from initial values of 0.60-0.75 to the values of about 0.40-0.45 with the addition of 7 per cent lime. Friction angles of soil samples increase when 3 per cent lime is added, and remains nearly the same when the lime content is increased to 7 per cent. This shows that the first 3 per cent lime affects soil samples much more than the extra added percentages of lime.

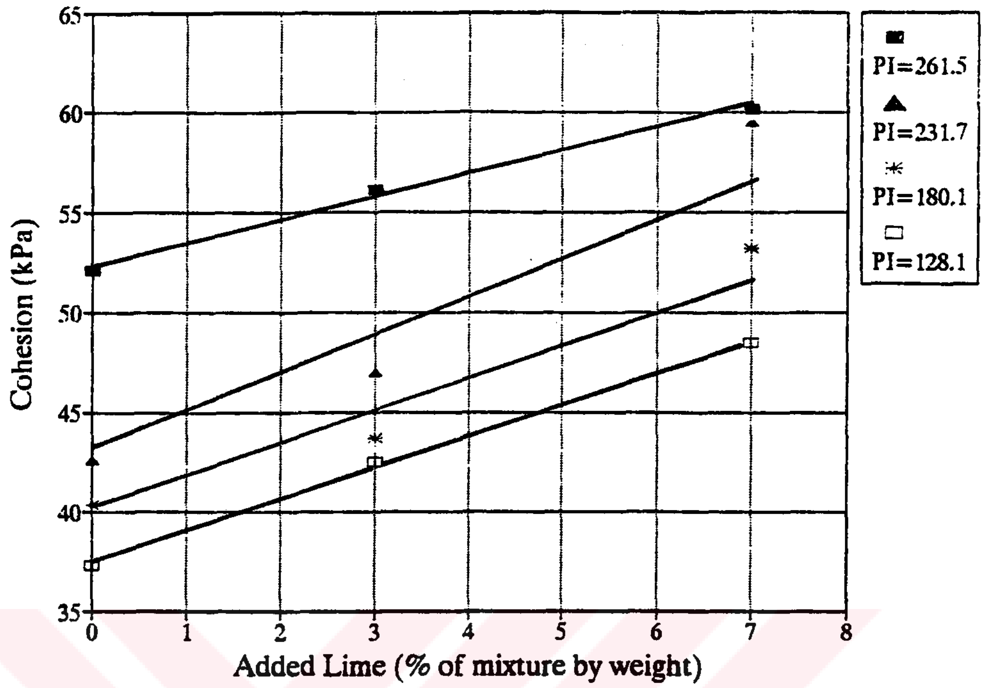
The ultimate shear resistance of soil samples increase more than the ultimate shear resistance of layered samples with the addition of lime. This means that when no lime treatment is made failure may occur both in a zone near to pile shaft or just at the interface but with the addition of lime it will occur most probably at the pile-soil interface.

A final remark on the results of direct shear test is about the physical observations on the layered sample after shearing. Succeeding the failure of test samples, it is observed that, a thin layer of soil remains on the concrete part for the samples having 3, and specially, 7 per cent lime in their soil sections. This situation might occur because of the large stress concentration at the soil-concrete interface of these samples. As discussed before, the strains are different at every point on the failure surface and there are serious stress concentrations at the sample boundaries. Therefore, for higher shear stresses, an uncontrolled rotation of principle planes and principle stresses might occur between the start of the test and failure. Thus, the failure plane shifts towards soil which is less in strength than concrete.

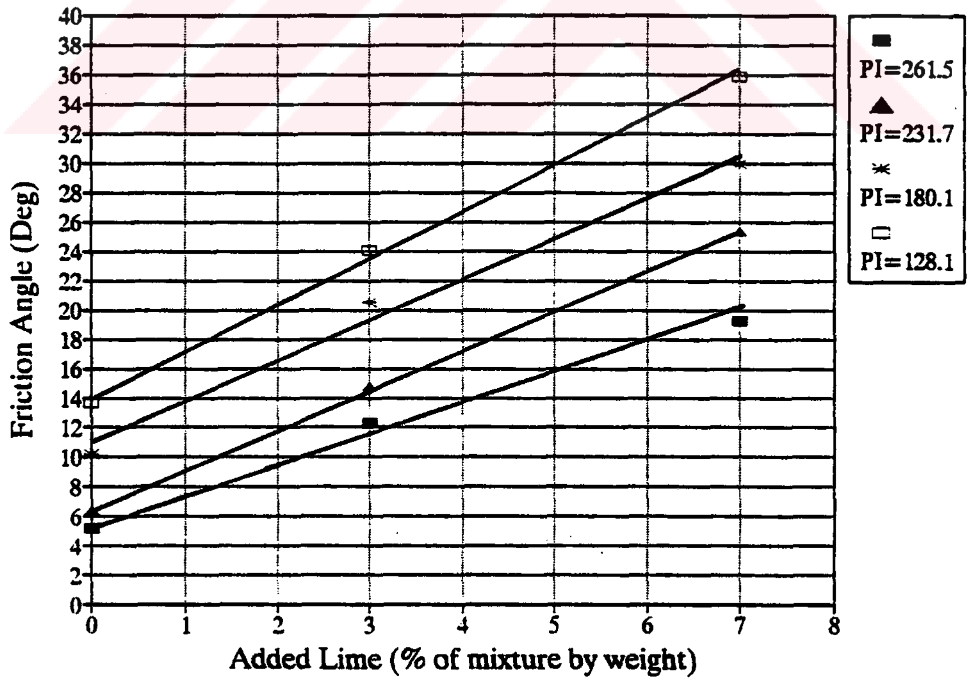
4.3. Application of the Method for Cast-in-situ Concrete Piles

From the direct shear test results, the shear strength parameters at the soil-concrete interface are obtained. These results are also representative for cast-in-situ concrete piles or drilled piers embedded into clayey-soil. By taking plasticity index values as a characteristic variable related to the soil type, the variation of the adhesion and friction angle values with added amount of lime into soil, are plotted in **Figures 4.5a** and **4.5b** respectively.

As can be seen in **Figure 4.5a**, the change of friction angle with the lime amount used for treatment shows almost a linear relationship for all of the changing PI values. Friction angle values are higher for lower PI values of soil as expected. On the other hand, from **Figure 4.5b**, it is observed that the adhesion values at the soil concrete interface is greater for higher PI values.



(a)



(b)

FIGURE 4.5. The change of shear strength parameters at the pile shaft with lime content a) cohesion vs added lime b) friction angle vs added lime

As given in **Equation 2.15**, Tomlinson (1971) [34] has proposed "the α method" for the calculation of the skin resistance of a pile. Rewriting the equation that Tomlinson derived for unit skin resistance of piles

$$f_s = \alpha\, c + \quad K\, tan\delta \qquad (2.15)$$

of which the definitions of the terms are given before, it can be seen that the adhesion, c_a, is a multiple of cohesion, c. For clayey-soil, Tomlinson [34] recommends a value of 0.40 for the multiplication constant, α. It is also observed from **Equation 2.15** that the effective vertical stresses in the soil are transformed to the pile shaft in rate of lateral earth pressure coefficient K, and affect the friction component of the shaft resistance.

For a sample application of the method for cast-in-situ concrete piles, the values of adhesion, c_a, and friction angle, δ, between pile and soil, can be used directly from test results together with cohesion, c, and friction angle, ϕ, of soil itself. For illustrating the direct shear test results, the unit shaft resistance of a pile for different penetration depths in clayey-soil is calculated for 0, 3, or 7 per cent lime which is provided for the treatment of the soil in a close zone around the pile shaft. Since the cohesion and friction angle values of the direct shear test samples are changing with the sand content in bentonite-sand mixture, the plasticity index values are taken as a sign of these variations. Therefore, unit shaft resistance is estimated for different plasticity index values of soil. By taking K as K_o, considering the state at rest and taking the effective unit weight of the soil 8 kN/m^3 being an average value for clayey-soil, the results are obtained as illustrated on **Table 4.5**. In this table, the per cent increase in unit shaft resistance is also shown.

In **Table 4.5**, it is observed that when the pile length increases, the growth in unit shaft resistance is higher. Also, the increments in unit shaft resistance are much more noticeable when the added lime content is greater. For instance, for a 10 m long pile, the increment in unit shaft resistance is as much as 25-30 per cent of the initial value when 3 per cent lime is added into soil. However, for a pile with embedment depth of 30 m, the unit shaft resistance increases to twice the initial value by the treatment of soil with 7 per cent lime.

For illustrating the convenience of real applications better, the skin resistance capacity of a pile with a diameter of 50 cm is shown in **Table 4.5.** Besides, for a comparative study, the necessary pile diameters for supplying the same strengths without lime treatment of soil are also given **Table 4.5**.

TABLE 4.5. Shaft resistance of a pile with 50 cm diameter and the necessary diameter for the same resistance when there no lime treatment

Emb. Depth (m)	PI	Added Lime (%)	c_n kPa	γ' of soil kN/m³	q kPa	ϕ of soil Deg	K	δ at interf Deg	f_s kPa	per cent increase in f_s	Q_s for D=50cm kN	D for no lime treat.-cm
10	261.5	0	53.6	8	40	3.2	0.944	5.6	57.3	—	900.1	50
		3	53.2	8	40	3.2	0.944	11.3	60.7	6.0	954.2	53
		7	61.7	8	40	3.2	0.944	19.8	75.3	31.4	1182.8	66
	231.7	0	42.6	8	40	3.9	0.932	6.4	46.8	—	734.8	50
		3	47.0	8	40	3.9	0.932	14.7	56.8	21.4	981.9	61
		7	59.5	8	40	3.9	0.932	25.4	77.2	65.0	1212.7	83
	180.1	0	40.3	8	40	5.3	0.908	10.2	46.8	—	735.6	50
		3	43.7	8	40	5.3	0.908	20.5	57.3	22.3	899.7	61
		7	53.2	8	40	5.3	0.908	30.0	74.2	58.4	1164.9	79
	128.1	0	37.3	8	40	7.4	0.871	13.6	45.7	—	718.3	50
		3	42.5	8	40	7.4	0.871	24.1	58.1	27.0	912.4	64
		7	48.5	8	40	7.4	0.871	35.9	73.7	61.2	1158.1	81
20	261.5	0	53.6	8	80	3.2	0.944	5.6	61.0	—	1916.6	50
		3	53.2	8	80	3.2	0.944	11.3	68.3	11.9	2145.5	56
		7	61.7	8	80	3.2	0.944	19.8	88.9	45.7	2792.7	73
	231.7	0	42.6	8	80	3.9	0.932	6.4	51.0	—	1601.1	50
		3	47.0	8	80	3.9	0.932	14.7	66.6	30.6	2091.0	65
		7	59.5	8	80	3.9	0.932	25.4	94.9	86.2	2981.5	93
	180.1	0	40.3	8	80	5.3	0.908	10.2	53.4	—	1676.5	50
		3	43.7	8	80	5.3	0.908	20.5	70.8	32.8	2225.8	66
		7	53.2	8	80	5.3	0.908	30.0	95.1	78.2	2988.3	89
	128.1	0	37.3	8	80	7.4	0.871	13.6	54.2	—	1701.5	50
		3	42.5	8	80	7.4	0.871	24.1	73.7	36.0	2314.6	68
		7	48.5	8	80	7.4	0.871	35.9	99.0	82.7	3018.7	91
30	261.5	0	53.6	8	120	3.2	0.944	5.6	64.7	—	3049.4	50
		3	53.2	8	120	3.2	0.944	11.3	75.8	17.2	3573.9	59
		7	61.7	8	120	3.2	0.944	19.8	102.5	58.4	4829.8	79
	231.7	0	42.6	8	120	3.9	0.932	6.4	55.1	—	2598.6	50
		3	47.0	8	120	3.9	0.932	14.7	76.3	38.4	3597.4	69
		7	59.5	8	120	3.9	0.932	25.4	112.6	104.2	5306.4	102
	180.1	0	40.3	8	120	5.3	0.908	10.2	59.9	—	2822.6	50
		3	43.7	8	120	5.3	0.908	20.5	84.4	40.9	3978.3	70
		7	53.2	8	120	5.3	0.908	30.0	116.1	93.8	5470.3	97
	128.1	0	37.3	8	120	7.4	0.871	13.6	62.6	—	2949.6	50
		3	42.5	8	120	7.4	0.871	24.1	89.3	42.6	4206.5	71
		7	48.5	8	120	7.4	0.871	35.9	124.2	98.4	5851.7	99

In order to compare the costs of making a lime slurry treatment or not for obtaining the same skin resistance capacities, from **Table 4.5,** a single pile of length 20 m is chosen arbitrarily for the case when it is embedded into soil having the PI value of 180.1 . With the 7 per cent of lime adsorbtion through a zone around the pile shaft, 2988.3 kN skin resistance is obtained when the diameter is 50 cm . The cost of such a pile is about 610 US Dollars including the cost of lime slurry. For this calculation, the cost of lime is taken as 1 US Dollars for 25 kg lime which is the current cost in Turkiye. In unit m^3 of water, 100 kg lime is considered to be mixed. The total cost of the slurry treatment is estimated as 50 US Dollars roughly for the whole length of 20 m and added to the cost of pile. For a cast-in-situ concrete pile of diameter 50 cm, the unit cost for of construction has been estimated as approximately 28 US Dollars per each meter. On the other hand, the necessary pile diameter for obtaining the same skin resistance capacity is 89 cm without lime treatment for the same embedment length and soil type. The approximate cost of making such a pile is 1160 US Dollars (the unit price is 58 US Dollars per meter). This shows that by the treatment of the soil in a thin zone around the pile shaft, the same shear strength as the strength of a 89 cm diameter pile is obtained with 50 cm diameter and approximately 550 US Dollars is saved for a single pile of 20 m length.

The skin resistance capacity of the pile in consideration increases markedly with the addition of lime into soil. When the amount of lime is increased, the growth is much more noticeable. When the point resistance is neglected, the load carrying capacity of the pile, for different embedment lengths and soil types, increases as the same rate with the growth in unit shaft resistance. Similarly, the necessary pile diameters for obtaining the same capacity when there is no lime treatment is increased in the same order.

The questionable point here is that in the direct shear tests, the compacted samples in which lime is mixed prior to compaction were sheared. However, in the field, diffusion mechanism will be benefited for the soil improvement. The sufficient percentage of lime and the necessary time period for providing 3 or 7 per cent adsorption into soil are not tested in this study. For making estimation, similar methods and related studies in the literature are utilized.

The solubility of lime in water is very low (0.634 kg/m^3 at 25 C°,) but the slurry is considered to have lime particles in suspension for abundance of lime. In bentonite slurries, the amount of bentonite added is generally about 50 kg in 1 m^3 of water. Although the aims are very different, the methods are similar for making bentonite or lime slurries. Thus, for making a lime slurry in muddy state, with a conservative approach, 100 kg lime can be considered to be added into 1 m^3 of water in the bore-hole. However, this point needs further determination.

With sufficient amount of lime, Stocker [28] states that 7 per cent lime was acquired in one day, by 15 per cent lime stabilized clay cores. Despite the difference of initial lime stabilization, this statement explains the question of how much lime is adsorbed by diffusion to some extent. As explained before, the lime stabilization in a thin zone around the pile shaft is sufficient for increasing the unit shaft resistance markedly.

If the application is performed in saturated soil where the ground water table is near to the ground surface, the diffusion will occur through the saturated soil matrix. Schakelford and Daniel [27] found that the effective diffusion coefficients of clay types that they have tested are within a narrow range of 4.0×10^{-10} m^2/s to 2.0×10^{-9} m^2/s.

5. CONCLUSIONS

The effect of lime stabilization of soil on shear strength parameters at the interface of concrete was investigated using direct shear tests. The conclusions given below are restricted to the testing method selected, however, this study has been initiated as the first phase of a potential original application. Although field trial is not done at this phase of the study, the results of the direct shear tests are extrapolated for field applications.

1) The shear strength parameters between clayey-soil and concrete increase markedly with the lime stabilization of soil at the soil-concrete interface.

2) The increase in shear strength parameters are more noticeable exceeding 100 per cent when the lime content in soil is increased to 7 per cent by weight.

3) Because of the increase in shear strength parameters at the interface of clayey-soil and concrete with the addition of lime into soil, slip deformations at failure also increase. However, as many investigators explained, the deformations necessary for the full mobilization of the skin resistance do not exceed 10 mm regardless of the soil and pile type and pile dimensions. Therefore, the increase in slip deformations can not be considered as a drawback for the real applications.

4) The effect of lime stabilization on shear strength parameters of soil is much greater than it is on shear strength parameters at soil-concrete interface. Therefore, with the field application of the method, the failure is expected just at the interface of soil and concrete or at a remoter plane from the pile shaft in soil which sufficient amount of lime cannot reach through diffusion.

5) With the simulation of the test results to real case applications, it is found that much increase in the unit shaft resistance can be obtained by the help of lime stabilization. Since this leads to decrease in pile dimensions for supplying the desired skin resistance, a great amount of cost savings can be achieved.

Although the results of this study are very encouraging for the field applications, some alternatives must also be considered such as the addition of the fly-ash together with lime and/or usage of unslaked lime instead of hydrated lime to increase the effectiveness of the method. More experiments are needed for determination of this point.

In this study, cast-in-situ concrete piles or drilled piers were considered to be the suitable types for the method proposed and simulation was done for these types. However, other types of bored piles such as bored precast concrete piles can also be taken into consideration for the application of the method. Because, addition of lime into soil does not only provide better bonding with concrete but also improves the soil properties.

Finally, for convenience, compacted samples are used in this study, and lime is added prior to compaction. However, the recommended application bases on the diffusion mechanism from the bore-hole through the neighboring soil. Although correlation is mentioned from the literature, this point still needs further determination. For obtaining stronger proof and for standardization of the method, a wide range of field tests must be conducted in lights of this study.

APPENDIX

The direct shear test results are illustrated in **Figures** from **A.1** to **A.20.**

In **Figures** from **A.1** to **A.4,** shear stress versus deformation behavior of soil samples having different proportions of bentonite and sand and lime are shown.

In **Figures** from **A.5** to **A.8,** the failure envelops of soil samples may be seen.

In **Figures** from **A.9** to **A.16,** shear stress versus deformation behavior are shown for layered samples of concrete and bentonite-sand mixture into which 0, 3, and 7 per cent lime are added respectively.

In **Figures** from **A.17** to **A.20,** the failure envelops of soil samples are illustrated.

(a)

(b)

(c)

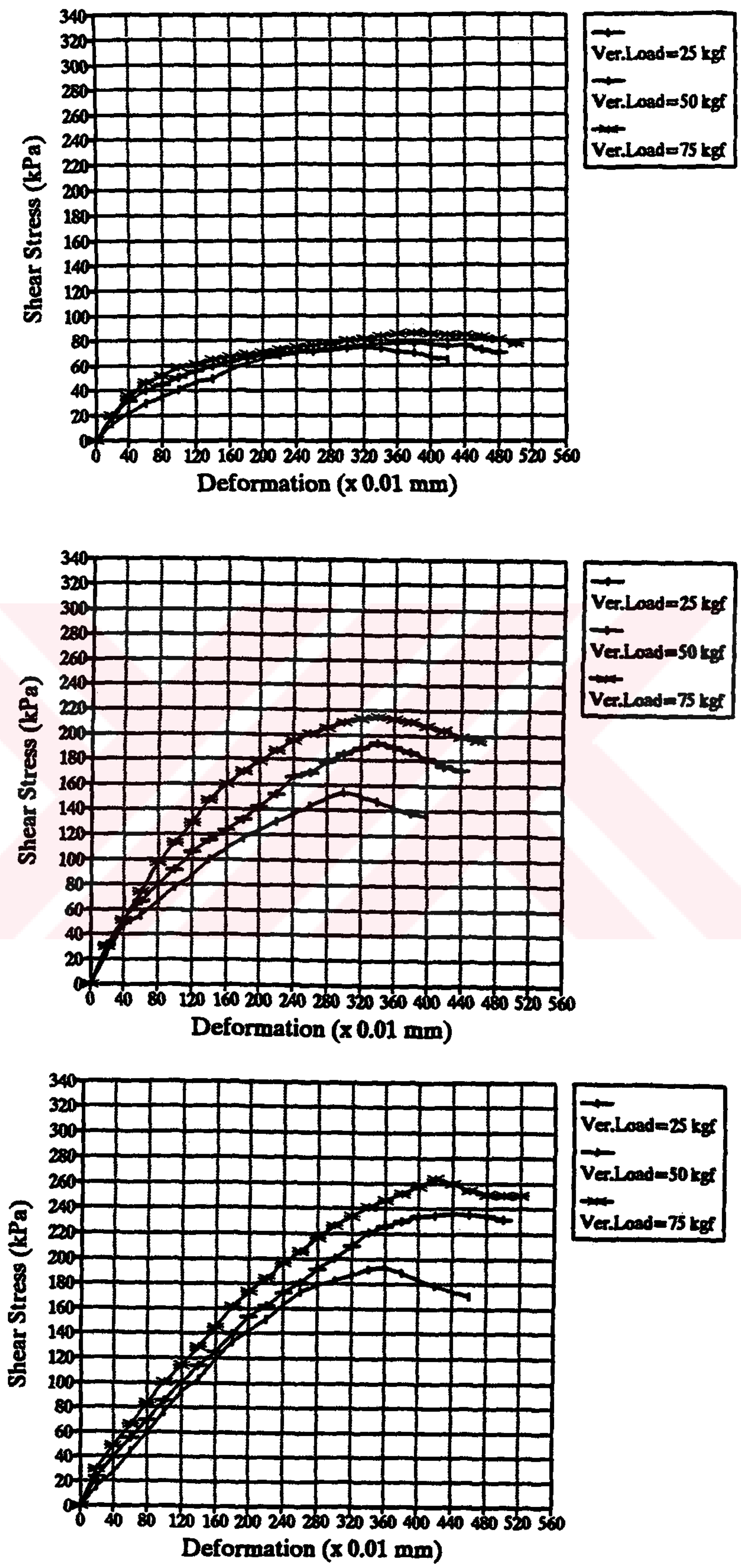


FIGURE A.1. Shear Strength vs deformation behavior of soil samples which is composed of 100 % bentonite a) with no lime, b) with 3 % lime, c) with 7 % lime

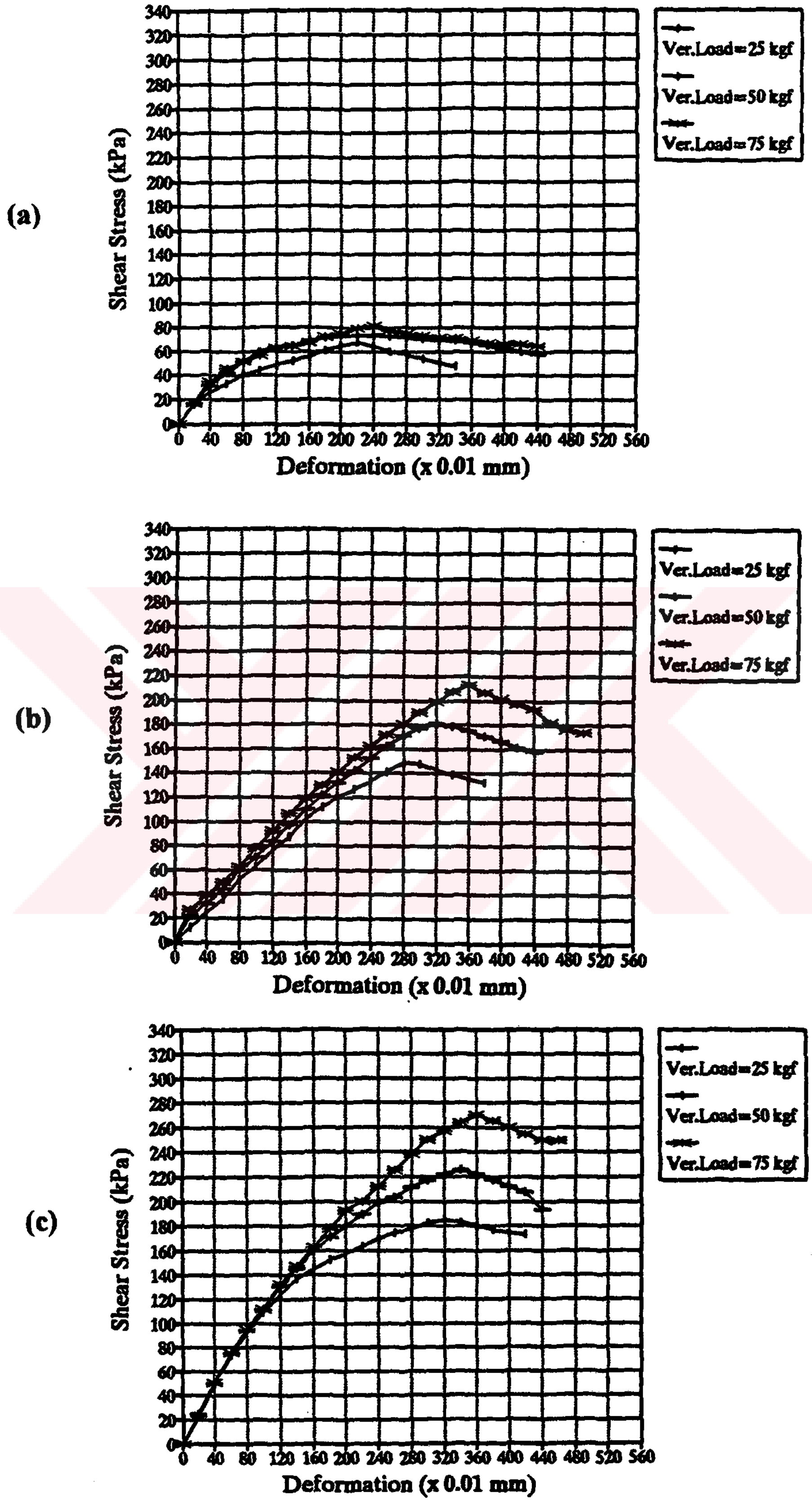


FIGURE A.2. Shear Strength vs deformation behavior of soil samples which is composed of 10% sand and 90% bent.a)with no lime, b)with 3% lime, c)with 7% lime

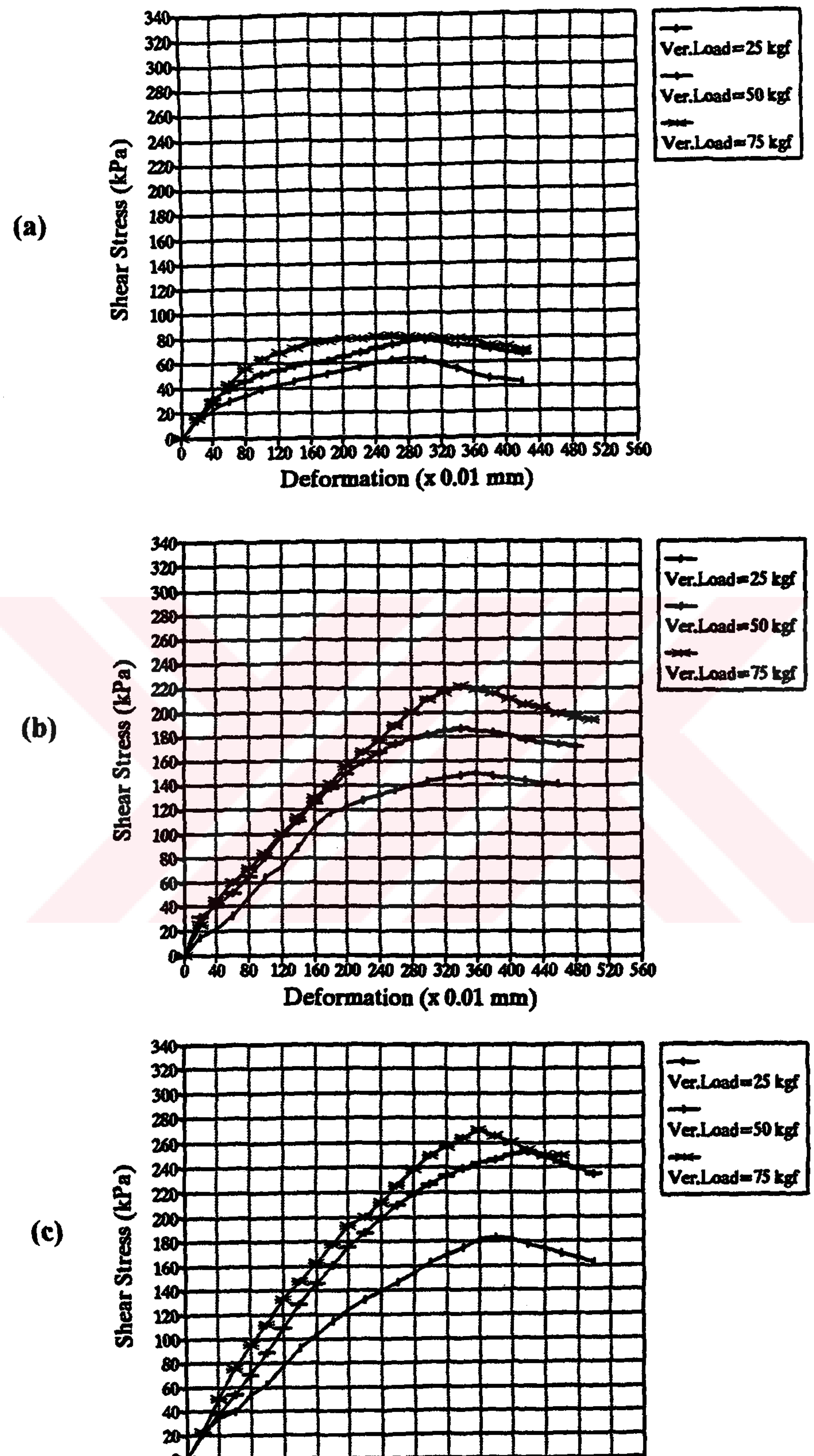


FIGURE A.3. Shear Strength vs deformation behavior of soil samples which is composed of 30% sand and 70% bent.a)with no lime, b)with 3% lime, c)with 7% lime

(a)

(b)

(c)

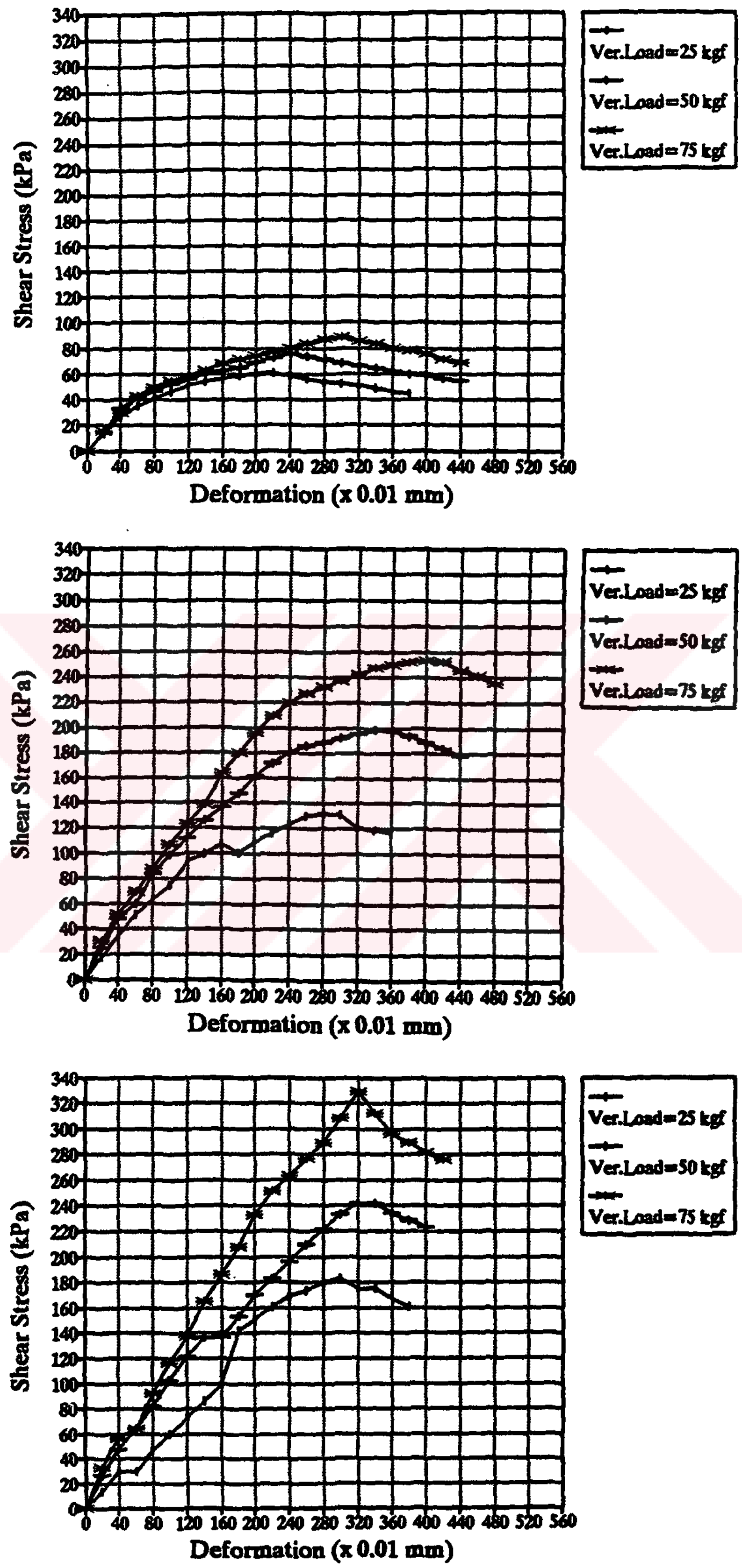


FIGURE A.4. Shear Strength vs deformation behavior of soil samples which is composed of 50% sand and 50% bent.a)with no lime, b)with 3% lime, c)with 7% lime

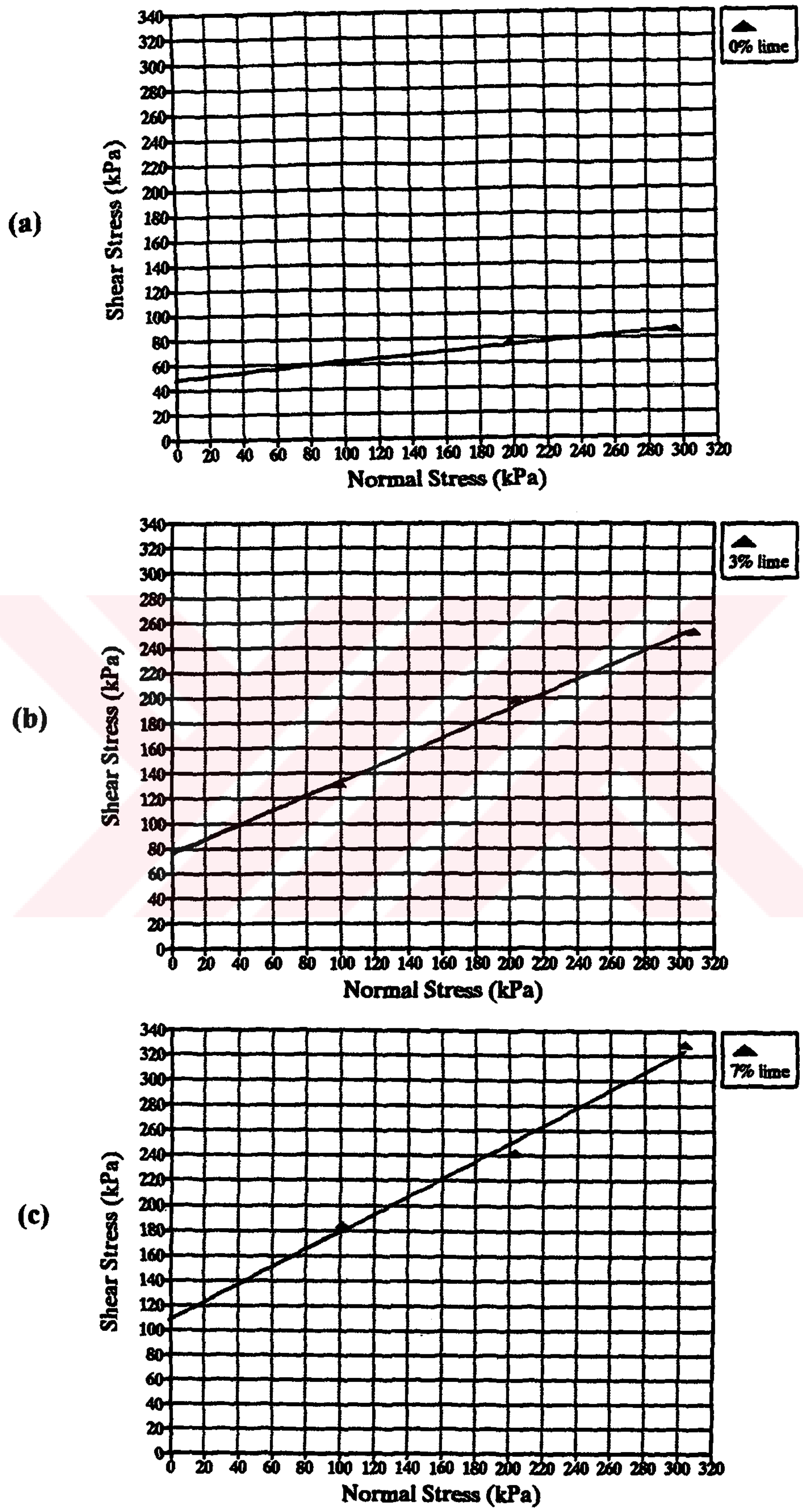
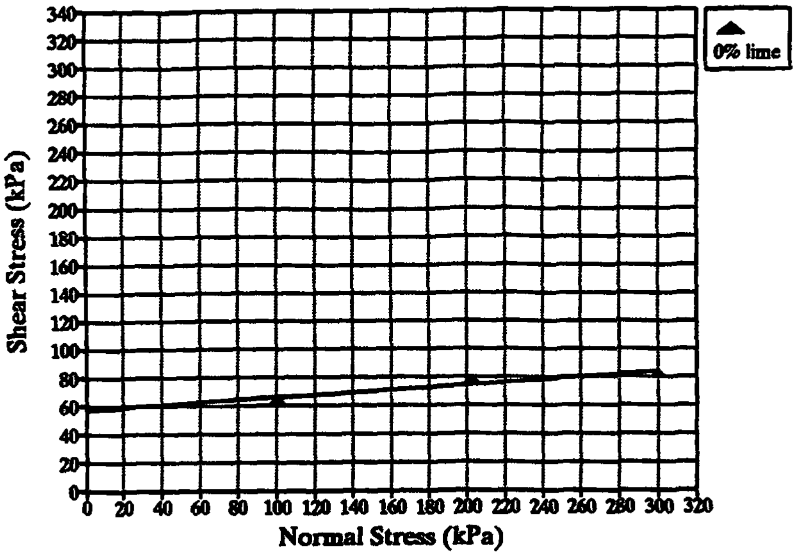
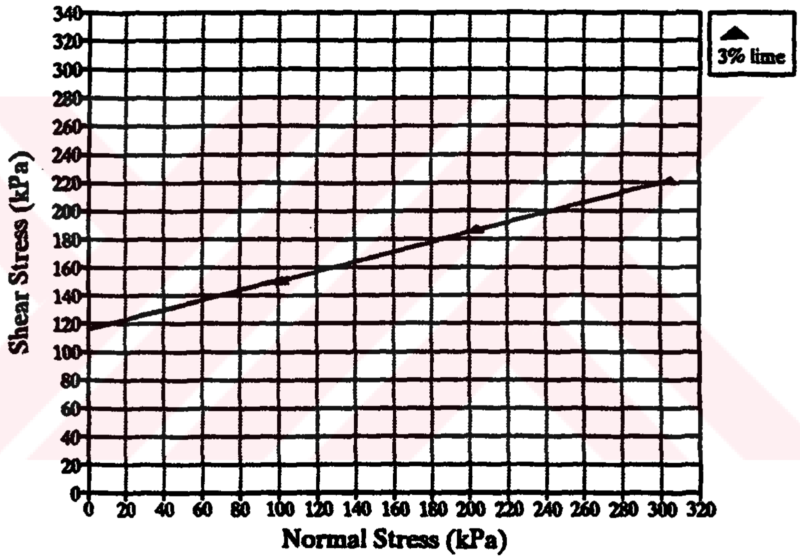


FIGURE A.5. Failure envelops of soil samples which is composed of 100% bentonite a) with no lime, b) with 3% lime, c) with 7% lime

(a)



(b)



(c)

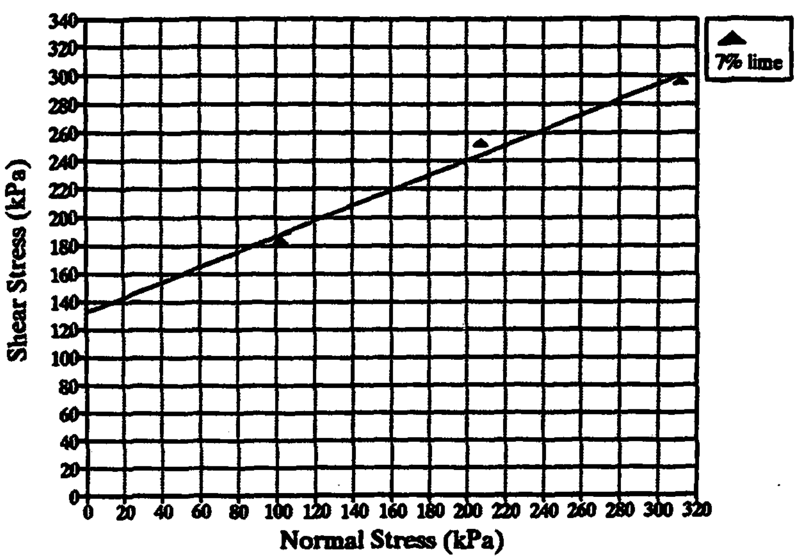


FIGURE A.6. Failure envelops of soil samples which is composed of 10% sand and 90% bentonite a) with no lime, b) with 3% lime, c) with 7% lime

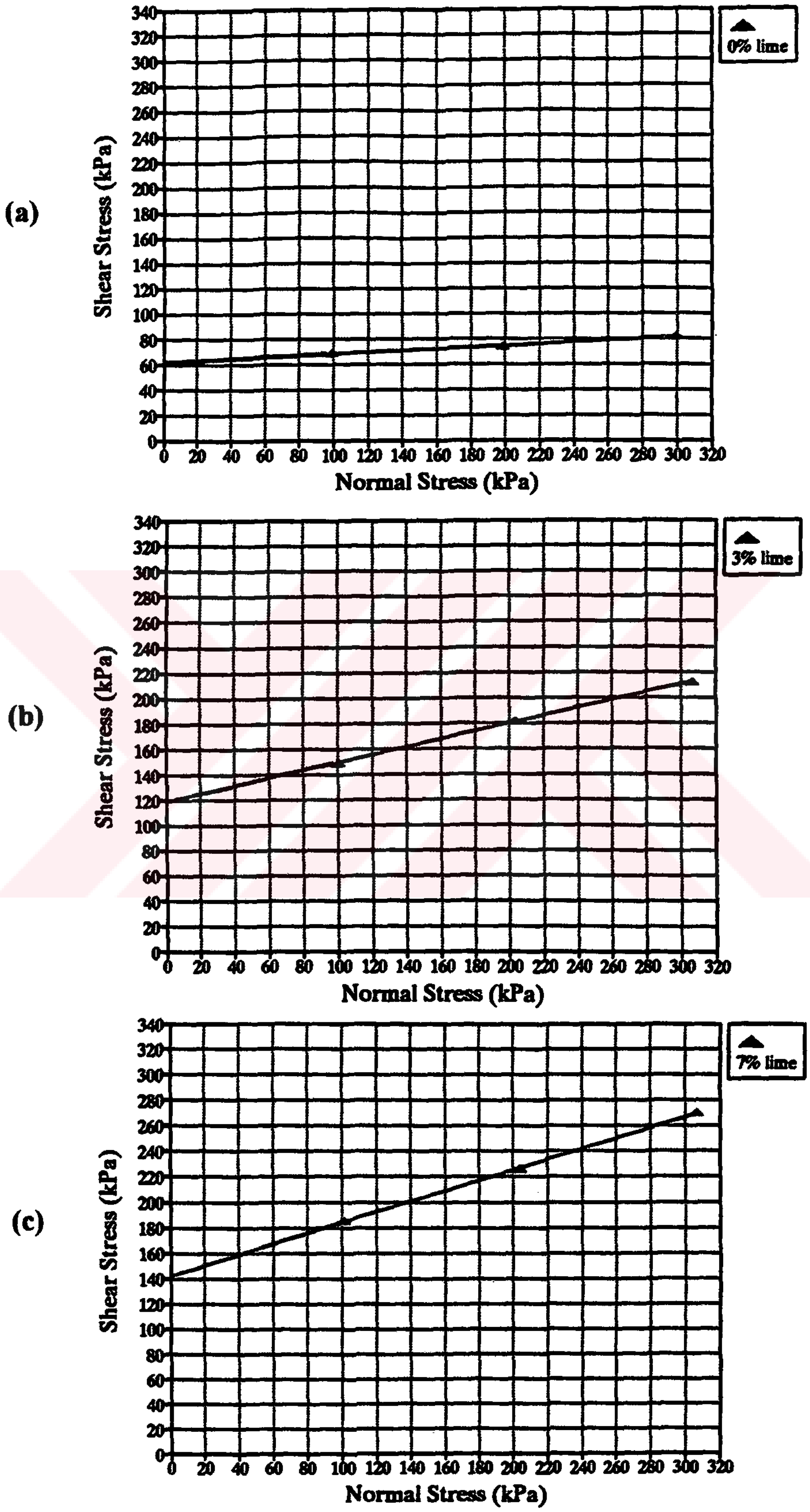


FIGURE A.7. Failure envelops of soil samples which is composed of 30% sand and 70% bentonite a) with no lime, b) with 3% lime, c) with 7% lime

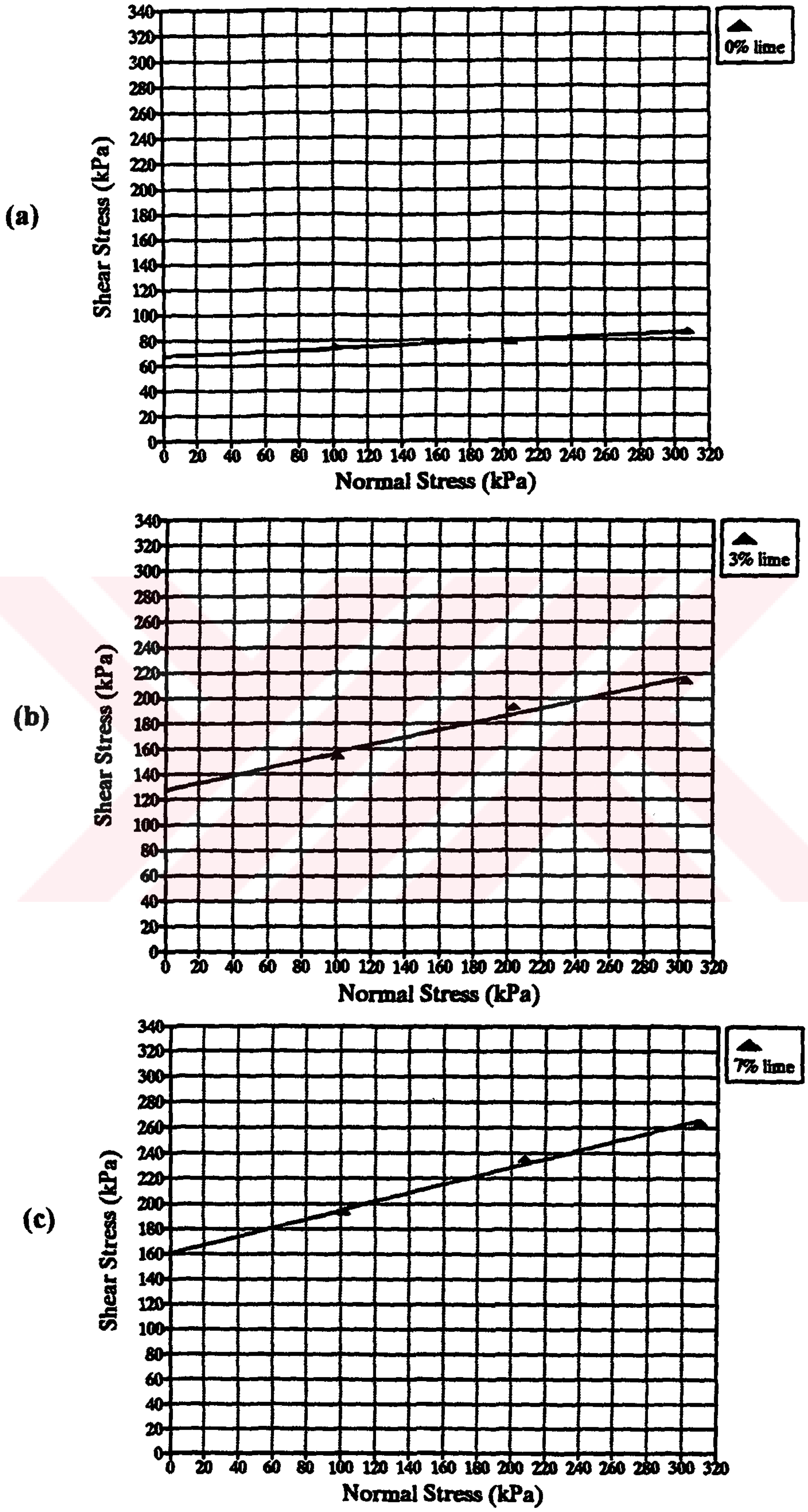


FIGURE A.8. Failure envelops of soil samples which is composed of 50% sand and 50% bentonite a) with no lime, b) with 3% lime, c) with 7% lime

(a)

(b)

(c)

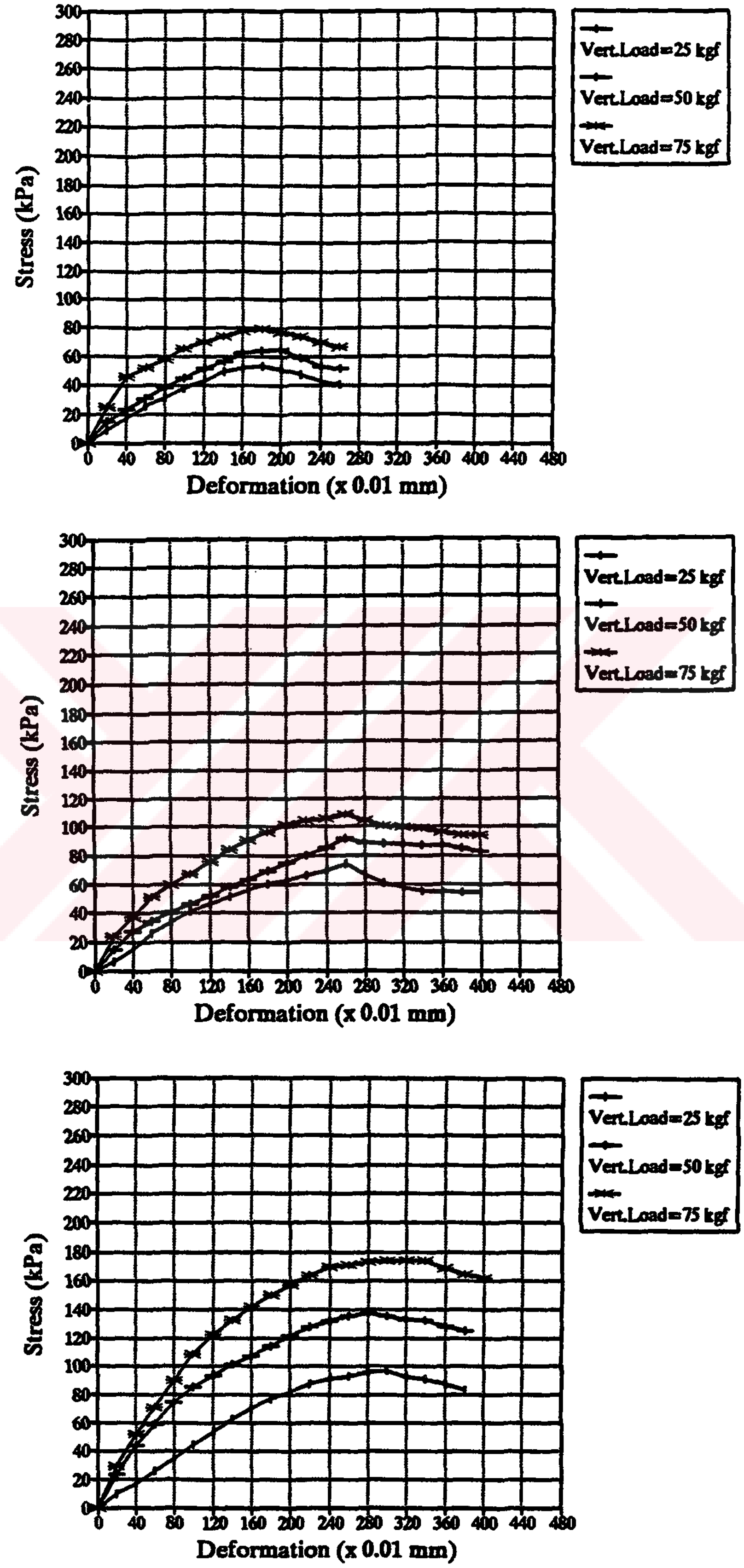


FIGURE A.9. Shear Strength vs deformation behavior of layered samples (soil sections: 100% bentonite) a) with no lime, b) with 3% lime, c) with 7% lime

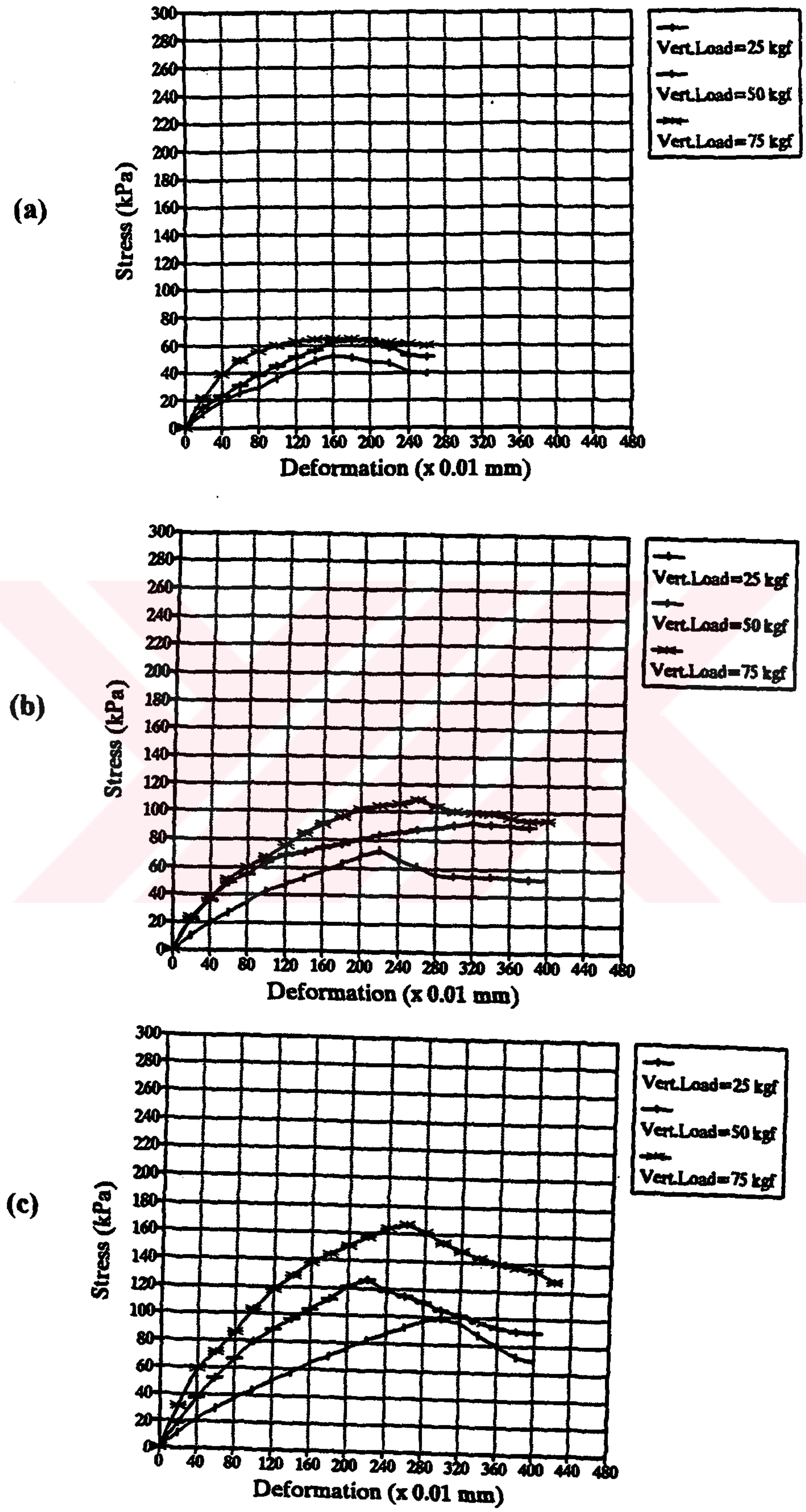


FIGURE A.10. Shear Strength vs deformation behavior of layered samples (soil sections: 100% bentonite) a) with no lime, b) with 3% lime, c) with 7% lime

(a)

(b)

(c)

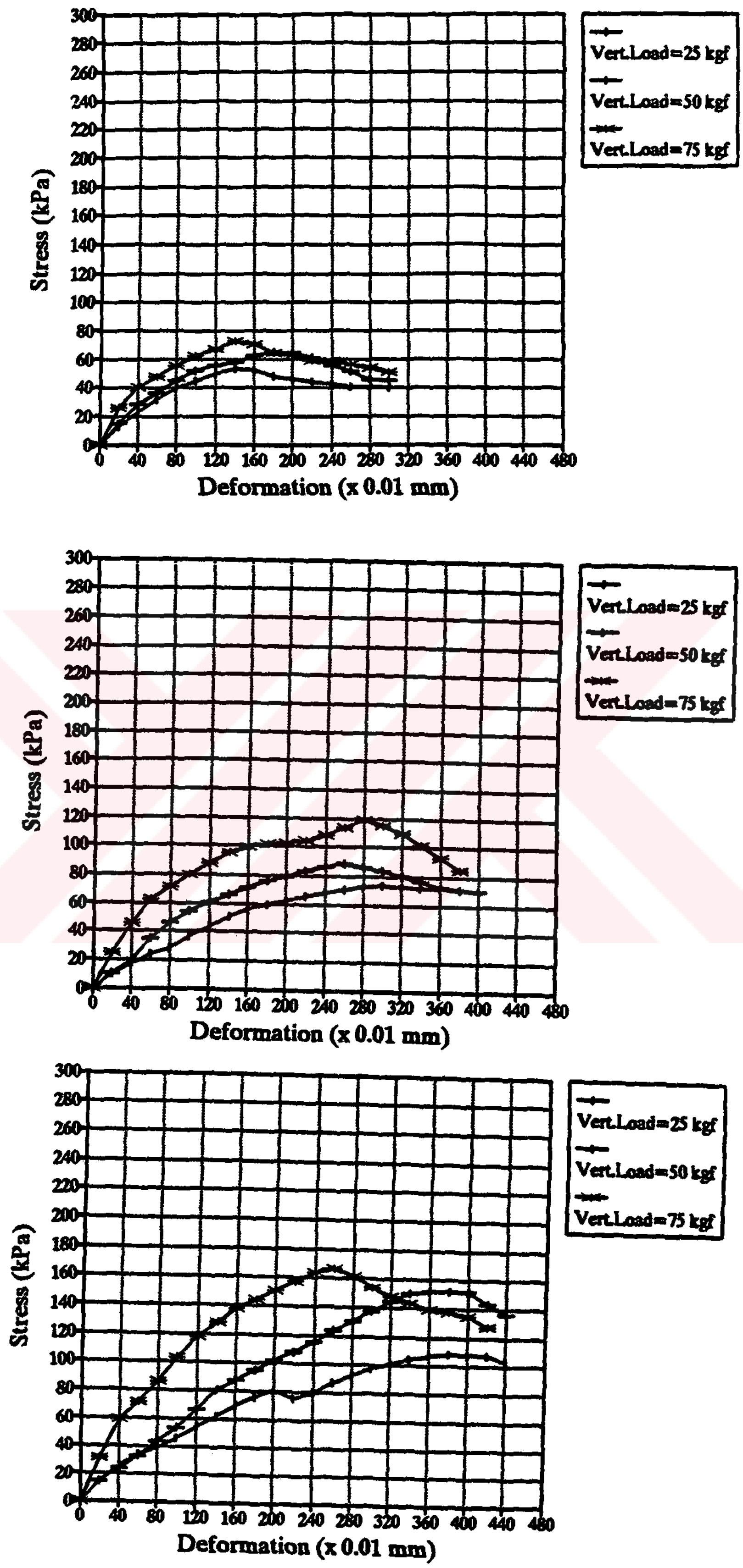


FIGURE A.11. Shear Strength vs deformation behavior of layered samples (soil sections: 10% sand and 90% bent.) a)with no lime, b)with 3% lime, c)with 7% lime

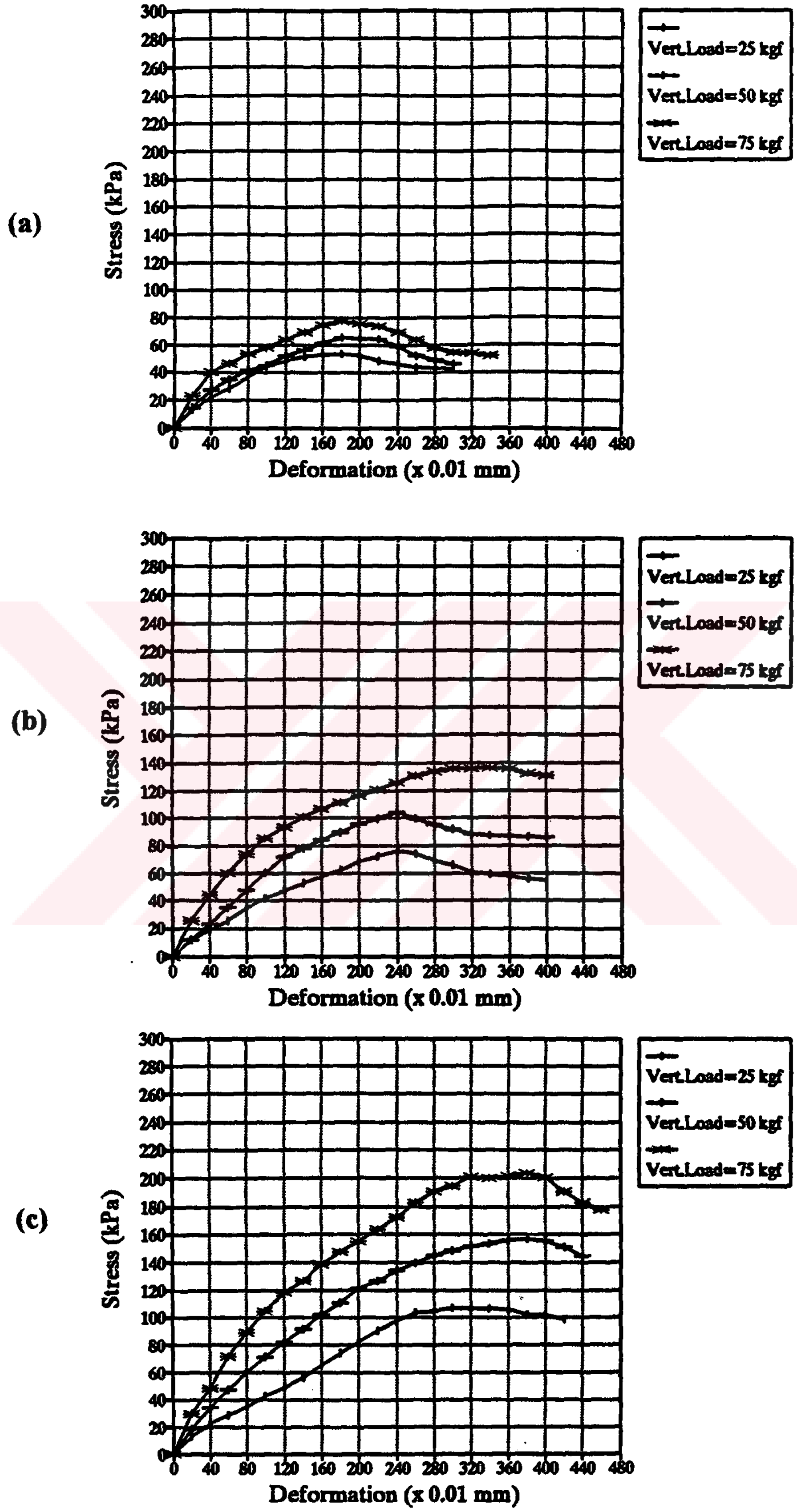


FIGURE A.12. Shear Strength vs deformation behavior of layered samples (soil sections: 10% sand and 90% bent.) a)with no lime, b)with 3% lime, c)with 7% lime

(a)

(b)

(c)

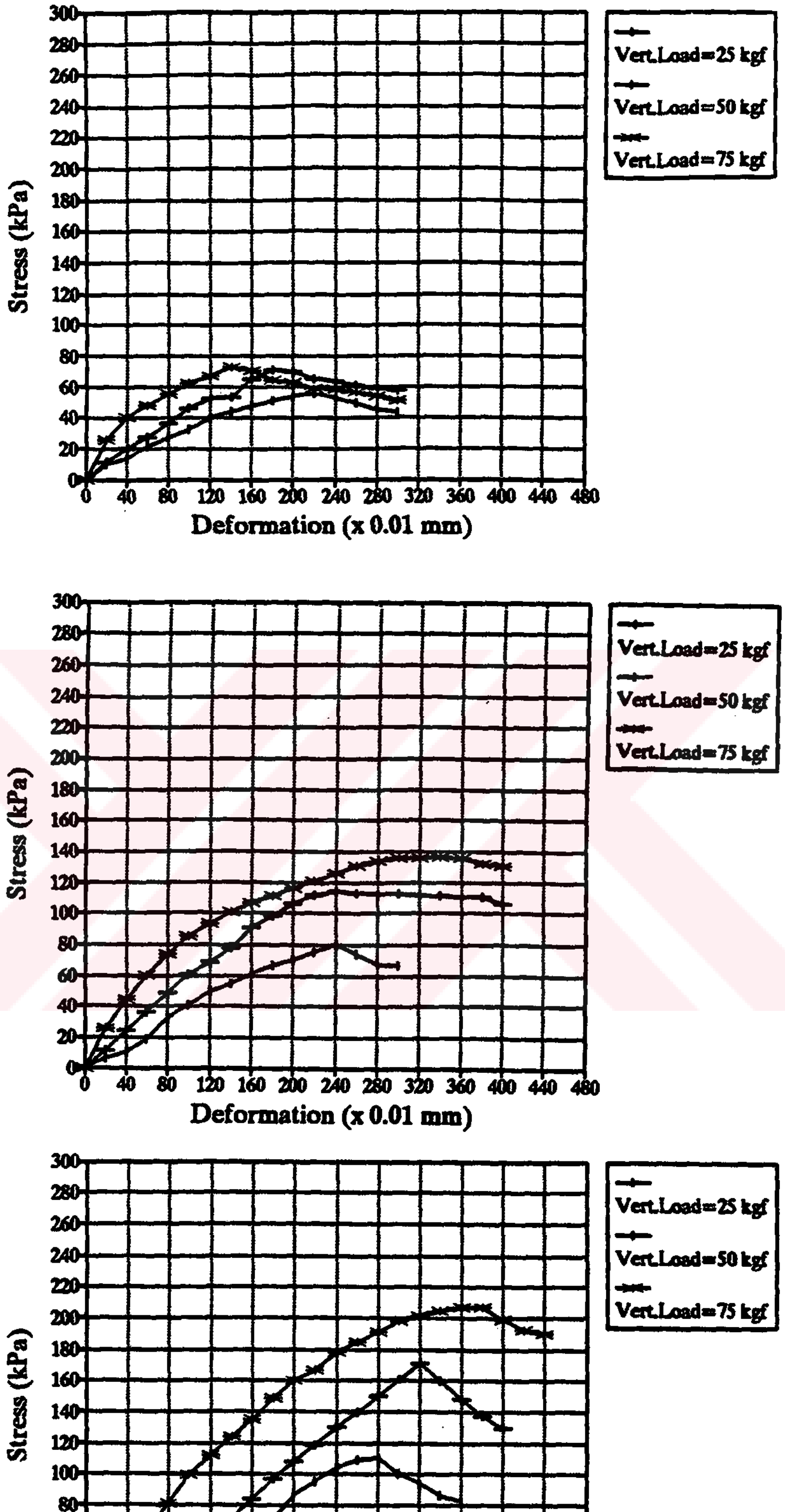


FIGURE A.13. Shear Strength vs deformation behavior of layered samples (soil sections: 30% sand and 70% bent.) a)with no lime, b)with 3% lime, c)with 7% lime

(a)
(b)
(c)

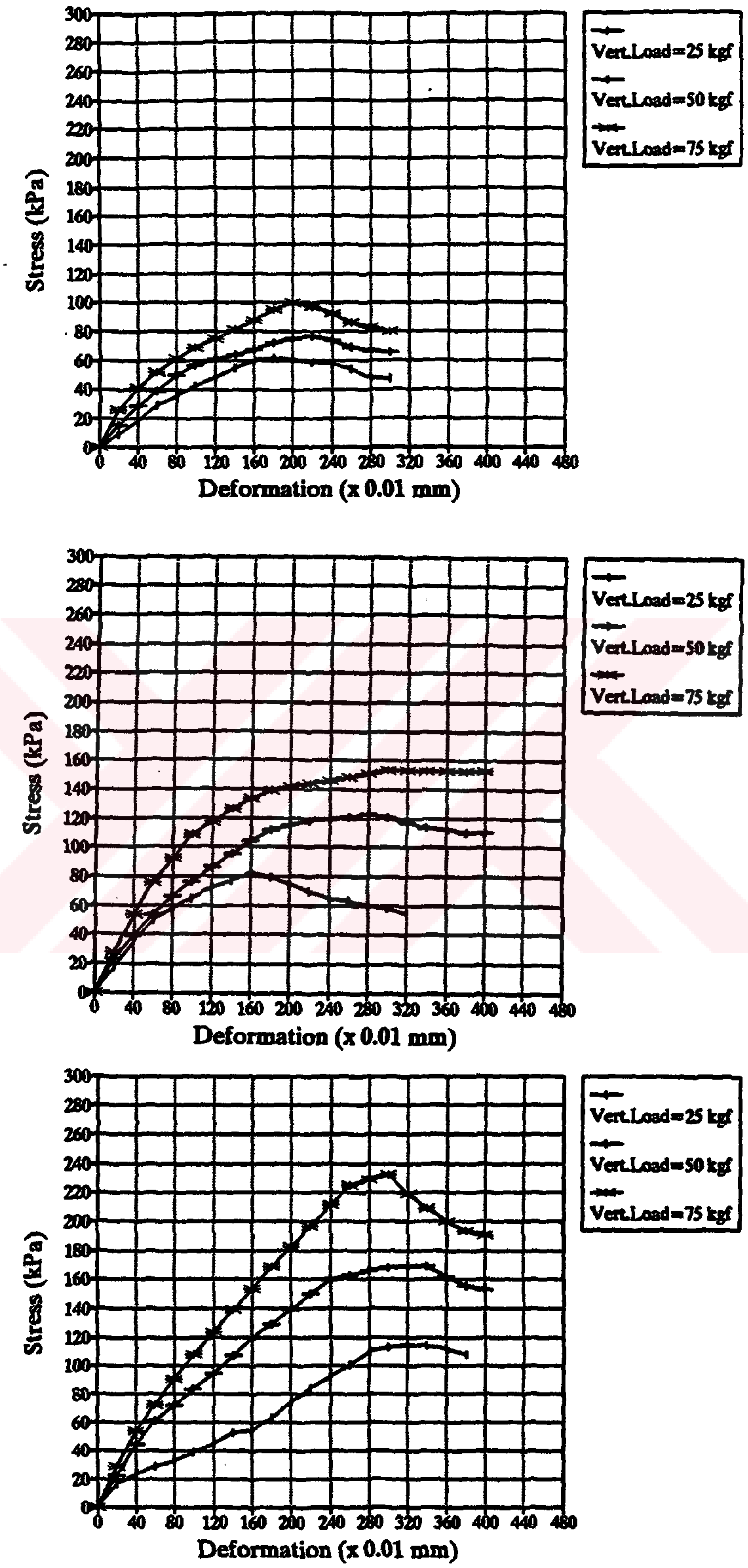


FIGURE A.14. Shear Strength vs deformation behavior of layered samples (soil sections: 30% sand and 70% bent.) a)with no lime, b)with 3% lime, c)with 7% lime

(a)

(b)

(c)

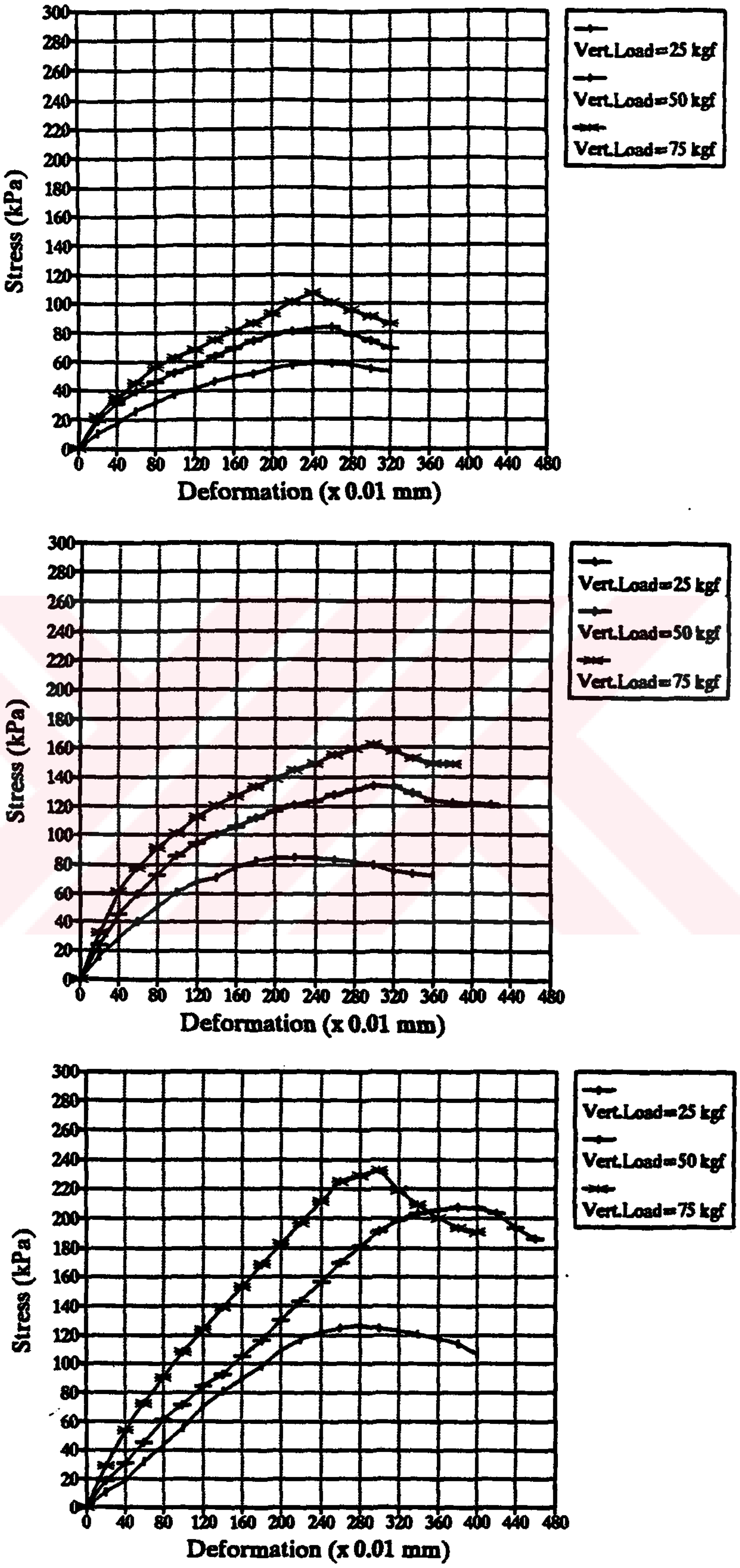


FIGURE A.15. Shear Strength vs deformation behavior of layered samples (soil sections: 50% sand and 50% bent.) a)with no lime, b)with 3% lime, c)with7 % lime

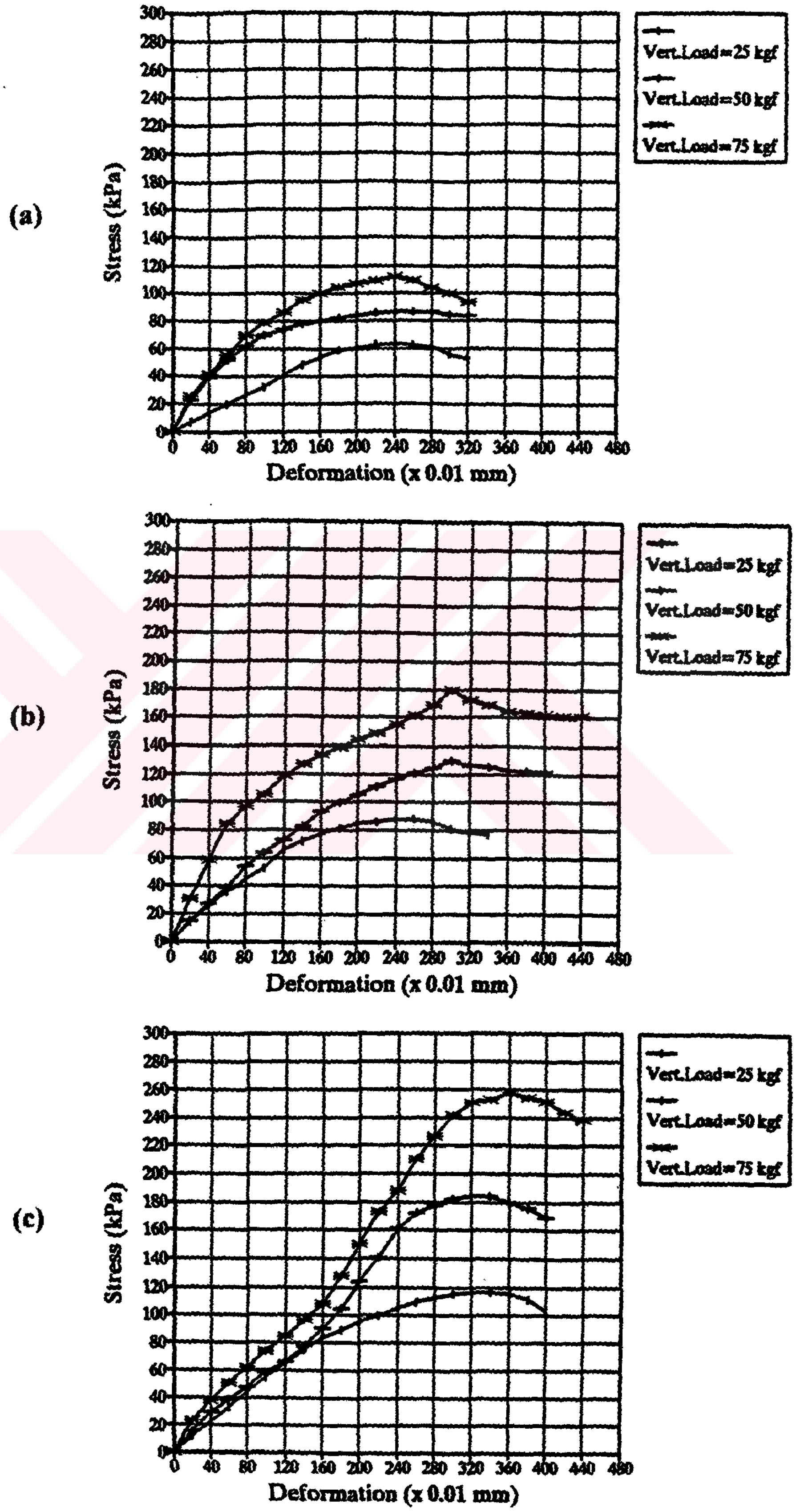
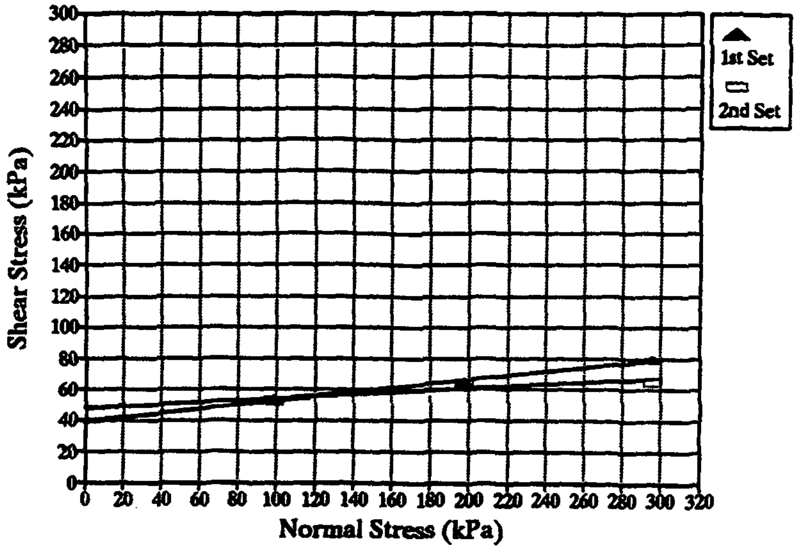
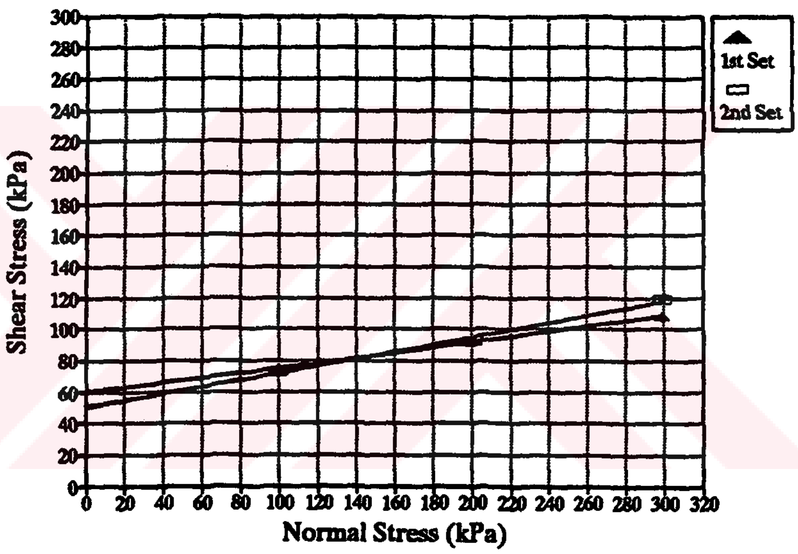


FIGURE A.16. Shear Strength vs deformation behavior of layered samples (
sections: 50% sand and 50% bent.) a)with no lime, b)with 3% lime, c)with 7%

(a)



(b)



(c)

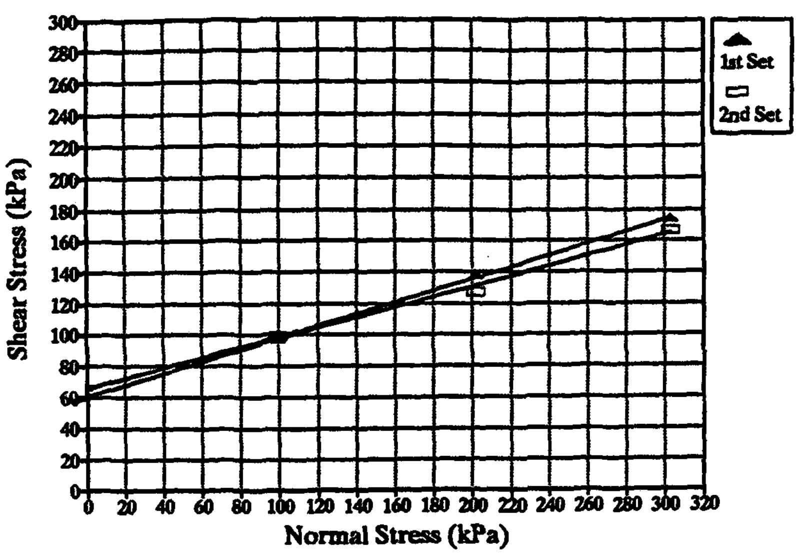
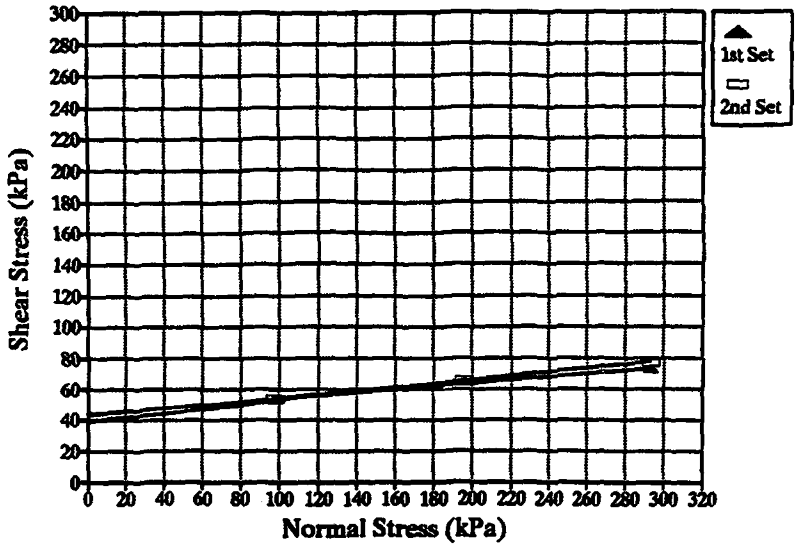
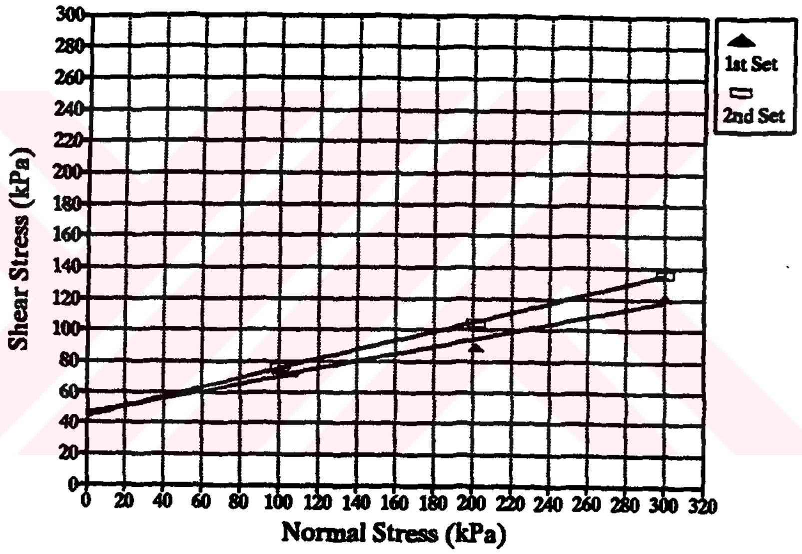


FIGURE A.17. Failure envelops of layered samples (soil sections: 100% bentonite) a) with no lime, b) with 3% lime, c) with 7% lime

(a)



(b)



(c)

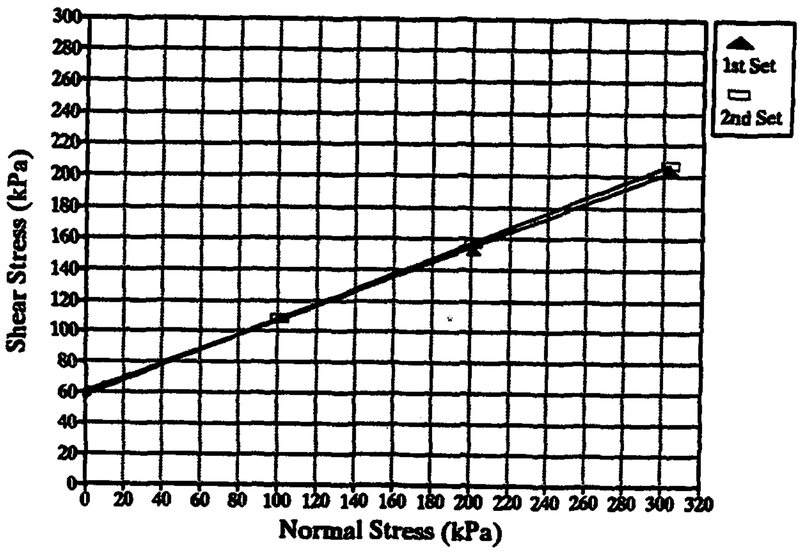


FIGURE A.18. Failure envelops of layered samples (soil sections:10% sand and 90% bentonite) a) with no lime, b) with 3% lime, c) with 7% lime

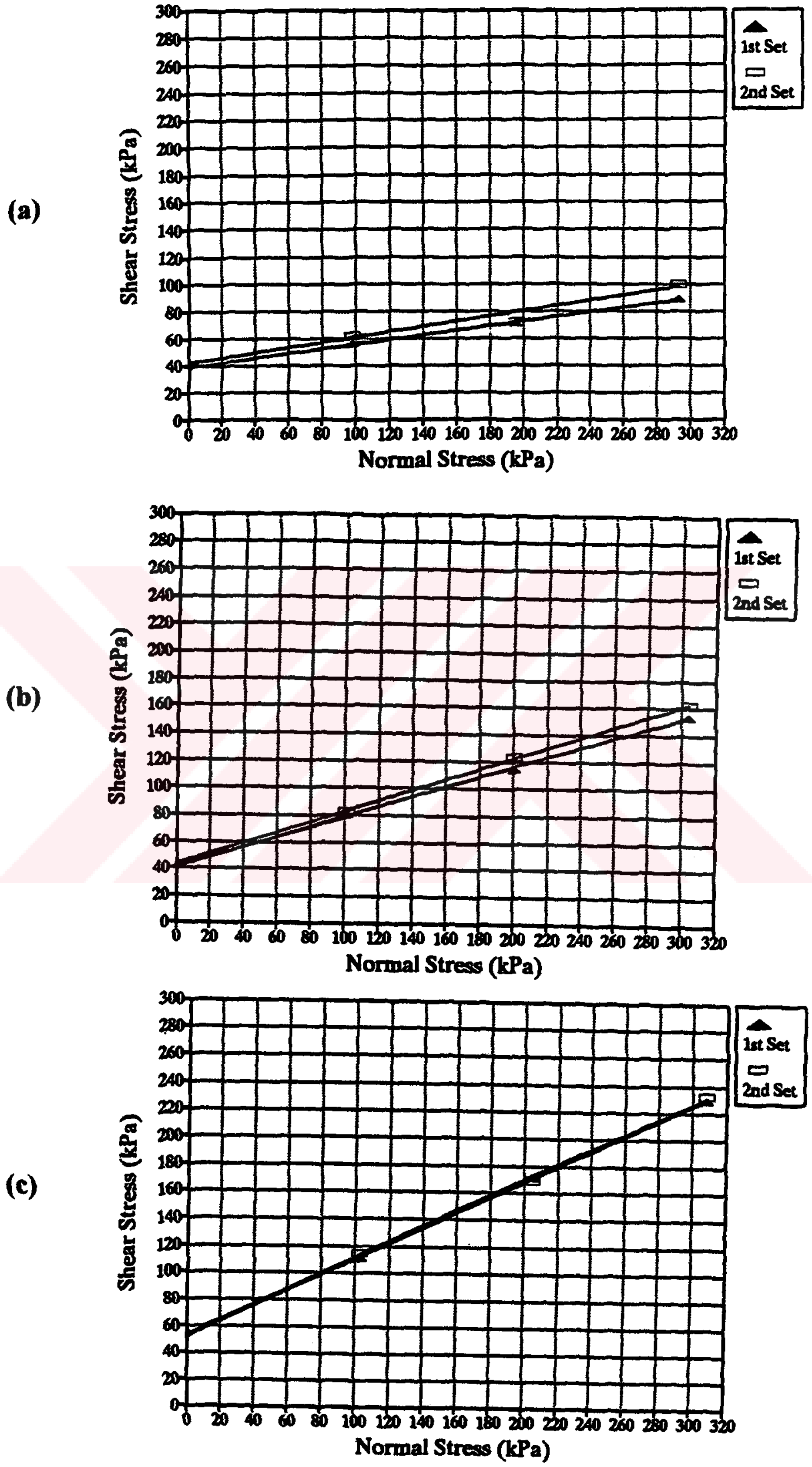


FIGURE A.19. Failure envelops of layered samples (soil sections: 30% sand and 70% bentonite) a) with no lime, b) with 3% lime, c) with 7% lime

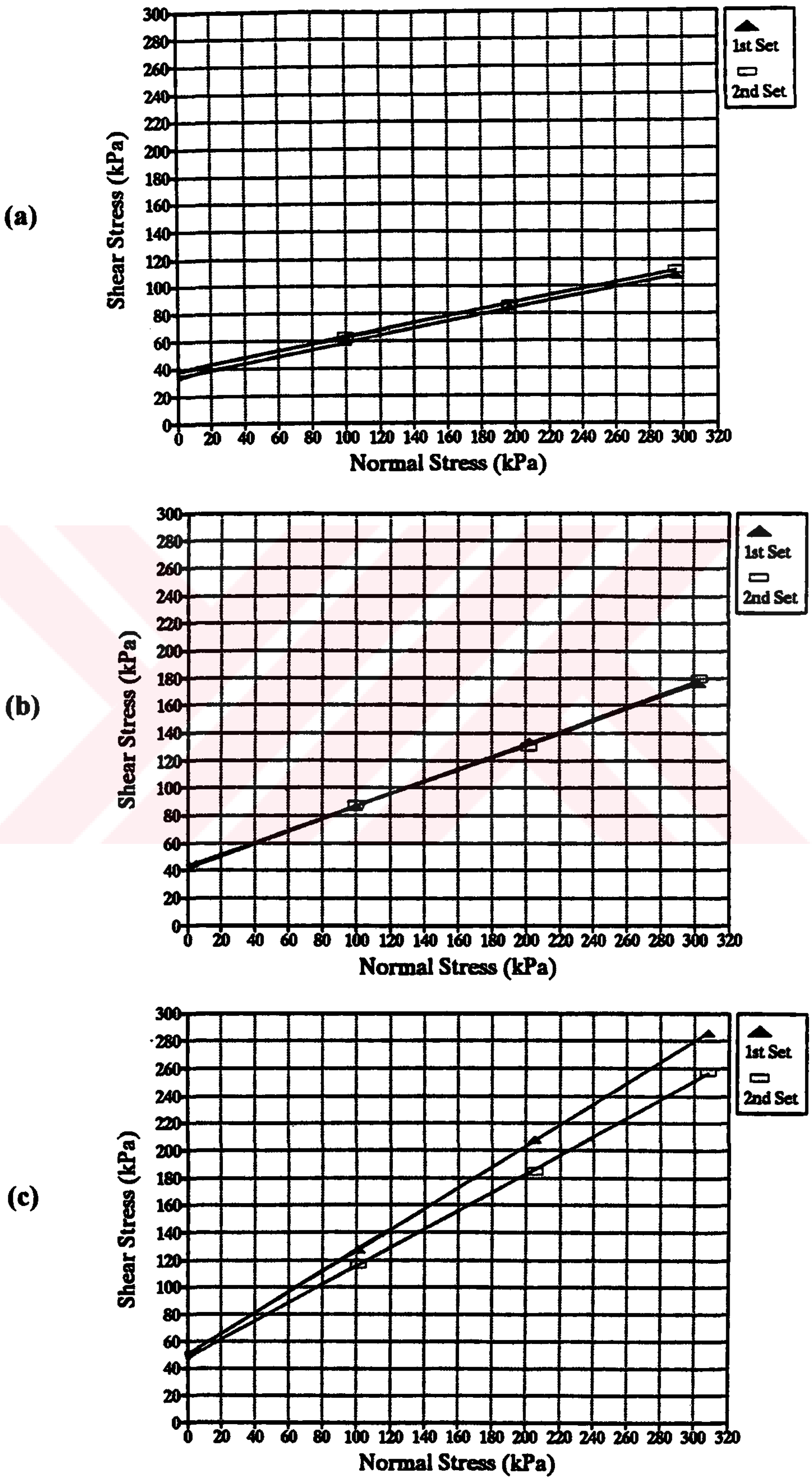


FIGURE A.20. Failure envelops of layered samples (soil sections: 50% sand and 50% bentonite) a) with no lime, b) with 3% lime, c) with 7% lime

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