STABILIZATION OF EXPANSIVE SOILS BY ÇAYIRHAN FLY ASH AND DESULPHOGYPSUM

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
THE MIDDLE EAST TECHNICAL UNIVERSITY

BY

SERTAN IŞIK ÇETİNER

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

IN

THE DEPARTMENT OF CIVIL ENGINEERING

JANUARY 2004

ABSTRACT

STABILIZATION OF EXPANSIVE SOILS BY ÇAYIRHAN FLY ASH AND DESULPHOGYPSUM

Çetiner, Sertan Işık
M.S., Department of Civil Engineering
Supervisor: Prof. Dr. Erdal Çokça

January 2004, 107 Pages

Expansive soils are one of the most serious problems which the foundation engineer faces. Several attempts are being made to control the swell-shrink behavior of these soils. One of the most effective and economical methods is to use chemical additives. Fly ash and desulphogypsum, both of which are by-products of coal burning thermal power plants, are accumulating in large quantities all over the world and pose serious environmental problems. In this study, the expansive soil was stabilized using the fly ash and desulphogypsum obtained from Çayırhan Thermal Power Plant. Fly ash and desulphogypsum were added to the expansive soil from 0 to 30 percent. Lime was used to see how efficient fly ash and desulphogypsum on expansive soil stabilization were, and was added to the expansive soil from 0 to 8 percent. The properties obtained were chemical composition, grain size distribution, consistency limits, swelling percentage, and rate of swell. Fly ash, desulphogypsum, and lime added samples were cured for 7 days and 28 days, after which they were subjected to free swell tests. Swelling percentage decreased and rate of swell increased with increasing stabilizer percentage. Curing resulted in further reduction in swelling percentage and further increase in rate of swell. 25 percent and 30 percent fly ash and desulphogypsum additions reduced the swelling percentage to levels comparable to lime stabilization.

Keywords: Desulphogypsum, Expansive Soil, Fly Ash, Lime, Rate of Swell, Stabilization, Swelling Percentage

ÖZ

ŞİŞEN ZEMİNLERİN ÇAYIRHAN UÇUCU KÜLÜ VE DESÜLFOJİPS İLE STABİLİZASYONU

Çetiner, Sertan Işık
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Prof. Dr. Erdal Çokça

Ocak 2004, 107 Sayfa

Şişen zeminler temel mühendisinin karşılaştığı en ciddi sorunlardan bir tanesidir. Bu zeminlerin şişme-büzülme davranışını kontrol edebilmek için bir çok girişimler yapılmaktadır. En etkili ve ekonomik metodlardan bir tanesi kimyasal katkılar kullanmaktır. Kömür yakan termik santrallerin yan ürünleri olan uçucu kül ve desülfojips tüm dünyada büyük miktarlarda birikmekte ve ciddi çevresel sorunlar teşkil etmektedir. Bu çalışmada, şişen zemin Çayırhan Termik Santrali'nden elde edilen uçucu kül ve desülfojips ile stabilize edilmiştir. Uçucu kül ve desülfojips şişen zemine 0'dan yüzde 30'a kadar eklenmiştir. Uçucu kül ve desülfojipsin şişen zemin stabilizasyonunda ne kadar verimli olduklarını anlamak için kireç kullanılmış, ve şişen zemine 0'dan yüzde 8'e kadar eklenmiştir. Kimyasal bileşim, dane dağılımı, kıvam limitleri, şişme yüzdesi ve şişme hızı elde edilen özelliklerdir. Uçucu kül, desülfojips ve kireç eklenmiş numunelere, sonrasında serbest şişme deneylerine tabi tutulmak üzere, 7 günlük ve 28 günlük kür uygulanmıştır. Stabilizan yüzdesinin artmasıyla şişme yüzdesi düşmüş ve şişme hızı artmıştır. Kür şişme yüzdesinin daha da düşmesine ve şişme hızının daha da artmasına sebep olmuştur. Yüzde 25 ve yüzde 30 uçucu kül ve desülfojips eklenmesi şişme yüzdesini kireç stabilizasyonuyla karşılaştırılabilir seviyelere düşürmüştür.

Anahtar Kelimeler: Desülfojips, Kireç, Stabilizasyon, Şişen Zemin, Şişme Hızı, Şişme Yüzdesi, Uçucu Kül

To My Parents

ACKNOWLEDGEMENTS

The author acknowledges Prof. Dr. Erdal Çokça for his guidance, valuable suggestions, friendly approaches and insight throughout the research.

Thanks are extended to Ali Bal for his very important contributions in accuracy of the experiments.

TABLE OF CONTENTS

CHAPTER

APPENDICES

LIST OF TABLES

TABLE

LIST OF FIGURES

FIGURE

LIST OF ABBREVIATIONS

A: Sample A

ASTM: American Society for Testing and Materials

CH: Inorganic clays of high plasticity

CL: Inorganic clays, silty clays, sandy clays of low plasticity

dH: Change in initial height of the sample

DSG: Desulphogypsum

FA: Fly ash

G_s: Specific gravity

H: Initial height of the sample

L: Lime

LL: Liquid limit

MH: Inorganic silts of high plasticity

ML: Inorganic silts, silty or clayey fine sands, with slight plasticity

PI: Plasticity index

PL: Plastic limit

SI: Shrinkage index

SL: Shrinkage limit

t_{50}: Time to reach fifty percent swell

CHAPTER 1

INTRODUCTION

Expansive soils are a worldwide problem that poses several challenges for civil engineers. They are considered a potential natural hazard, which can cause extensive damage to structures if not adequately treated. Such soils swell when given an access to water and shrink when they dry out (Al-Rawas et al. 2002).

In general, expansive soils have high plasticity, and are relatively stiff or dense. The expansive nature of soil is most obvious near the ground surface where the profile is subjected to seasonal, environmental changes. The pore water pressure is initially negative and the deposit is generally unsaturated. These soils often have some montmorillonite clay mineral present. The higher the amount of monovalent cations absorbed to the clay mineral (e.g. sodium), the more severe the expansive soil problem (Fredlund and Rahardjo, 1993).

Expansive soils have been reported from many parts of the world, mainly in the arid or semi-arid regions of the tropical and temperate zones like Africa, Australia, India, South America, United States, and some regions in Canada. This never means that expansive soils do not exist elsewhere, because they can be found almost everywhere. However, in the humid regions water tables are generally at shallow depth and moisture changes, which are responsible for volume changes in soils, are minimal excepting under extended drought conditions (Arnold, 1984; Shuai and Fredlund, 1998; Wayne et al. 1984).

The problems with foundations on expansive soils have included heaving, cracking and break-up of pavements, roadways, building foundations, slab-on-grade members, channel and reservoir linings, irrigation systems, water lines, and sewer lines (Çokça, 2001).

It is reported that damage to the structures due to expansive soils has been the most costly natural hazard in some countries (in United States more than the cost of damage from floods, hurricanes, tornadoes, and earthquakes on an average annual basis) for years (Kehew, 1995; Shuai and Fredlund, 1998).

Laboratory tests have to be carried out to determine the swelling properties of such soils before a structure can be designed for such sites, and stabilization methods must be investigated prior to construction to eliminate possible future problems. One of the most effective and economical methods to prevent volume changes of an expansive clay soil is through the use of chemical additives. Lime and cement have been used for this purpose for many years.

On the other hand, the increasing demand for energy has resulted in construction of many coal-fired power plants in Turkey. This development brought it with the problem of safe disposal or beneficial utilization of large quantities of by-products from these power plants (Çokça, 2001). Çayırhan Thermal Power Plant is located at 120 km from Ankara and 22 km from Beypazarı. The plant covers a total area of 5,032,000 m^2. It has four boiler units, two of them (Units I and II) with 150 MW capacity and two of them (Units III and IV) with 160MW capacity. Units I and II have been working since 1987 and Units III and IV have been working since 1998. All of the four units are equipped with flue gas desulphurization systems. These four units, with a total capacity of 620MW, use 5,000,000 tons of lignite coal and generate 4,200,000,000 kW-h electricity per year. The lignite coal, extracted from the underground mines of the Beypazarı Basin, is of low calorific value (2200 kcal/kg), high dust (30 – 45%), and high sulphur (4 – 5%) content. As a result of their electricity generation the four units of Çayırhan Thermal Power Plant produce 1,350,000 tons of fly ash and 680,000 tons of desulphogypsum annually. Fly ash and desulphogypsum are collected by means of electrostatic precipitators and are sent through 2.5 km transfer bands into open stock areas which now cover a total area of 1,137,000 m^2. Less than 1% of fly ash and none of the desulphogypsum is productively employed. The plant is estimated to work for minimum another 20 years, and it is conceivable to mention that this will duplicate the fly ash and desulphogypsum stocks. These stocks pose a serious problem in terms of both land use and potential environmental pollution. An effective utilization of these industrial by-products must be regarded as economically and environmentally beneficial.

In this study, the expansive soil was stabilized using the fly ash and desulphogypsum obtained from Çayırhan Thermal Power Plant. Lime was used to see how effective fly ash and desulphogypsum on expansive soil stabilization were. An extensive laboratory testing program was undertaken to provide information on the geotechnical properties of expansive soils treated with Çayırhan fly ash and desulphogypsum, and lime.

CHAPTER 2

REVIEW OF EXPANSIVE SOILS

The term expansive soil applies to soils, which have the tendency to swell when their moisture content is allowed to increase. The moisture may come from rain, flooding, leaking water or sewer lines, or from a reduction in surface evapotranspiration when an area is covered by a building or pavement. The term cracking soils is also used for these soils since they have the tendency to shrink and crack when the moisture is allowed to decrease. Soils containing the clay mineral montmorillonite generally exhibit these properties (Komine and Ogata, 1996; Rao and Triphaty, 2003; Sivapullaiah et al. 1996; Wayne et al. 1984).

There are many correlations that are useful in identifying potentially expansive soils. It may also be possible to identify them visually. Visual indications include (Wayne et al. 1984):

1) Wide and deep shrinkage cracks occurring during dry periods
2) Soil is rock-hard when dry, but very sticky and soft when wet
3) Damages on the surrounding structures due to expansion of soil

2.1. Structure of Clay Minerals

Clay minerals are primarily the end product of the chemical weathering of feldspathic rock. Chemically, these minerals are essentially hydrous aluminum silicates, although occasionally the aluminum atoms are replaced with atoms of other elements, such as magnesium, iron, potassium, or sodium (Duncan, 1992). The atomic structure of a clay mineral is highly complex, and consists of a variety of combinations and arrangements of two basic building blocks called the *silica tetrahedron* and the *alumina octahedron* (Fig 2.1).

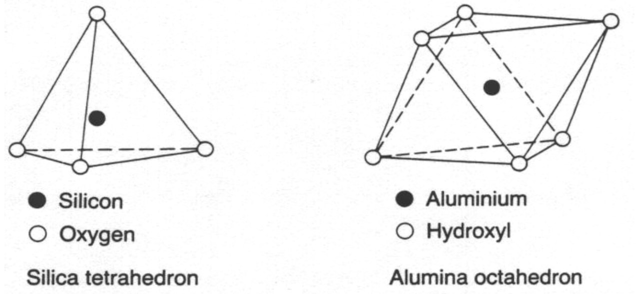

Figure 2.1. Basic Units of Clay Minerals (Craig, 1993)

The various building blocks that make up a clay mineral are arranged in orderly sheets (Figure 2.2.a and Figure 2.2.b). The particular arrangement and chemical composition of these blocks determines the type of clay mineral and its general characteristics.

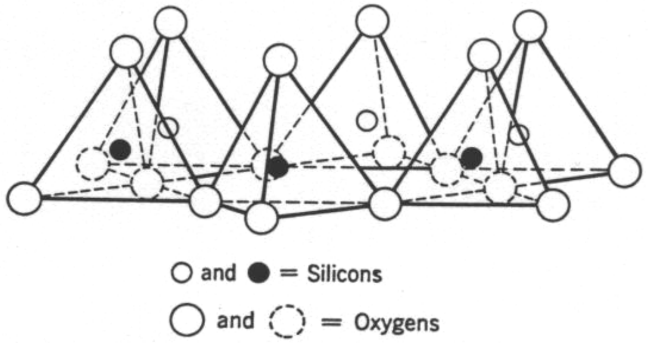

Figure 2.2.a. Silica Sheet (Mitchell, 1993)

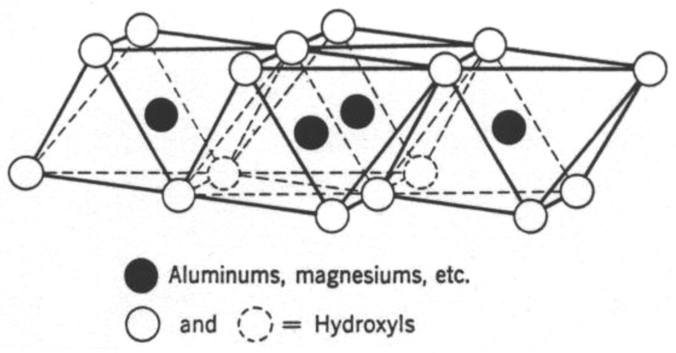

Figure 2.2.b. Octahedral Sheet (Mitchell, 1993)

2.1.1. Major Clay Groups

Clay minerals are grouped according to chemistry, and interaction with water. The three main groups of clay are a) kaolinite, b) illite, and c) montmorillonite.

a) Kaolinite – $Al_4Si_4O_{10}(OH)_8$

The kaolinite group of clays, of which the mineral kaolinite is the principle member, are the most prevalent of all clays. A kaolinite mineral is composed of two sheets, one consisting of silica tetrahedrons and the other of alumina octahedrons (Figure 2.3). These sheets are very strongly bonded together. Kaolinite, therefore, is very stable and has little tendency to change volume when exposed to water or to drought. Kaolinite contains no interlayer water because of the way the sheets fit together. It does, however, have the ability to absorb sufficient water to develop plasticity.

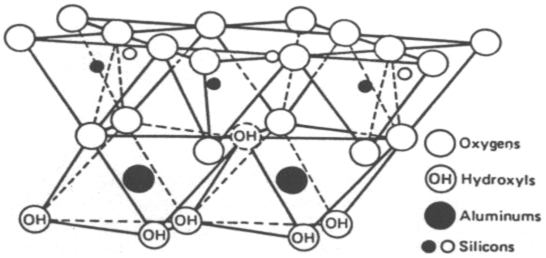

Figure 2.3. Structure of Kaolinite (Mitchell, 1993)

b) Illite

The illite group of clays does not have a principal mineral. Instead, this name refers to a group of micalike clay minerals. The basic structural unit of an illite clay is composed of two silica tetrahedral sheets with a central octahedral sheet (Figure 2.4). Potassium is the primary element in the central sheet. Illite exhibits more plasticity than kaolinite, and has little tendency to change volume when exposed to a change in moisture content unless there is a deficiency in potassium, in which case the illite particle will exhibit an increased tendency for volume change.

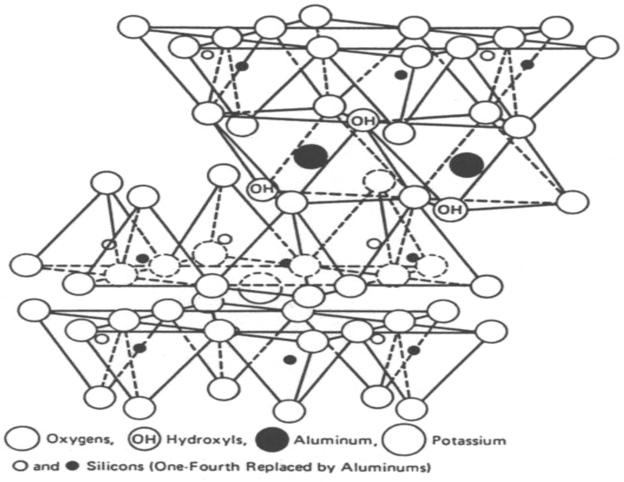

Figure 2.4. Structure of Illite (Mitchell, 1993)

c) Montmorillonite – $Si_8Al_4O_{20}(OH)_4 . nH_2O$

Montmorillonite is a group name for clay minerals which have expansive structures, and is also the name of the principal mineral of the group. The structure of montmorillonite consists of an alumina sheet held between two silica sheets to form a weakly bonded, three sheet layer (Figure 2.5). This mineral exhibits considerable variation in characteristics because of the interchange between elements within each sheet. Iron or aluminum, for example, may replace the aluminum in the alumina sheet, and aluminum may replace some of the silicons in the silica sheet.

This mineral exhibits the highly undesirable characteristic of undergoing considerable change in volume when moisture is added to or deleted from the soil mass. This characteristic can lead to very serious problems of heaving or of settlement.

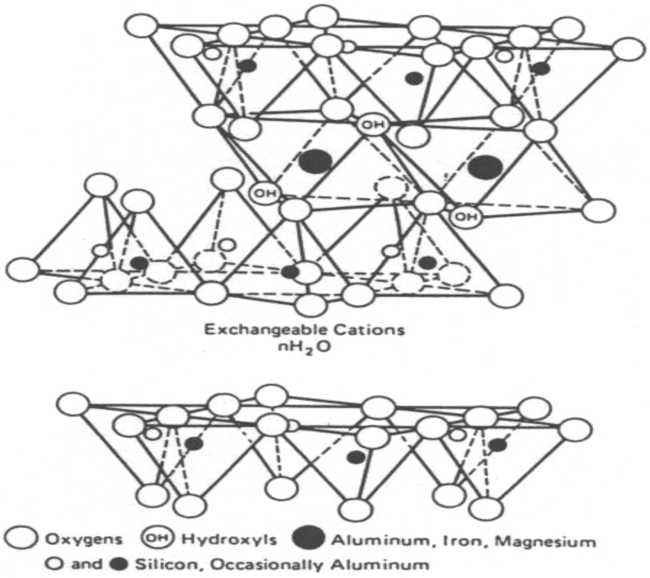

Figure 2.5. Structure of Montmorillonite (Mitchell, 1993)

2.2. Mechanism of Swelling

The mechanism of swelling in expansive clays is complex and is influenced by a number of factors. Expansion is a result of changes in the soil water system that disturb the internal stress equilibrium. Clay particles generally are platelets having negative electrical charges on their surfaces and positively charged edges. The negative charges are balanced by cations in the soil water that become attached to the surfaces of the platelets by electrical forces. The electrical interparticle force field is a function of both the negative surface charges and the electrochemistry of the soil water (Figure 2.6). Van der Waals surface forces and adsorptive forces between the clay crystals and water molecules also influence the interparticle force field. The internal electrochemical force system must be in equilibrium with the externally applied stresses and capillary tension in the soil water (Nelson and Miller, 1992).

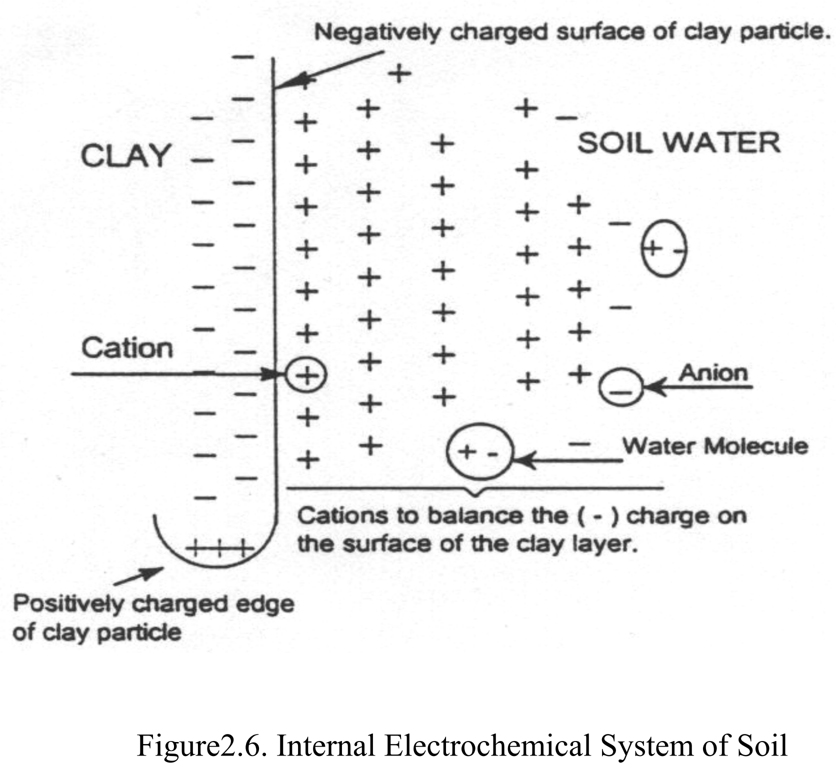

Figure2.6. Internal Electrochemical System of Soil

Considering the internal electrochemical force system, there are two basic mechanisms involved in swelling phenomena (Figure 2.7):

1) Interparticle or intercrystalline swelling, which is effective for all kinds of clay minerals. In a nearly dry clay deposit relict water holds the particles together under tension from capillary forces. On wetting, the capillary tensions are relaxed and the clay expands. The short dashes shown in the upper part of Figure 2.7 which link the layers of the clay crystals imply that the layers are strongly bonded by molecular forces (Popescu, 1986).

2) Intracrystalline swelling, which is chiefly a characteristic of the montmorillonite group of minerals. The layers that make up the individual single crystals of montmorillonite are weakly bonded, mainly by water in combination with exchangeable cations. On wetting, water enters not only between the single crystals, but also between the individual layers that make up the crystals (Popescu, 1986).

There can be two factors causing intracrystalline swelling:

a) The unbalanced electrostatic charges on clay-particle surfaces draw water molecules into the area between silicate sheets, thus forcing them apart (Kehew, 1995).

b) The cations attracted to the clay surfaces provide the other factor in swelling behavior. Because of the attraction of the negatively charged clay-particle surfaces for cations, small spaces within or between clay particles may contain a higher concentration of cations than larger pores within the soil. These conditions (Figure 2.8) create an osmotic potential between the pore fluids and the clay-mineral surfaces. Normally, cations diffuse from a higher concentration to a lower concentration in order to evenly distribute the ions throughout the solution. In expansive soils, because ions are held by

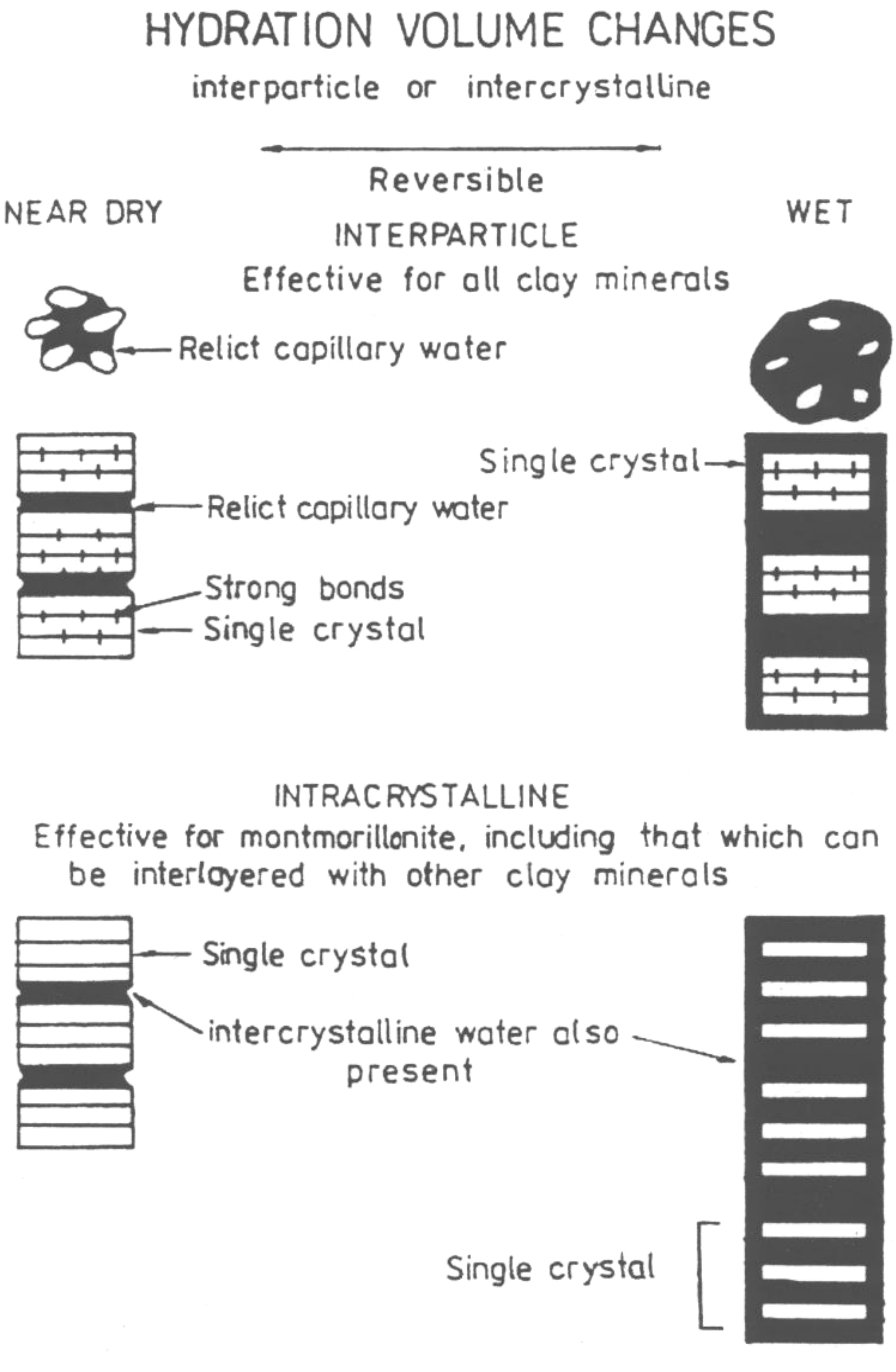

Figure 2.7. Mechanism of swelling (Popescu, 1986)

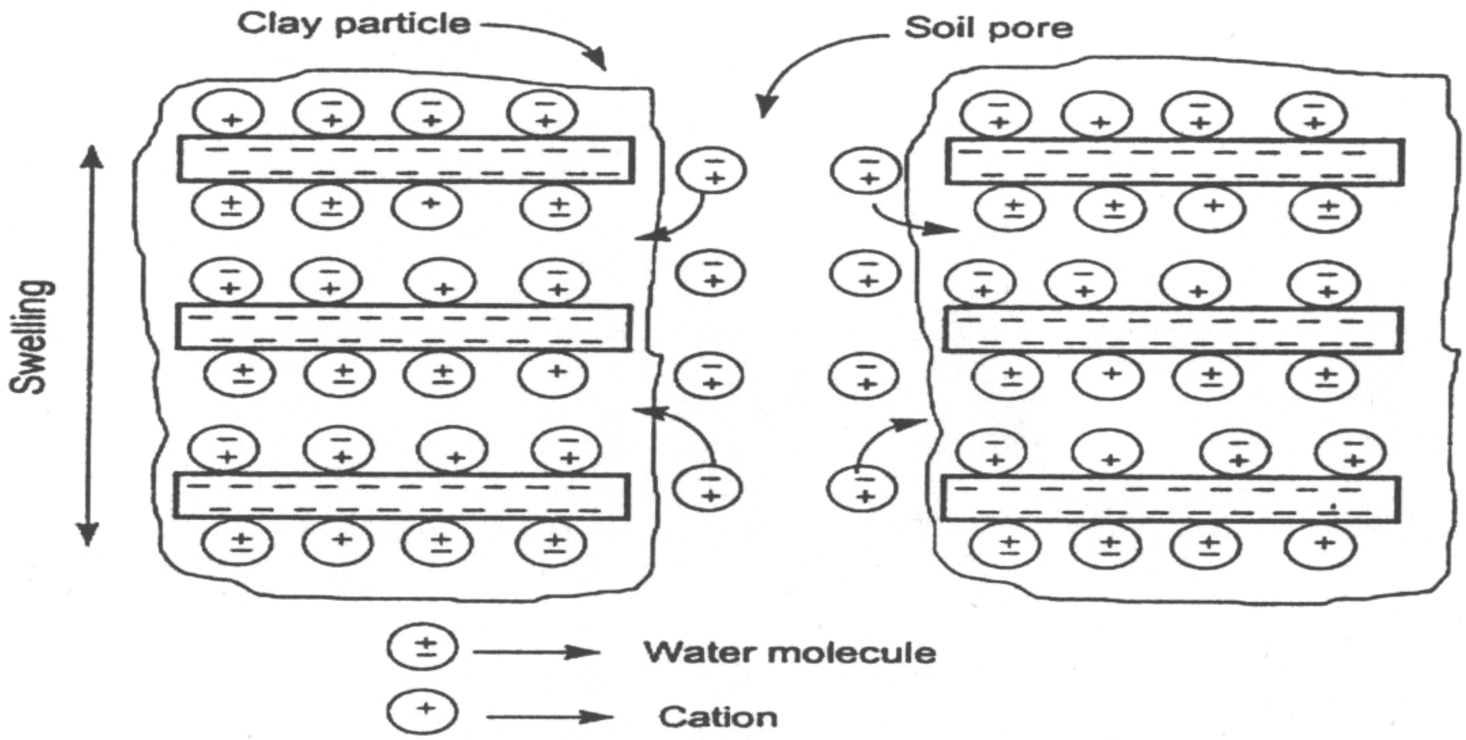

Figure 2.8. Swelling of Clay Rich Soils (Kehew, 1995)

the clay particles, water moves from areas of low ionic concentration (high concentration of water) to areas of high ionic concentration (low concentration of water) within clay particles or aggregates. This influx of water exerts pressure, which causes the clay to swell (Kehew, 1995).

If a clay soil is subjected to drying conditions, for example, when evaporation is removing water from the soil near land surface, a suction effect is exerted on the soil that causes water molecules that are not held tightly to clay particles to be drawn out into the large pores of the soil and to move upward to replace the evaporated water. This loss of water from the clay leads to shrinkage, the reversal of swelling process (Kehew, 1995).

2.3. Factors Affecting Swelling

The factors influencing the shrink-swell potential of a soil can be considered in three different groups:

- The *soil characteristics* that influence the basic nature of the internal force field (Table 2.1)
- The *environmental factors* that influence the changes that may occur in the internal force system (Table 2.2)
- The *state of stress* (Table 2.3)

Table 2.1. Soil Properties that Influence Swelling Potential (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
Clay Content and Mineralogy	Clay minerals which typically cause soil volume changes are montmorillonites, vermiculites, and some mixed layer minerals. Swelling potential increases with the increasing amount of clay minerals.
Soil Water Chemistry	Swelling is repressed by increased cation concentration and increased cation valance. For example, Mg^{+2} cations in the soil water would result in less swelling than Na^{+} cations.
Soil Structure and Fabric	Flocculated clays tend to be more expansive than dispersed clays. Cemented particles reduce swell.
Initial Dry Density	Higher densities indicate closer particle spacings which mean greater repulsive forces between particles and larger swelling potential.
Initial Water Content	As the initial water content increases, the initial degree of saturation will also increase and the affinity of soil to absorb water will decrease, so the amount of swelling will decrease.
Coarse Grained Fraction	As the amount of fine particles increase, the amount of swelling will increase due to the larger surface area.
Plasticity	In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling. Plasticity is an *indicator* of swelling potential.

Table 2.2. Environmental Conditions that Influence Swelling Potential (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
Climate	Amount and variation of precipitation and evapotranspiration greatly influence the moisture availability and depth of seasonal moisture fluctuation. Greatest seasonal heave occurs in semiarid climates that have short wet periods.
Groundwater	Shallow water tables provide a source of moisture and fluctuating water tables contribute to moisture.
Drainage	Poor surface drainage leads to moisture accumulations or ponding.
Vegetation	Vegetation (trees, shrubs, grasses, etc.) deplete moisture from the soil through transpiration, and cause the soil to be differentially wetted in areas of varying vegetation.
Permeability	Soils with higher permeabilities, particularly due to fissures and cracks in the field soil mass, allow faster migration of water and promote faster rates of swell.
Temperature	Increasing temperatures cause moisture to diffuse to cooler areas beneath pavements and buildings.

Table 2.3. Stress Conditions that Influence Swelling Potential (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
Stress History	An overconsolidated soil is more expansive than the same soil at the same void ratio, but normally consolidated.
Loading	Magnitude of surcharge load determines the amount of volume change that will occur for a given moisture content and density. An externally applied load acts to balance interparticle repulsive forces and reduces swell.
Soil Profile	The thickness and location of potentially expansive layers in the profile considerably influence potential movement. Greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone.

2.4. Oedometer Methods to Measure Swelling Properties

Oedometer methods are the easiest and most widely used methods in practice for measuring the swelling properties of soils. Three alternative test methods are presented in ASTM D 4546 for the determination of the magnitude of swell for soil samples. These test methods require that a soil specimen be restrained laterally and loaded axially in an oedometer with access to free water. Initially, terminology of the experiments is presented.

Swell: Increase in elevation or dilation of soil column following sorption of water.

Free Swell, %: Percent heave, $\Delta h/h*100$, following sorption of water at the seating pressure.

Primary Swell: An arbitrary short-term swell usually characterized as being completed at the intersection of the tangent of reverse curvature to the curve of a dimensional change-logarithm of time plot with the tangent to the straight line portion representing long-term or secondary swell (Figure 2.9).

Secondary Swell: An arbitrary long term swell usually characterized as the linear portion of a dimensional change-logarithm of time plot following completion of short-term or primary swell (Figure 2.9).

Swell Pressure: A pressure which prevents the specimen from swelling.

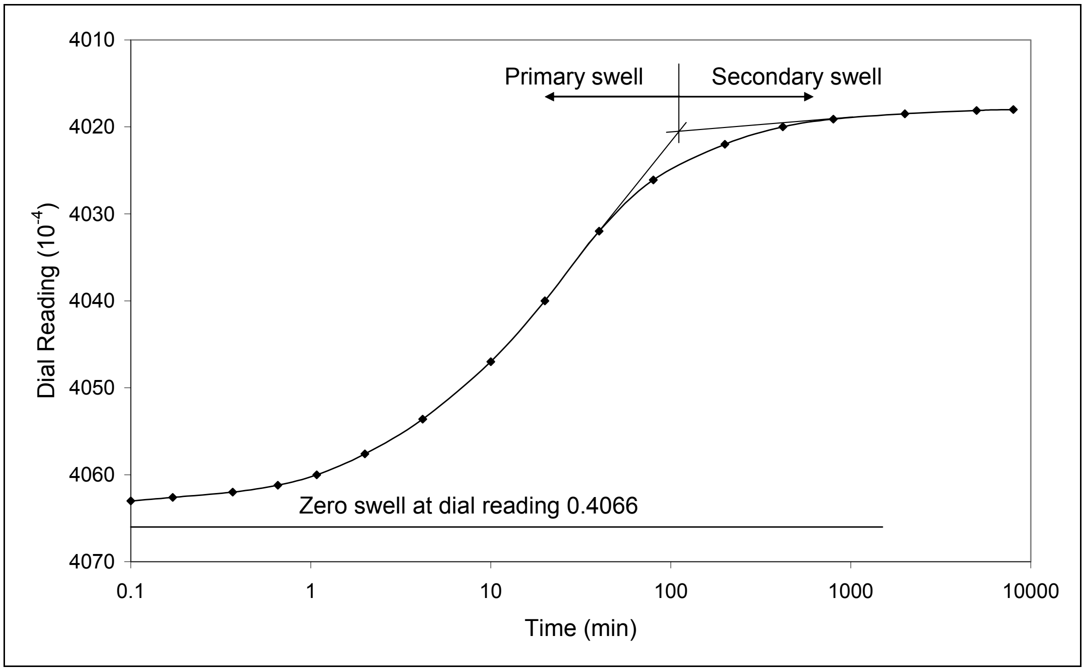

Figure 2.9. Time – Swell Curve

2.4.1. Method A

The specimen is inundated and allowed to swell vertically at the seating pressure applied by the weight of the top porous stone and the loading plate. Readings of swell are taken at 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 minutes and 1, 2, 4, 8, 24, 48, and 72 hours. The specimen is left to swell till the end of primary swell (Figure 2.10, Step 3 – 4). After completion of primary swell a vertical pressure of approximately 5, 10, 20, 40, 80, etc., kPa is applied until the specimen is recompressed to its initial void ratio/height (Figure 2.10, Step 4 – 6).

Method A can be modified to place an initial vertical stress, σ_1, on the specimen equivalent to the estimated vertical pressure on the in situ soil within 5 minutes of placing the seating pressure and securing the zero deformation reading. Deformation is read within 5 minutes of placing the vertical pressure (Figure 2.10, Step 1 – 2). Then the vertical stress is removed, except for the seating pressure. Deformation is recorded within 5 minutes after removal of σ_1 (Figure 2.10, Step 2 – 3), the specimen is inundated, and the test is continued as explained in the preceding paragraph.

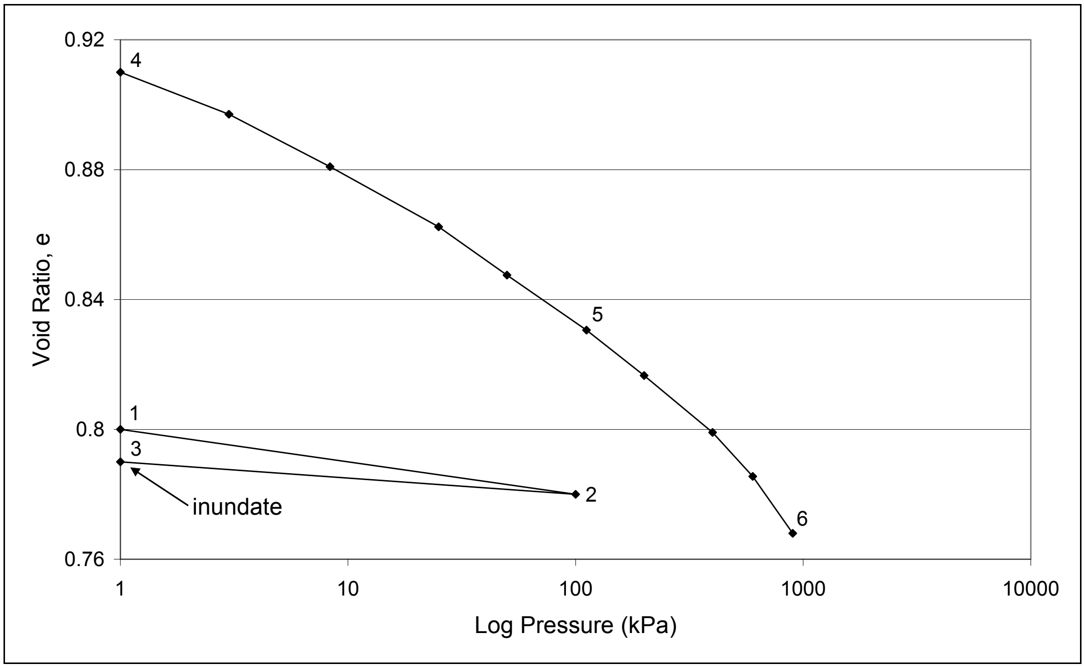

Figure 2.10. Void Ratio – Log Pressure Curve for Method A

This method measures (a) the primary swell, (b) percent heave for vertical confining pressures up to the swell pressure, and (c) the swell pressure.

2.4.2. Method B

A vertical pressure exceeding the seating pressure is applied within 5 minutes of placing the seating pressure. Deformation is read within 5 minutes of placing the vertical pressure (Figure 2.11, Step 1 – 2). The specimen is inundated immediately after the deformation is read and deformation is recorded after elapsed times similar to Method A until primary swell is complete (Figure 2.11, Step 2 – 3). The specimen is loaded vertically up to its initial void ratio/height as in Method A (Figure 2.11, step 3 – 5).

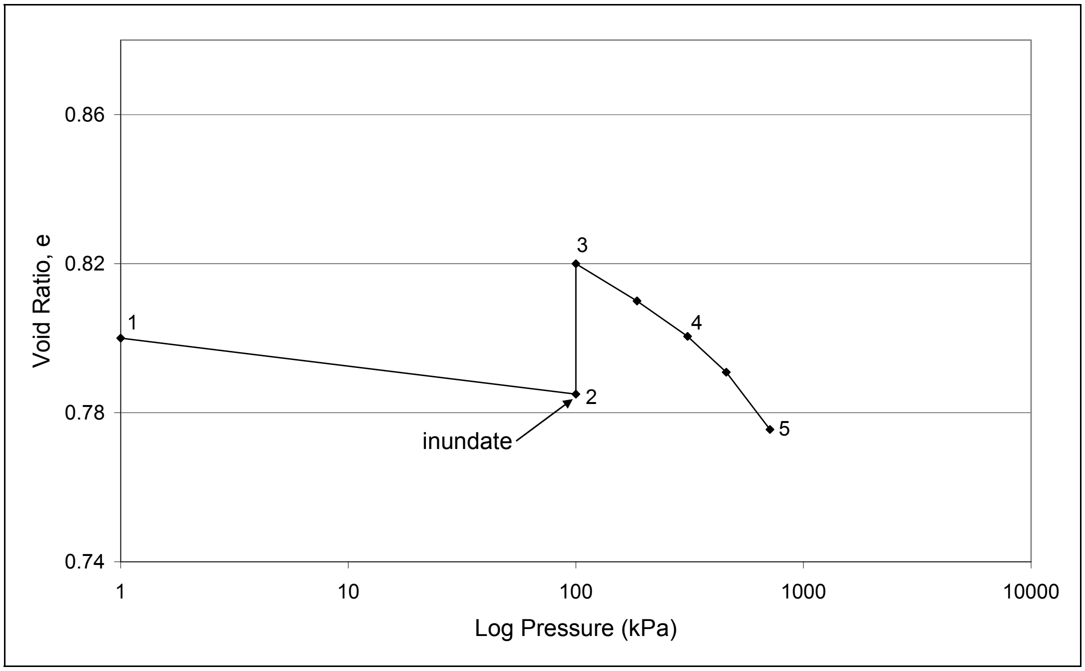

Figure 2.11. Void Ratio – Log Pressure Curve for Method B

This method measures (a) the percent heave or settlement for vertical pressure usually equivalent to the estimated in situ vertical overburden and other vertical pressure up to the swell pressure, and (b) the swell pressure.

2.4.3. Method C

An initial vertical pressure, σ_1, which is equivalent to the estimated vertical in situ pressure or swell pressure is applied within 5 minutes after placement of the seating pressure. Deformation is read within 5 minutes after placing σ_1 (Figure 2.12, step 1 – 2), and the specimen is immediately inundated with water. Increments of vertical stress as needed to prevent swell is applied (Figure 2.12, Step 2 – 3). The specimen is loaded vertically as in Method A (Figure 2.12, Step 3 – 7). The rebound curve following consolidation is determined (Figure 2.12, after Step 7).

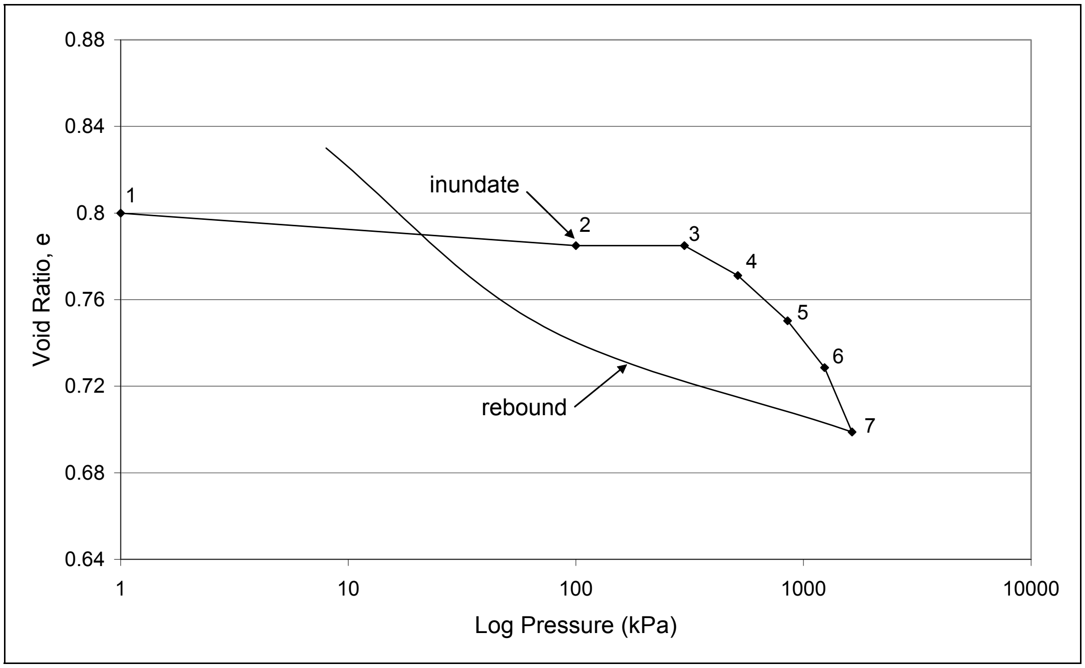

Figure 2.12. Void Ratio – Log Pressure Curve for Method C

This method measures (a) the swell pressure, (b) preconsolidation pressure, and (c) percent heave or settlement within the range of applied vertical pressures.

CHAPTER 3

SOIL STABILIZATION

Treatment procedures that are available for stabilizing expansive soils include:

- Prewetting:
  Prewetting is based on the theory that increasing the moisture content in the expansive foundation soils will cause heave to occur prior to construction and thereby eliminate problems afterward (Chen, 1975).

- Moisture Control:
  Moisture control methods are applied around the perimeter of structures in an attempt to minimize edge wetting or drying of foundations (Das, 1990).

- Soil Replacement:
  Removal of expansive soils and replacement with nonexpansive soils is one method to provide stable foundation material (Nelson and Miller, 1992).

- Compaction Control:
  Expansive soils expand very little when compacted at low densities and high water contents, but expand greatly when compacted at high densities and low water contents (Abduljauwad, 1993; Chen, 1975).

- Chemical Stabilization:
  Chemical additives, such as lime, cement, fly ash, and other chemical compounds, have been used in soil stabilization for many years at various degrees of success. The effectiveness of these additives depends on the soil conditions, stabilizer properties, and type of construction (i.e. houses, roads, etc.). The selection of a particular additive depends on costs, benefits, availability, and practicality of its application (Al-Rawas et al. 2002).

Fly ash, desulphogypsum, and lime were used to stabilize the expansive soil in this study.

3.1. Fly Ash Stabilization

Large quantities of coal are being burnt in thermal power stations to meet the ever increasing demand for thermal power. Combustion of coal results in a residue consisting of inorganic mineral constituents and organic matter which is not fully burned. The inorganic mineral constituents from ash: About 80% of this ash is fly ash. Environmentally safe disposal of large quantities of ash is not only tedious but also expensive. To reduce the problems of disposal, great efforts are being made to utilize fly ash. The use of fly ash as a soil-stabilizing agent is beneficial for improving the engineering properties of the soil, while at the same time it provides an opportunity for the utilization of an industrial waste that will otherwise require costly disposal (Ferreira et al. 2003; Nalbantoğlu and Güçbilmez, 2002; Sivapullaiah et al. 1998).

Fly ash produced from the burning of pulverized coal in a coal-fired boiler is a fine-grained, powdery particulate material that is carried off in the flue gas and collected from the flue gas by means of electrostatic precipitators (TFHRC, 2003; Vassilev et al. 2003).

Fly ash is useful in many construction applications because it is a pozzolan, meaning it is a siliceous or alimino-siliceous material which in itself possess little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (ASTM, 1993).

A microscopic view of fly ash reveals mainly glassy spheres with some crystalline and carbonaceous matter. The principal chemical constituents are silica (SiO_2), alumina (Al_2O_3), ferric oxide (Fe_2O_3), and calcium oxide (CaO). Other components are magnesium oxide (MgO), sulfur trioxide (SO_3), titanium oxide (TiO_2), alkalies (Na_2O and K_2O), phosphorous oxide (P_2O_5), and carbon (related to loss on ignition). Water added to fly ash usually creates an alkaline solution, with pH in the range 6 to 11.

Because of the variations in coals from different sources, as well as the differences in the design of coal-fired boilers, not all the fly ash is the same. Factors affecting the physical, chemical, and engineering properties of fly ash include (TFHRC, 2003):

- Coal type and purity
- Degree of pulverization
- Boiler type and operation
- Collection and stockpiling methods

Two classes of fly ash are defined in ASTM C 618: Class F fly ash, and Class C fly ash. Class F fly ash is normally produced from burning anthracite or bituminous coal. This class fly ash has pozzolanic properties. Class C fly ash is normally produced from burning lignite or subbituminous coal. This class of fly ash, in addition to having pozzolanic properties, also has some self-cementing properties, meaning that it has ability to harden and gain strength in the presence of water alone. Typical chemical compositions of Class F and Class C fly ashes are given in Table 3.1.

Table 3.1. Typical Chemical Compositions of Class F and Class C Fly Ashes (expressed as percent by weight) (TFHRC, 2003).

Component	Class F Fly Ash	Class C Fly Ash
SiO_2	20 – 60	40 – 60
Al_2O_3	5 – 35	10 – 30
Fe_2O_3	10 – 40	4 – 15
CaO	1 – 12	5 – 30
MgO	0 – 5	1 – 6
SO_3	0 – 4	0 – 4
Na_2O	0 – 4	0 – 6
K_2O	0 – 3	0 – 4
Loss on Ignition	0 – 15	0 – 3

ASTM D 5239 classifies fly ashes into three categories according to their soil stabilization performances:

1) Non Self-Cementing (Class F) Fly Ash Stabilization

Non self-cementing fly ash, by itself, has little effect on soil stabilization. It is a poor source of calcium and magnesium ions. The particle size of fly ash may exceed that of the voids in fine-grained soils, precluding its use as a filler. However, this fly ash in poorly graded sandy soils may be a suitable filler and, as such, may aid in compaction, may increase density, and may decrease permeability.

2) Non Self-Cementing (Class F) Fly Ash Mixed With Cement or Lime

The advantage of adding fly ash to fine-grained soils, along with cement or lime, is for its pozzolanic properties and improved soil texture. Some clays are pozzolanic in nature and only require lime to initiate the pozzolanic reaction. The use of this fly ash is suitable with clays requiring lime modification, provided lime is added to promote the pozzolanic reaction.

3) Self-Cementing (Class C) Fly Ash Stabilization

This fly ash is a better source of calcium and magnesium ions although not as good as lime or Portland cement. Self-cementing fly ash contains varying amounts of free (uncombined) lime (0 to 7% CaO by weight) that can provide cation exchange and ion crowding to fine-grained soils when used in significant amounts. It has been used successfully to control swell potential of expansive soils. It has also been used to stabilize coarse-grained soils.

3.1.1. Soil – Fly Ash Reactions

The improvements in the engineering properties of soils as fly ash is added can be explained by two basic reactions: a) Short-term reactions, consisting of cation exchange and flocculation-agglomeration, and b) Long-term reactions, involving pozzolanic activity.

a) Short-Term Soil-Fly Ash Reactions

When fly ash is added to a clay soil it has an immediate effect on the properties of the soil due to the cation exchange at the surfaces of the clay particles. Lime (CaO) of the fly ash quickly reacts with water introducing both Ca^{2+} cations and OH^- anions to the soil-fly ash system.

$$CaO + H_2O \longrightarrow Ca(OH)_2$$

$$Ca(OH)_2 \longrightarrow Ca^{2+} + 2(OH)^-$$

Ca^{2+} replaces all other monovalent cations (Li^+, Na^+, H^+, K^+, etc.) associated with the surfaces of the clay particles. The increase in valance around the clay particles results in a reduction in the extent of the hydrous double layers surrounding them. This alteration in the density of the electrical charge around the clay particles leads to them being attracted closer to eachother to form flocs, the process being termed flocculation. The formation of flocs and agglomerates takes place within the first few hours after fly ash addition and causes a significant reduction in the swelling of the soil.

b) Long-Term Soil-Fly Ash Reactions

The long-term reactions make use of fly ash's being a pozzolan. The pozzolanic activity of fly ash is mainly due to the reactions between reactive silica in the fly ash and free lime which produce calcium-silica-hydrates (CSH) (Sivapullaiah et al. 1998).

$$Ca^{2+} + 2(OH)^- + SiO_2 \text{ (Fly Ash Silica)} \longrightarrow CSH$$

The resulting cementitous calcium-silica-hydrates cause further amelioration of the soil by binding adjacent soil grains together. Pozzolanic reactions occur over a period of time (many weeks, months, or even years may be required for the completion of these reactions).

Pozzolanic reactivity is found to vary from one fly ash to another. Factors affecting the pozzolanic reactivity of fly ash include (Bell, 1993; Sivapullaiah et al. 1998):

- Amount of reactive silica in the fly ash
- Presence of moisture and free lime
- Fineness of the fly ash (surface area)
- Low carbon content, low hydrogen ion concentration, iron content
- The degree of self-hardening of the fly ash is also highly dependent on the ash's density, temperature and age.

All the silica present in fly ash is not readily available for reaction with lime. The significance of fly ash is largely because it is a source of reactive silica. This reactive silica in fly ash appears to be due to presence of a special microstructure of quartz named silica W, having a micro-amorphous fibrous silica structure along with an amorphous silica structure. The rest of silica present in fly ash is in the crystalline form of quartz or in association with alumina as mullite ($3Al_2O_3.2SiO_2$) and is not readily available for reactions with free lime. While the free lime content of fly ash can be supplemented, there is little that can be done to increase the silica content. Thus reactive silica present in fly ash should form a basis for assessing the pozzolanic reactivity of the fly ash (Sivapullaiah et al. 1998).

Silica which is soluble in about 2 to 3N hydrochloric acid can be taken as reactive silica. The acid-soluble silica can easily and accurately be determined by plasma emission spectrometry using a very small quantity of fly ash (Sivapullaiah et al. 1998).

3.2. Desulphogypsum Stabilization

In the last three decades, there has been a continuous effort by electric utility companies to reduce sulfur dioxide (SO_2) emissions from coal burning power plants (Sahu et al. 2002). To achieve the desired concentration of sulfur dioxide in the exhaust gases, they are processed in desulfurization plant. The most widely used method of removal of sulfur dioxide is the treatment of the flue gas with calcium oxide (CaO). In this process, known as flue gas desulfurization (FGD), calcium reacts with sulfur dioxide to produce hannebachite ($CaSO_3.1/2H_2O$) and/or gypsum ($CaSO_4.2H_2O$). The resulting gypsum is named desulphogypsum (Oman et al. 2002). The overall FGD reaction can be represented by the following (Chen, 1995):

$$CaO + H_2O \longrightarrow Ca(OH)_2$$
$$SO_2 + H_2O \longrightarrow H_2SO_3$$
$$H_2SO_3 + Ca(OH)_2 \longrightarrow CaSO_3.2H_2O$$
$$CaSO_3.2H_2O + 1/2O_2 \longrightarrow CaSO_4.2H_2O$$

FGD process generate voluminous desulphogypsum solid wastes that are usually landfilled, occupying thousands of acres of land and creating serious land pollution problems (Tao et al. 2001). The American Coal Ash Association reported for United States that less than 10% of desulphogypsum is currently used beneficially for gypsum binders, plasters and plasterboards manufacture, as well as an additive in Portland cement production (Clark et al. 2001; Galos et al. 2003). Utilization of desulphogypsum in geotechnical applications will be useful in decreasing the excessive stocks which cause environmental pollution, besides it will also provide a new and economical way to improve the engineering properties of soils.

Having the same chemical composition with natural gypsum, desulphogypsum contains impurities such as the finer fractions of fly ash. These impurities may be located in the crystal structure of desulphogypsum or may be sticked to the surface of the crystal structure. Chemical composition of these impurities vary according to the type and properties of the fuel and sorbent used, and the type of boiler (Galos et al.2003; Özkul, 2000; Sahu et al. 2002).

In general, including its impurities, desulphogypsum can be characterized as an alkaline material consisting of excess sorbent (either calcitic or dolomitic limestone), calcium oxide, calcium hydroxide (portlandite), calcium sulfate (anhydrite), calcium sulfite, magnesium sulfate (epsomite), magnesium oxide (periclase), and fly ash (Crews, 1998).

Desulphogypsum contains excess lime, and when it is mixed with soil, this lime will generate the same short-term and long-term reactions that will be discussed in the following section on lime stabilization.

3.3. Lime Stabilization

Lime stabilization is the most widely used means of chemically transforming unstable soils into structurally sound construction foundations. The use of lime in stabilization creates a number of important engineering properties in soils, including improved strength; improved resistance to fracture, fatigue, and permanent deformation; improved resilient properties; reduced swelling; and resistance to the damaging effects of moisture. The most substantial improvements in these properties are seen in moderately to highly plastic soils, such as heavy clays (Little et al. 2003).

All types of clay minerals react with lime. The addition of lime may not make much difference in kaolinitic clay soils, but it can have a significant effect in clay soils containing montmorillonite. In fact, expansive clays tend to react readily with lime, losing plasticity immediately (Bell and Coulthard, 1990). This is because expansive clay minerals such as montmorillonite exhibit a high cation exchange capacity, whereas non-expansive clay minerals like kaolinite have a relatively low cation exchange capacity.

3.3.1. Soil – Lime Reactions

The addition of lime to a soil initiates a two stage reaction. Short-term reactions show their effect right after the addition of lime, while long term reactions are accompanied by a period of time.

The short-term effect of the addition of lime to a clay soil is to cause flocculation and agglomeration of the clay particles, as explained in Section 3.1.1 on fly ash stabilization, for cation exchange takes place between the metallic ions of the clay particles and the calcium ions of the lime. It is this process which is primarily responsible for the modification of the engineering properties of clay soils when they are treated with lime (Bell, 1993, 1996).

When lime is added to a clay soil, it must first satisfy the affinity of the soil for lime, that is, ions are adsorbed by clay minerals and are not available for pozzolanic reactions until this affinity is satisfied. Because this lime is fixed in the soil and is not available for other reactions, the process has been referred to as lime fixation (Hilt and Davidson, 1960). The lime fixation point corresponds with the point where further addition of lime does not bring about further changes in the plastic limit. This therefore is the optimum addition of lime needed for maximum modification of the soil. Beyond this point excess lime can only produce cementitious compounds, which bind the flocculated particles and develop extra strength (Al-Rawas et al. 2002; Bell, 1996; Mathew and Rao, 1997).

The long-term reactions are the pozzolanic reactions. The addition of lime to soil produces a highly alkaline environment, due to the OH^- anions from the hydration of lime, which gives rise to a slow solution of silica and alumina from clay particles (Kinuthia et al.1999; Mathew and Rao, 1997). The cementation process develops from the reaction between calcium present in lime and dissolved silica and alumina from soil, forming calcium-silica-hydrates (CSH), calcium-alumino-hydrates (CAH), and calcium-alumino-silica-hydrates (CASH) (Nalbantoğlu and Tuncer, 2001).

$$Ca^{2+} + 2(OH)^{-} + SiO_2 \text{ (Clay Silica)} \longrightarrow CSH$$
$$Ca^{2+} + 2(OH)^{-} + Al_2O_3 \text{ (Clay Alumina)} \longrightarrow CAH$$
$$Ca^{2+} + 2(OH)^{-} + SiO_2 \text{ (Clay Silica)} + Al_2O_3 \text{ (Clay Alumina)} \longrightarrow CASH$$

These cementitious reaction products contribute to flocculation by bonding adjacent soil particles together and as curing occurs they strengthen the soil. Such pozzolanic reactions are time and temperature dependent, with strength developing gradually over a long period of time (in some instances this may take several years).

Extended curing times and elevated temperatures promote pozzolanic reactions, and thereby produce additional cementing agents. Conversely, if the temperature falls below around +4°C, pozzolanic reactions are retarded and may cease at lower temperatures (Bell, 1996). Hence, pozzolanic reactions may remain dormant during periods of low temperatures, but regain reaction potential when temperatures increase.

The development of cementitious products from pozzolanic reactions should be a permanent, non-reversible phenomenon.

Carbonation is the reaction of lime with carbon dioxide from the air to form weak cementing agents, notably calcium carbonate ($CaCO_3$). This is undesirable (Bell, 1993). Becoming unstable, calcium-silica, calcium-alimino and calcium-alumino-silica hydrates may react with carbon dioxide to revert back to silica, alumina, and calcium carbonate if the pH of the lime-stabilized soil drops sufficiently low. Carbonation reactions are harmful to the long-term strength and durability of the lime-stabilized soil. Using sufficient amount of lime (to provide enough alkalinity), compaction of the soil to high density and prompt placement after mixing lime with soil (to minimize carbon dioxide penetration) can minimize potential carbonation problems.

In a lime treated soil, if sulfates are present in the soil or water, then these sulfates may react with alumina released from clay and calcium from lime to form ettringite ($Ca_6[Al(OH)_6]_2.(SO4)_3.26H_2O$) (Mohamed, 2000). Ettringite has the capability of imbibing large volumes of water and dramatically increases the swelling potential of the lime-stabilized soil. It would be prudent to test and understand whether lime, soil, and sulfate swell or not when mixed and exposed to moisture, if sulfates are present where lime stabilization is applied.

CHAPTER 4

EXPERIMENTAL STUDY

4.1. Purpose

The purpose of the experimental study is to investigate the effects of the addition of fly ash, desulphogypsum, and lime on grain size distribution, Atterberg limits, swelling potential, and rate of swell of an expansive soil; and to investigate the effect of curing on swelling potential and rate of swell of an expansive soil treated with fly ash, desulphogypsum, and lime.

4.2. Material

Kaolinite: Kaolinite was obtained from the factory of ESAN Industrial Minerals Company of Eczacıbaşı in the form of gravel sized grains. These grains were crushed and passed through No. 40 sieve before usage.

Bentonite: Bentonite (Na – Montmorillonite) was obtained from Karakaya Bentonite Factory. Bentonite was passed through No. 40 sieve before usage.

Fly Ash: Fly ash was taken from Çayırhan Thermal Power Plant. Fly ash was passed through No. 40 sieve before usage. Fly ash is Class C, its specific gravity is 2.17.

Desulphogypsum: Desulphogypsum was taken from Çayırhan Thermal Power Plant. Desulphogypsum was passed through No. 40 sieve before usage. Specific gravity of desulphogypsum is 2.08.

Lime: Commercially available hydrated lime was used. Lime was passed through No. 40 sieve before usage. Specific gravity of lime is 2.76.

The chemical analyses of Çayırhan fly ash and desulphogypsum were done by 'Cement Producers Association of Türkiye'. The results of the chemical analyses are presented in Table 4.1.

Table 4.1. Results of the Chemical Analyses of Çayırhan Fly Ash and Desulphogypsum (expressed as percent by weight)

Component	Çayırhan Fly Ash	Çayırhan Desulphogypsum
SiO_2	50.38	2.03
Al_2O_3	14.06	0.52
Fe_2O_3	9.90	0.21
CaO	13.25	31.91
MgO	1.20	0.42
SO_3	3.16	43.13
Na_2O	3.18	-
K_2O	1.97	-
TiO_2	0.90	-
P_2O_5	0.58	-
Loss on Ignition	0.86	20.88

4.3. Preparation of Samples

Expansive soil sample used in this study was prepared in the laboratory. The expansive soil sample was composed of 85% kaolinite and 15% bentonite. This sample, which gave a significant swell percentage, was designated as ‘Sample A’ (Figure 4.1). In the beginning of the preliminary studies fly ash and desulphogypsum were pre-tested to examine their capability of being used as soil stabilizers. It came to light that both fly ash and desulphogypsum were capable of reducing the swelling percentage of Sample A. Being the most widely used soil stabilizing agent, lime was decided to be necessary for being the checkpoint to compare how big the effects of fly ash and desulphogypsum on the expansive soil were.

All the materials used in this study were oven-dried for 1 day at 30°C, and were ground so that they could pass through No. 40 sieve. Each sample was prepared by mixing a calculated amount of stabilizer with Sample A to obtain a sample with predetermined percentage of stabilizer which varied from 0 to 30 percent (by dry weight of the sample) for fly ash and desulphogypsum, and 0 to 8 percent (by dry weight of the sample) for lime.

To prepare the samples the predetermined amount of materials were first mixed roughly using a trowel. Each time only 150 gr of each sample was mixed, as mixing higher amounts could prevent the particles from distributing uniformly in the mixtures. Then 10% (15 gr) water was added to the mixtures to form the samples, and to be able to mix such fine grained soil samples thoroughly, the constituents were sieved two times through No. 16 sieve (Figure 4.1).

For the experiments on cured samples, the samples prepared according to the above procedure were tightly encased by a plastic bag to prevent loss of moisture and were set to cure in the desiccator for 7 days and 28 days.

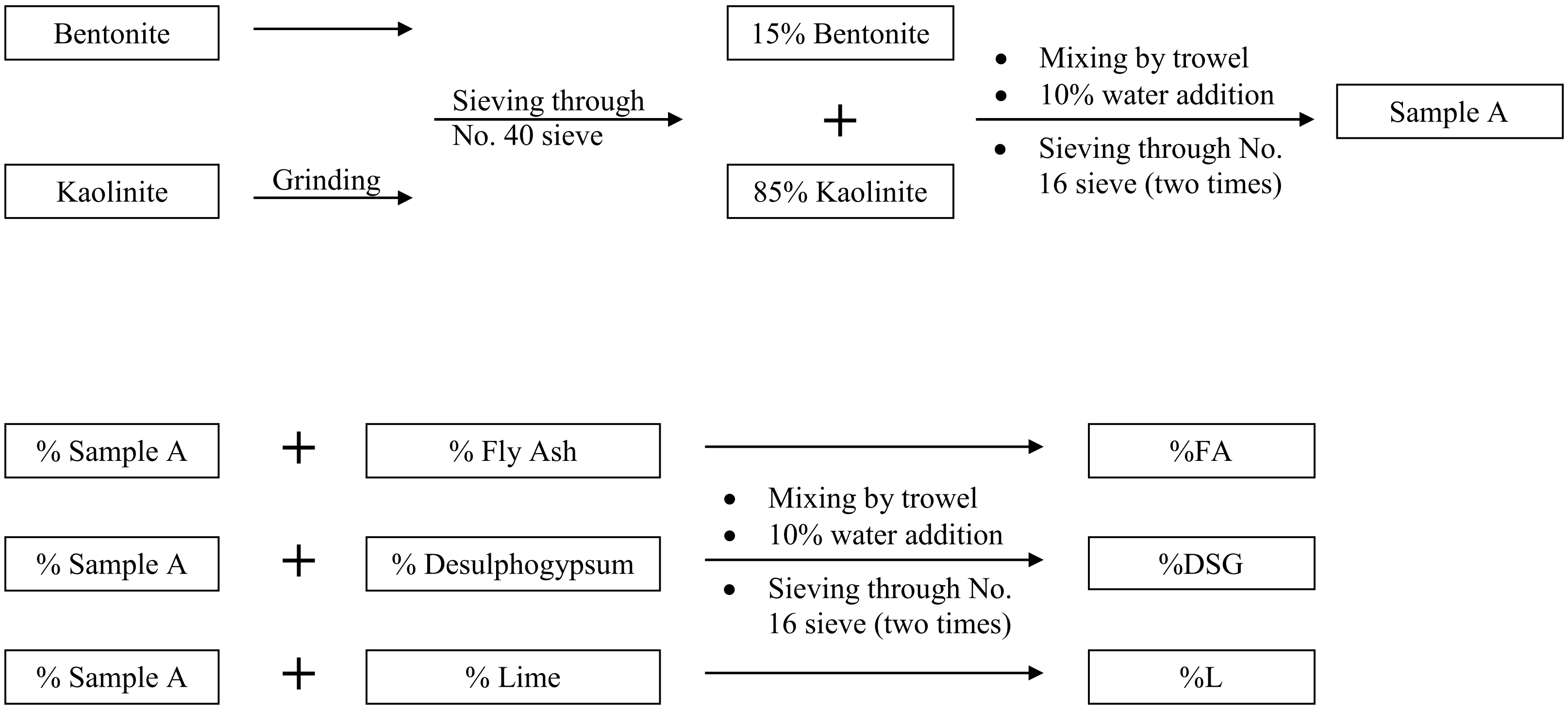

Figure 4.1. Preparation of Samples

4.4. Sample Properties

To determine the sample properties hydrometer tests, Atterberg limit tests, and specific gravity tests were applied to the samples according to the test procedures specified in ASTM D 2435.

The sample properties are tabulated in Table 4.2.

Clay and silt fractions of the fly ash and lime added samples are determined from the grain size distribution curves by hydrometer analyses; however for the desulphogypsum added samples hydrometer tests gave no result due to the precipitation of the samples at the bottom of the hydrometer flask within the first few hours of the tests. The rate of precipitation increased with increasing desulphogypsum percentage. With the minimum amount of desulphogypsum additive (5% of Sample A by weight) the precipitation became significant in about 3 hours, while with the maximum desulphogypsum addition (30% of Sample A by weight) the precipitation completed in the first 15 minutes of the test. Hence, grain size distribution curves, and clay and silt fractions are not available for the desulphogypsum added samples.

Soil classification is done according to the Unified Soil Classification System (Figure 4.2).

Swelling potential of the fly ash and lime added samples are calculated, using the PI values and clay percentages, according to the classification chart of Seed et al. (1962) (Figure 4.3).

Table 4.2. Sample Properties

Sample	Clay (%)	Silt (%)	G_s	LL (%)	PL (%)	PI (%)	SL (%)	SI (%)	Soil Class.	Act.	Swelling Potential
A	44.2	55.8	2.51	92	21	71	18	74	CH	1.61	Very High
5%FA	42.8	57.2	2.48	68	22	46	19	49	CH	1.07	High
10%FA	40.8	59.2	2.46	67	24	43	20	47	CH	1.05	High
15%FA	39.0	61.0	2.45	63	24	39	22	41	CH	1.00	High
20%FA	37.0	63.0	2.44	59	25	34	23	36	CH	0.92	Medium
25%FA	35.0	65.0	2.41	56	27	29	23	33	CH	0.83	Medium
30%FA	33.0	67.0	2.40	53	27	26	25	28	CH	0.79	Medium
5%DSG	-	-	2.46	64	23	41	20	44	CH	-	-
10%DSG	-	-	2.42	63	23	40	21	42	CH	-	-
15%DSG	-	-	2.40	59	24	35	21	38	CH	-	-
20%DSG	-	-	2.39	58	24	34	21	37	CH	-	-
25%DSG	-	-	2.38	57	25	32	23	34	CH	-	-
30%DSG	-	-	2.36	54	26	28	23	31	CH	-	-
1%L	42.1	57.9	2.53	82	25	57	20	62	CH	1.35	High
3%L	40.0	60.0	2.54	60	31	29	25	35	MH	0.73	Medium
5%L	35.5	64.5	2.56	55	33	22	27	28	MH	0.62	Low
8%L	30.2	69.8	2.57	51	34	17	30	21	MH	0.56	Low

A: Expansive Soil Sample (85% Kaolinite + 15% Bentonite)

FA: Fly Ash; DSG: Desulphogypsum; L: Lime

Act: Activity = PI / % Clay

Naming is explained with the following two examples:

5%FA ⟶ 95% Sample A + 5% Fly Ash

10%FA ⟶ 90% Sample A + 10% Fly Ash

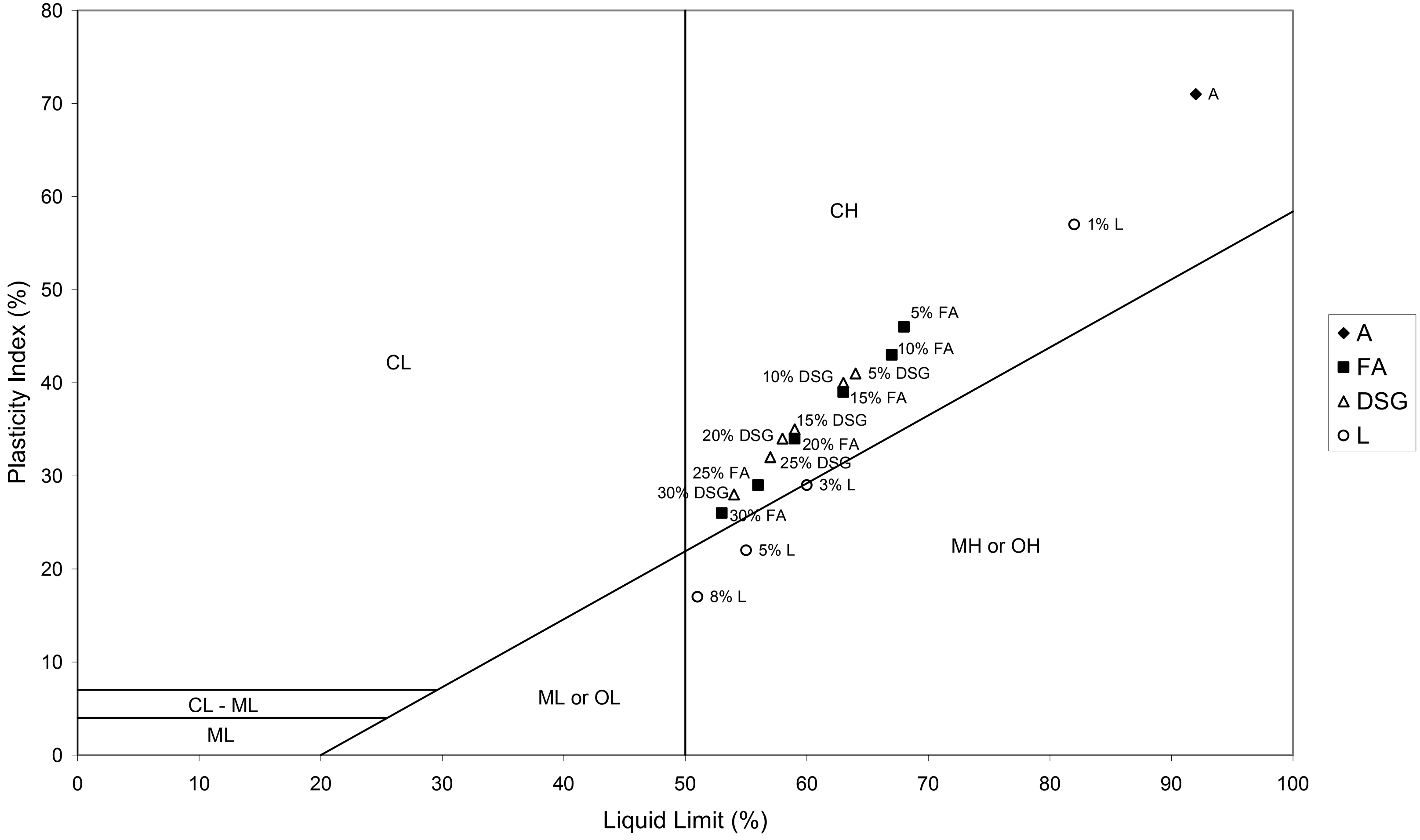

Figure 4.2. Plasticity Chart: Unified System

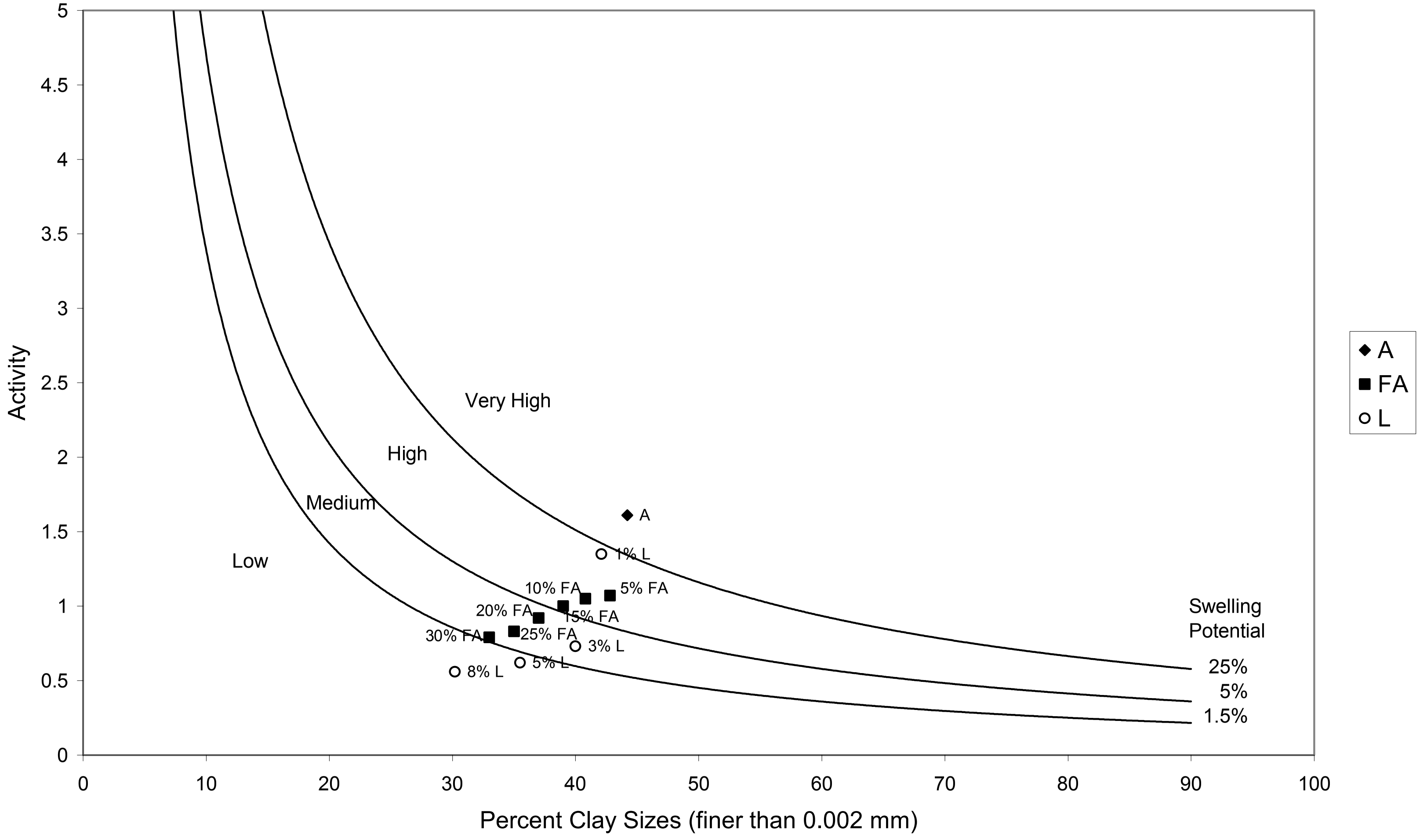

Figure 4.3. Classification Chart for Swelling Potential (Seed et al. 1962)

Grain size distribution curves of the fly ash and lime added samples are plotted separately (Figure 4.4, Figure 4.5). Grain size distribution curve of Sample A is plotted on both of the graphs to be able to examine the shifting of the curves due to the addition of the stabilizers. Grain size distribution curve of fly ash is also plotted on the same graph with the fly ash added samples; however although no problem was encountered with the lime added samples, hydrometer analysis of lime gave no result again due to the precipitation of lime at the bottom of the hydrometer flask, therefore grain size distribution curve of lime is not available.

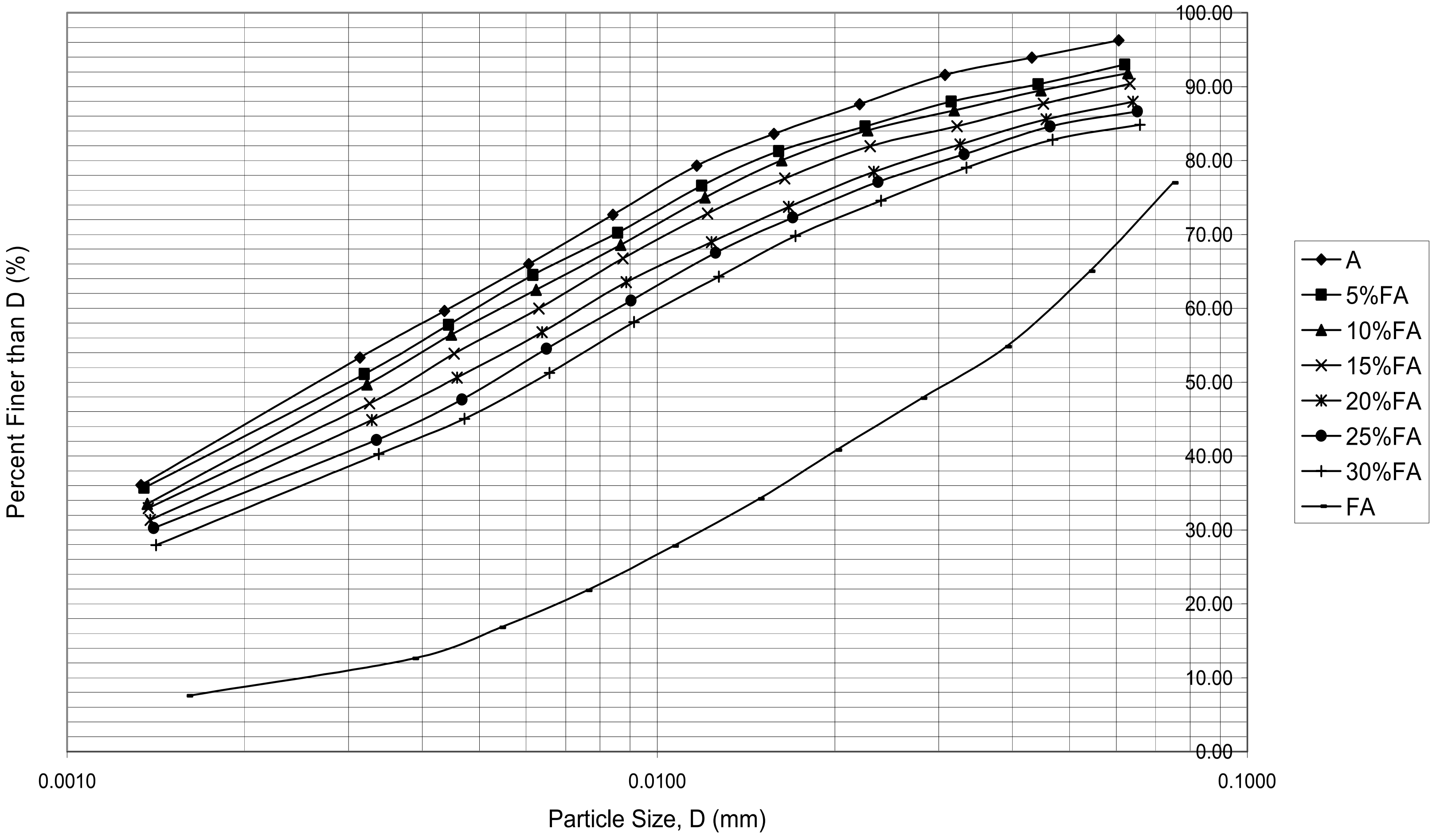

Figure 4.4. Grain Size Distribution Curves of Fly Ash Added Samples

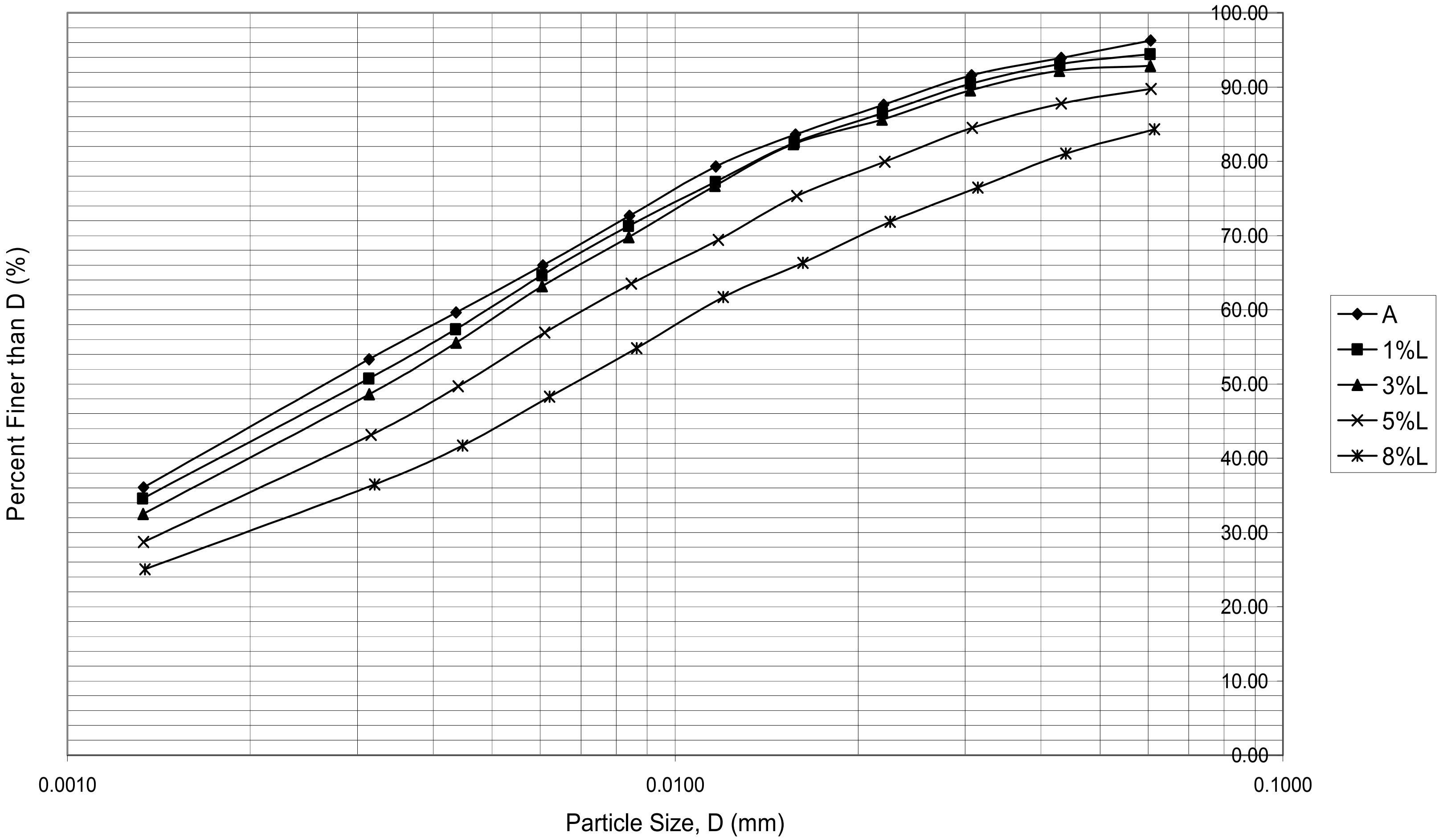

Figure 4.5. Grain Size Distribution Curves of Lime Added Samples

4.5. Test Procedure

In this study 'Free Swell' method was used for the determination of swelling properties of the soil samples. In order to apply this method the samples were prepared as specified in Section 4.3 and were statically compacted in a guide ring satisfying a bulk density of 1.80 Mg/m^3 and a dry density of 1.64 Mg/m^3. The samples were then transferred into the consolidation rings with the help of this guide ring. In doing the compaction and transference the setup shown in Figure 4.6 was used. First the calculated amount of soil sample was placed in the guide ring and the piston was placed on the guide ring with Part C of it in contact with the sample. Then the sample was compressed by applying pressure from the top of the piston (Part A) using a hydraulic jack till Part B of the piston came into contact with

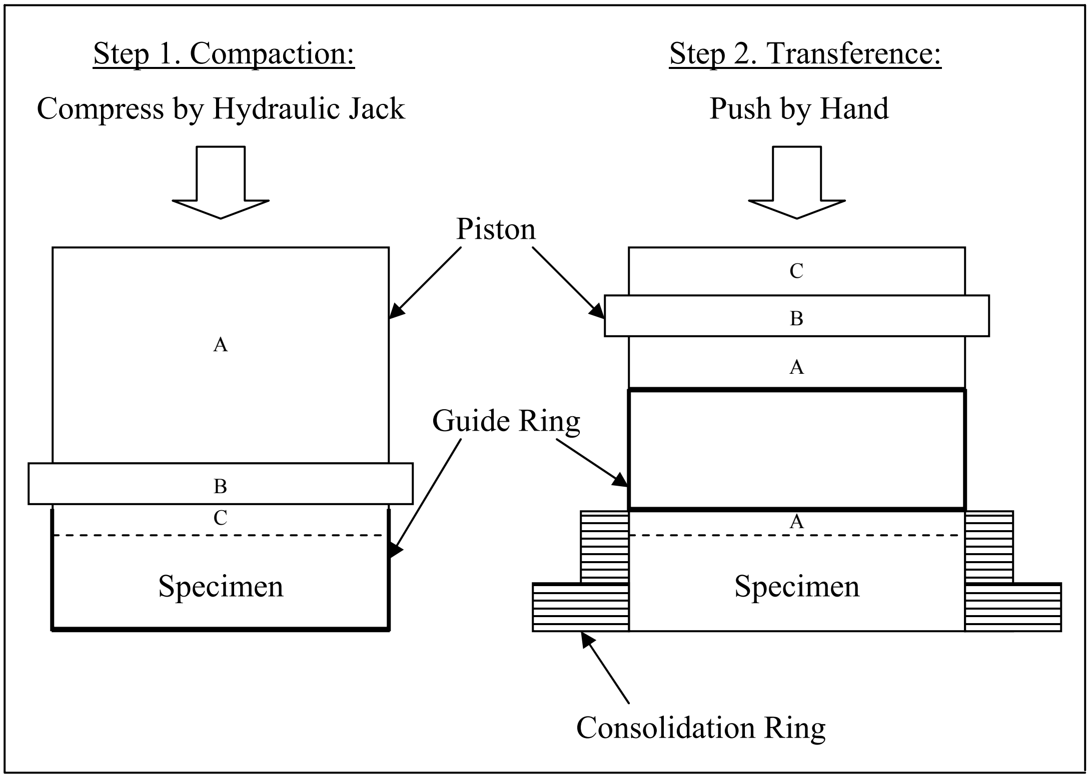

Figure 4.6. Static Compaction Setup

the guide ring (Figure 4.6, Step 1). After compaction finished the piston was removed and the guide ring, with the sample in it, was placed on the consolidation ring. The piston was again placed on the guide ring, this time with Part A of it in contact with the sample. By applying a strong and immediate push with hand from Part C of the piston the sample was pushed through the guide ring into the consolidation ring (Figure 4.6, Step 2).

4.5.1. Free Swell Test

The sample, which was compacted in the consolidation ring as explained above, was placed in the oedometer after placing dry filter papers on top and bottom of it. In placing the consolidation ring into the oedometer, air-dry porous stones were also placed on top and bottom of the sample (Figure 4.7). Then, the oedometer was mounted and the dial gauge was adjusted to zero reading. The sample was inundated by providing water through standpipes and by pouring water directly from the top of the oedometer. Swelling of the sample started right after the inundation of water. The sample was allowed to swell freely. As swelling continued deflections of the dial gauge was recorded. At some point the sample had no further tendency to swell and the maximum deflection recorded was used for the calculation of free swell. The percent of free swell was calculated from the following expression:

$$\text{Free Swell (\%)} = \Delta H/H*100$$

where ΔH = Change in initial height (H) of the sample

H = Initial height of the sample

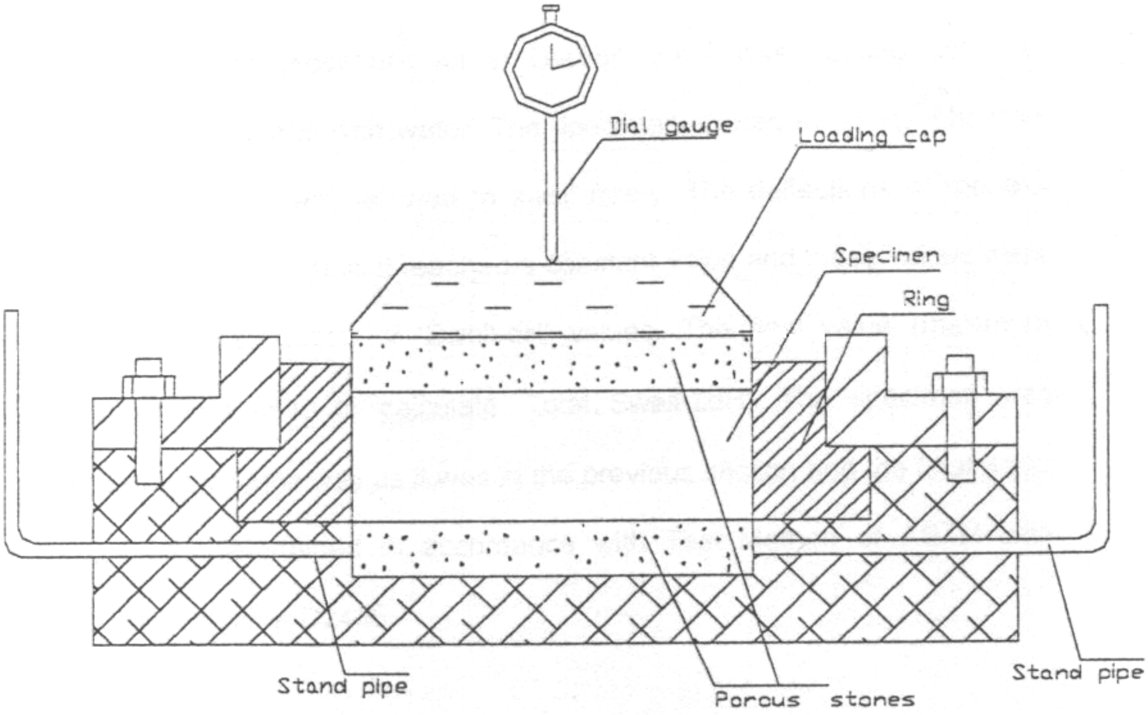

Figure 4.7. The Oedometer (İpek, 1998)

After swelling was complete the oedometer was dismantled and the consolidation ring was taken out. The filter papers were separated from the surface of the sample. The weight of the sample was measured and the sample was put in the oven to find its dry weight for the final water content determination.

4.6. Experimental Program

Upon the completion of the preliminary tests, the maximum and minimum amount of stabilizers to be added to Sample A were decided. Tests were decided to be performed on seventeen samples (Table 4.3).

Table 4.3. Samples Used in the Experimental Study

Fly Ash (FA)	Desulphogypsum (DSG)	Lime (L)
5% FA + 95% A (5%FA)	5% DSG + 95% A (5%DSG)	1% L + 99% A (1%L)
10% FA + 90% A (10%FA)	10% DSG + 90% A (10%DSG)	3% L + 97% A (3%L)
15% FA + 85% A (15%FA)	15% DSG + 85% A (15%DSG)	5% L + 95% A (5%L)
20% FA + 80% A (20%FA)	20% DSG + 80% A (20%DSG)	8% L + 92% A (8%L)
25% FA + 75% A (25%FA)	25% DSG + 75% A (25%DSG)	
30% FA + 70% A (30%FA)	30% DSG + 70% A (30%DSG)	

Experimental study was conducted in four phases:

1) Hydrometer tests, Atterberg limit tests, and specific gravity tests were applied to the samples.

2) Free swell tests were applied to the samples under the condition of no curing.

3) Free swell tests were applied to the samples after curing was applied for 7 days.

4) Free swell tests were applied to the samples after curing was applied for 28 days.

4.7. Test Results

The results of the liquid limit, plastic limit, and shrinkage limit tests are presented in Figures 4.8, 4.9, and 4.10 respectively for the fly ash, desulphogypsum, and lime added samples.

Variations of plasticity index (LL - PL) and shrinkage index (LL - SL) for the fly ash, desulphogypsum, and lime added samples are presented in Figures 4.11 and 4.12 respectively.

The effects of the addition of stabilizers on swelling percentage of Sample A for 0 days curing are given in Figure 4.13.

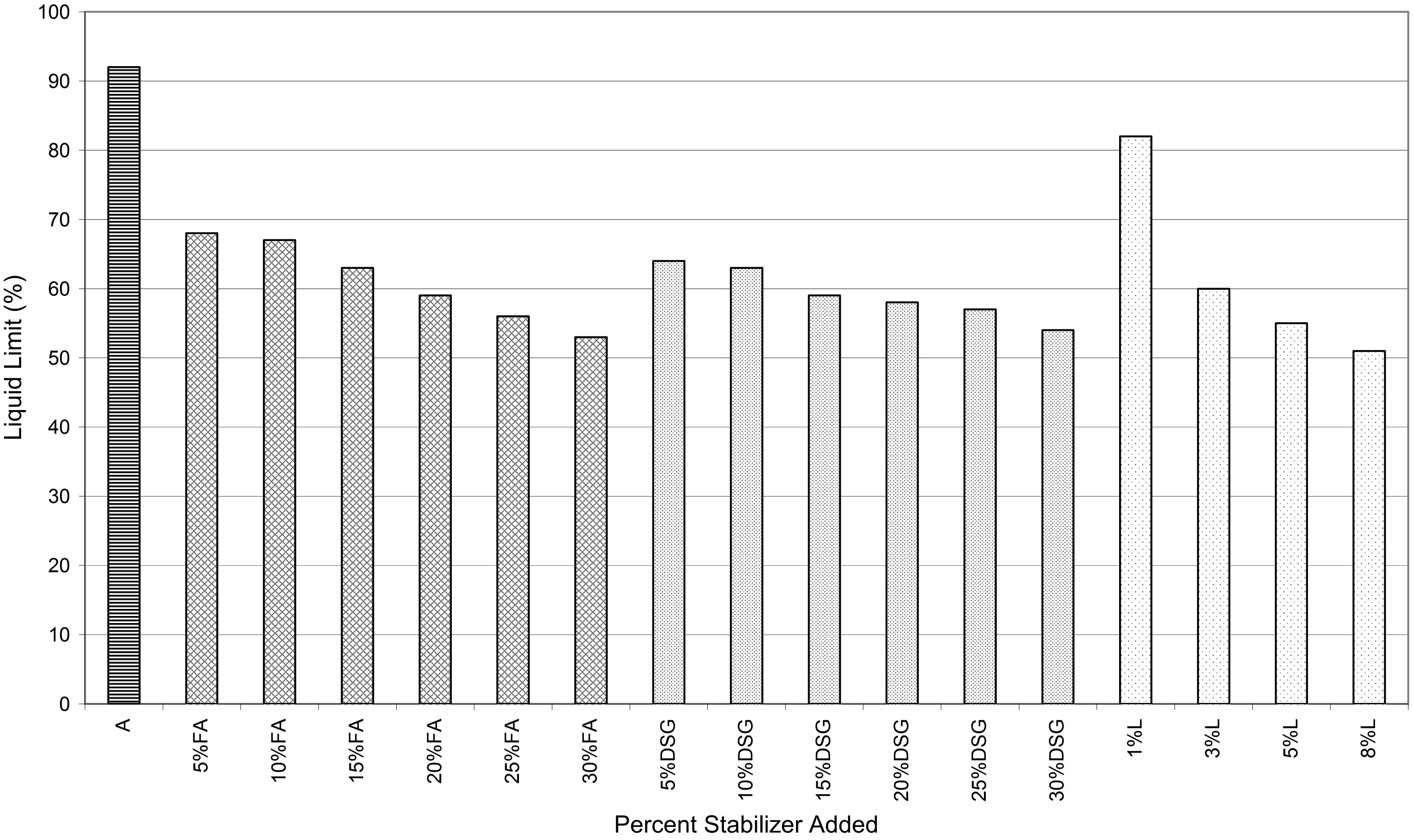

Figure 4.8. Effect of Fly Ash, Desulphogypsum, and Lime Addition on the Liquid Limit of Expansive Soil

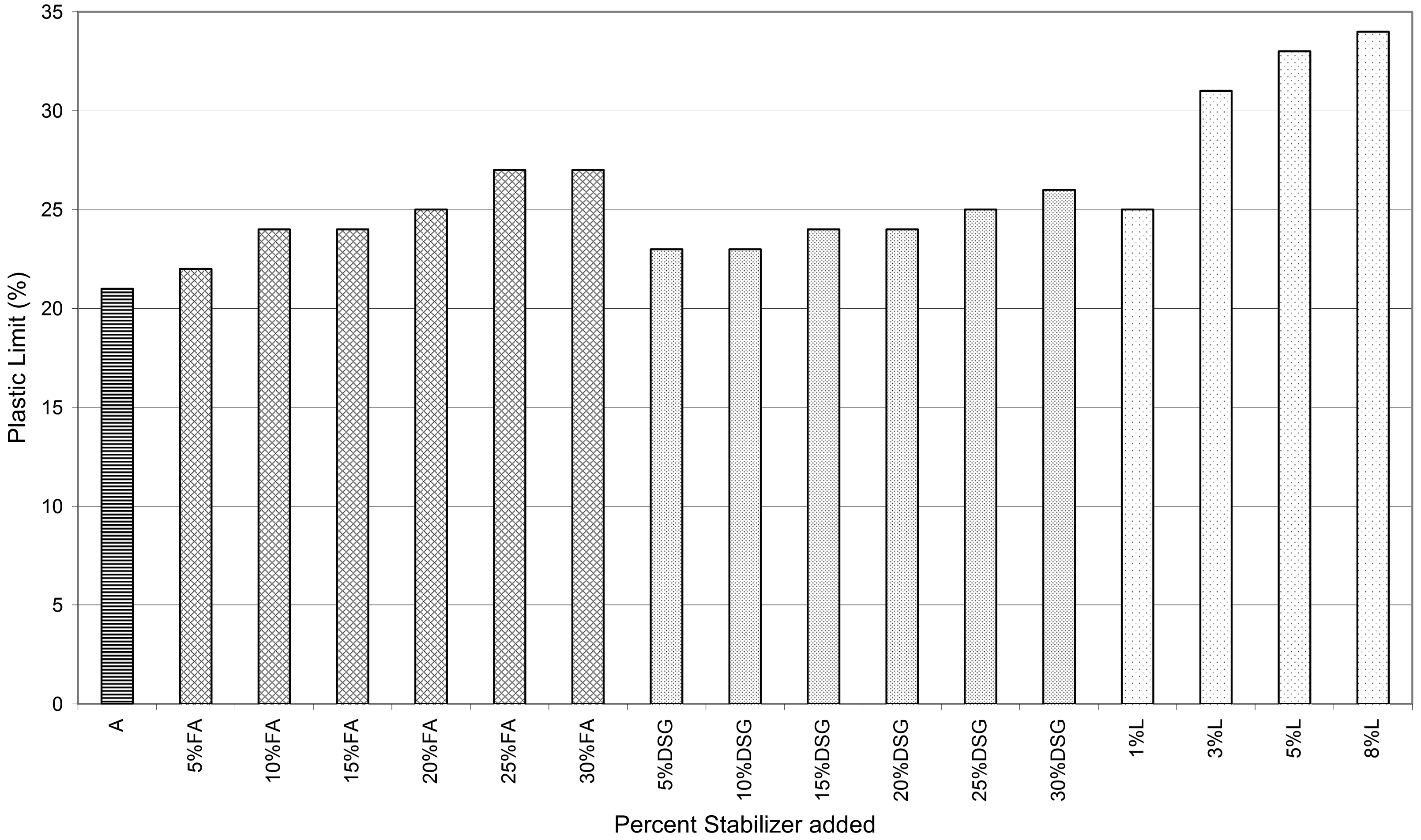

Figure 4.9. Effect of Fly Ash, Desulphogypsum, and Lime Addition on the Plastic Limit of Expansive Soil

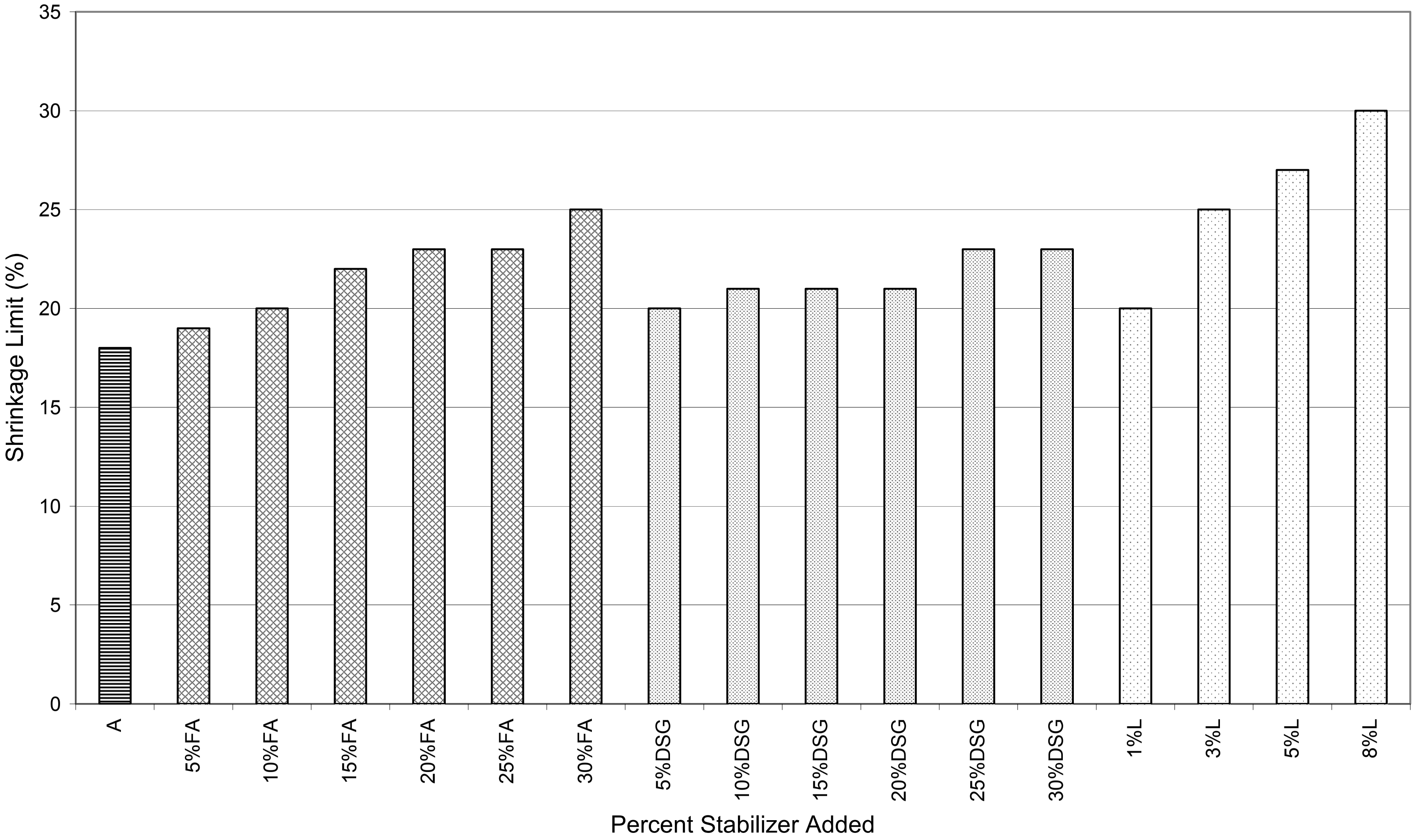

Figure 4.10. Effect of Fly Ash, Desulphogypsum, and Lime Addition on the Shrinkage Limit of Expansive Soil

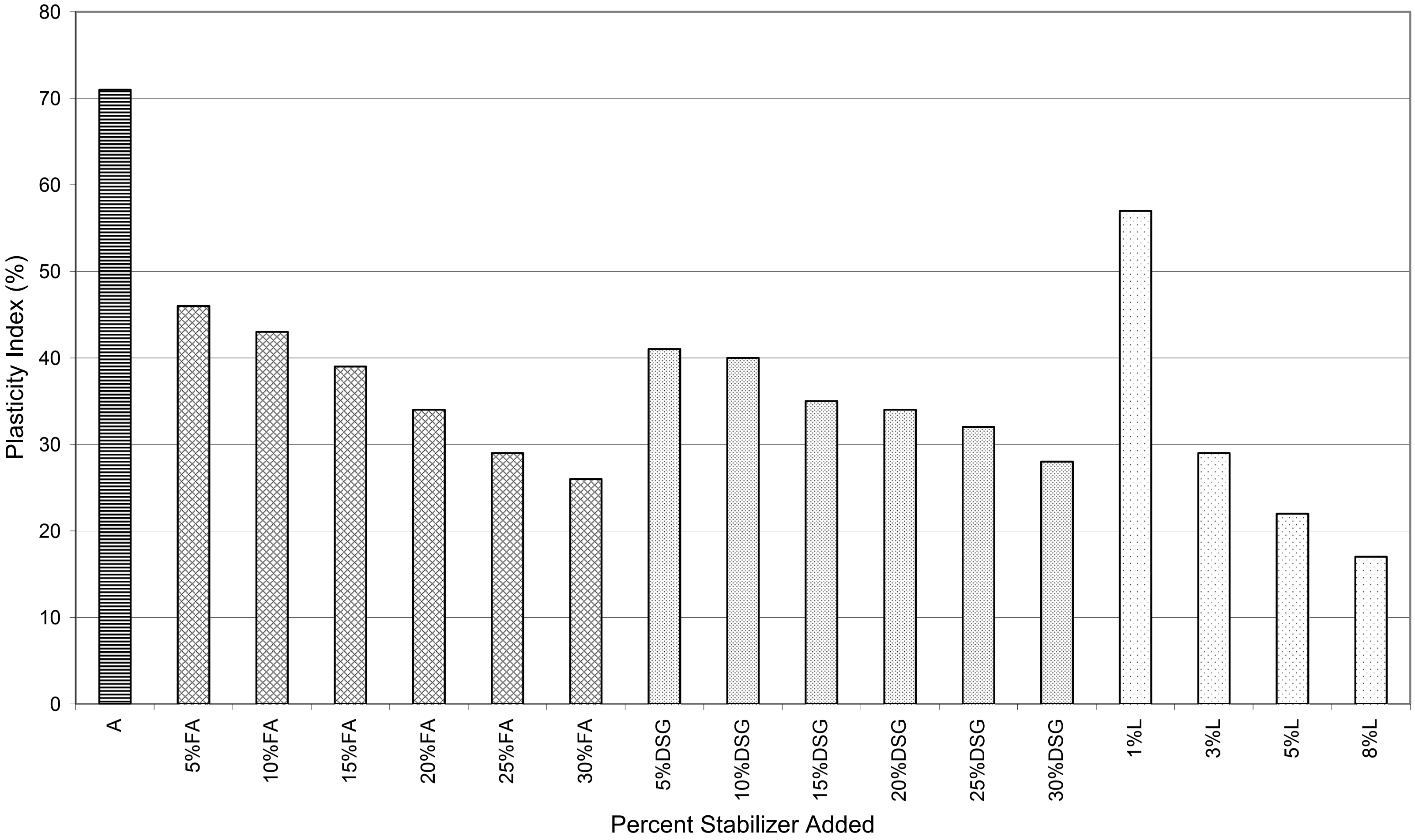

Figure 4.11. Effect of Fly Ash, Desulphogypsum, and Lime Addition on the Plasticity Index of Expansive Soil

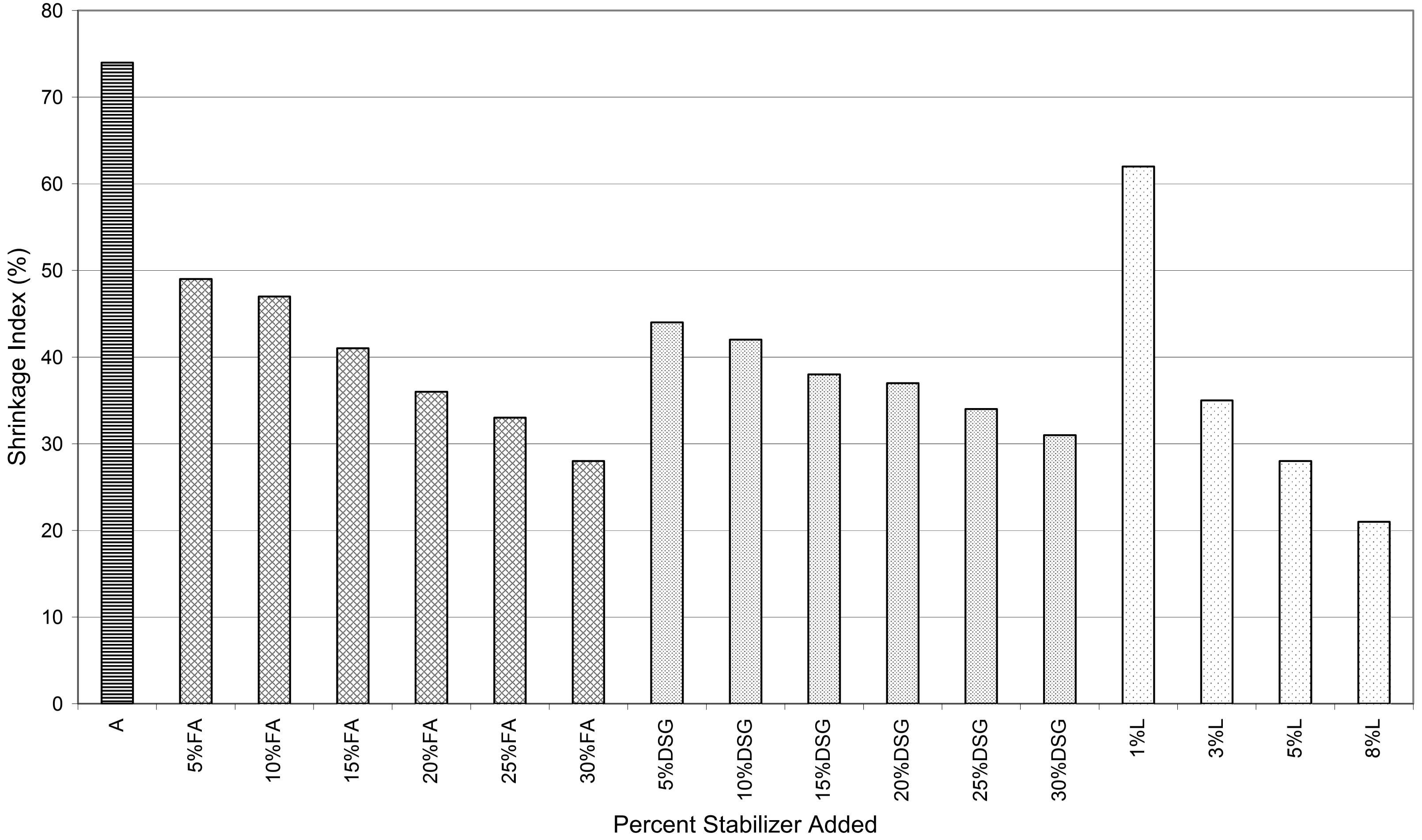

Figure 4.12. Effect of Fly Ash, Desulphogypsum, and Lime Addition on the Shrinkage Index of Expansive Soil

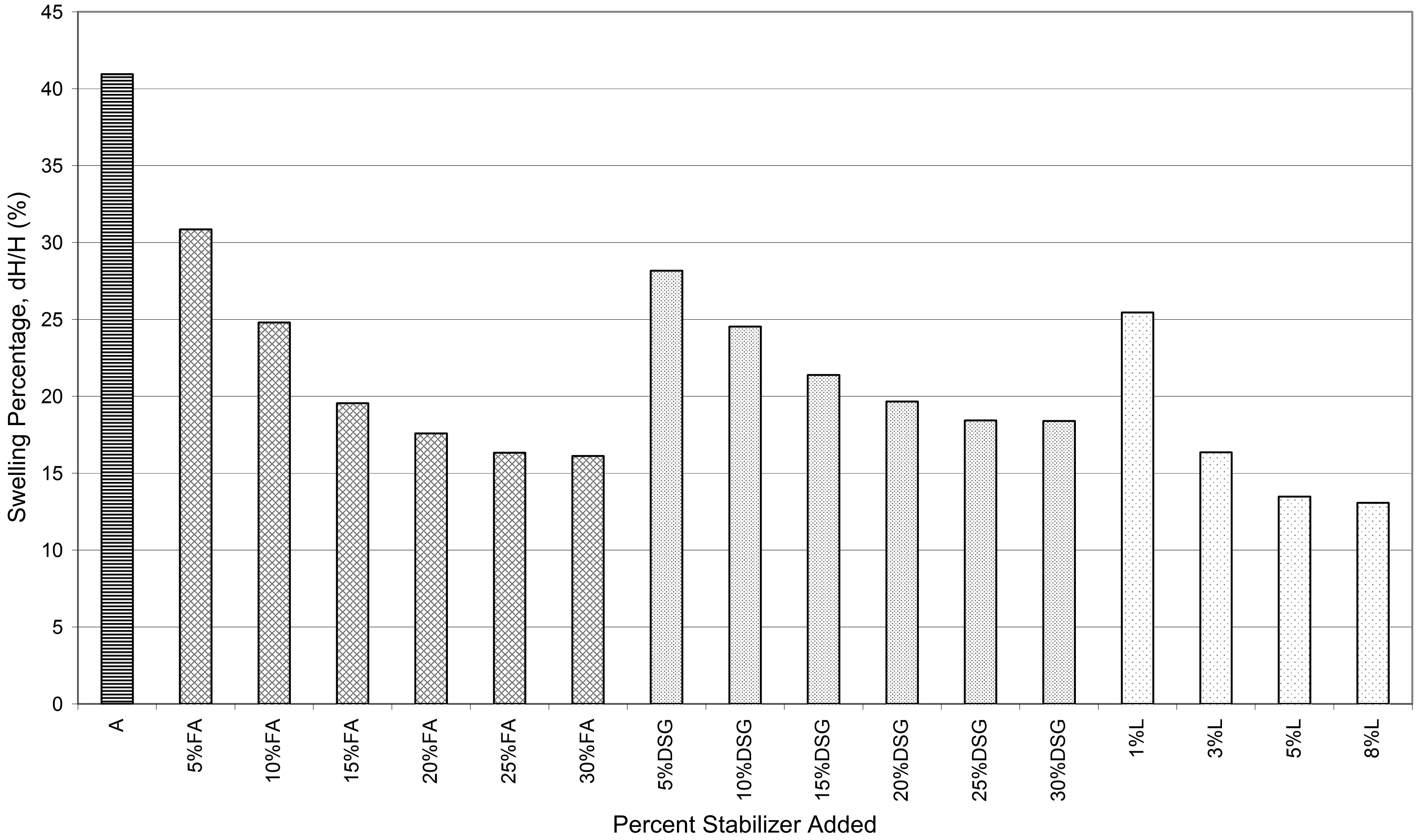

Figure 4.13. Variation of Swelling Percentage according to the Type and Amount of Stabilizer Added

Swelling percentage vs. time relationship for Sample A is presented in Figure 4.14. In order to plot dH/H vs. time graph dial readings were recorded during the free swell test as explained in Section 4.5.1. dH was calculated for each reading by subtracting the value read at that time from the initial reading value and dH/H vs. time was plotted.

Swelling percentage vs. time relationships of the fly ash, desulphogypsum, and lime added samples are given in Appendix A.

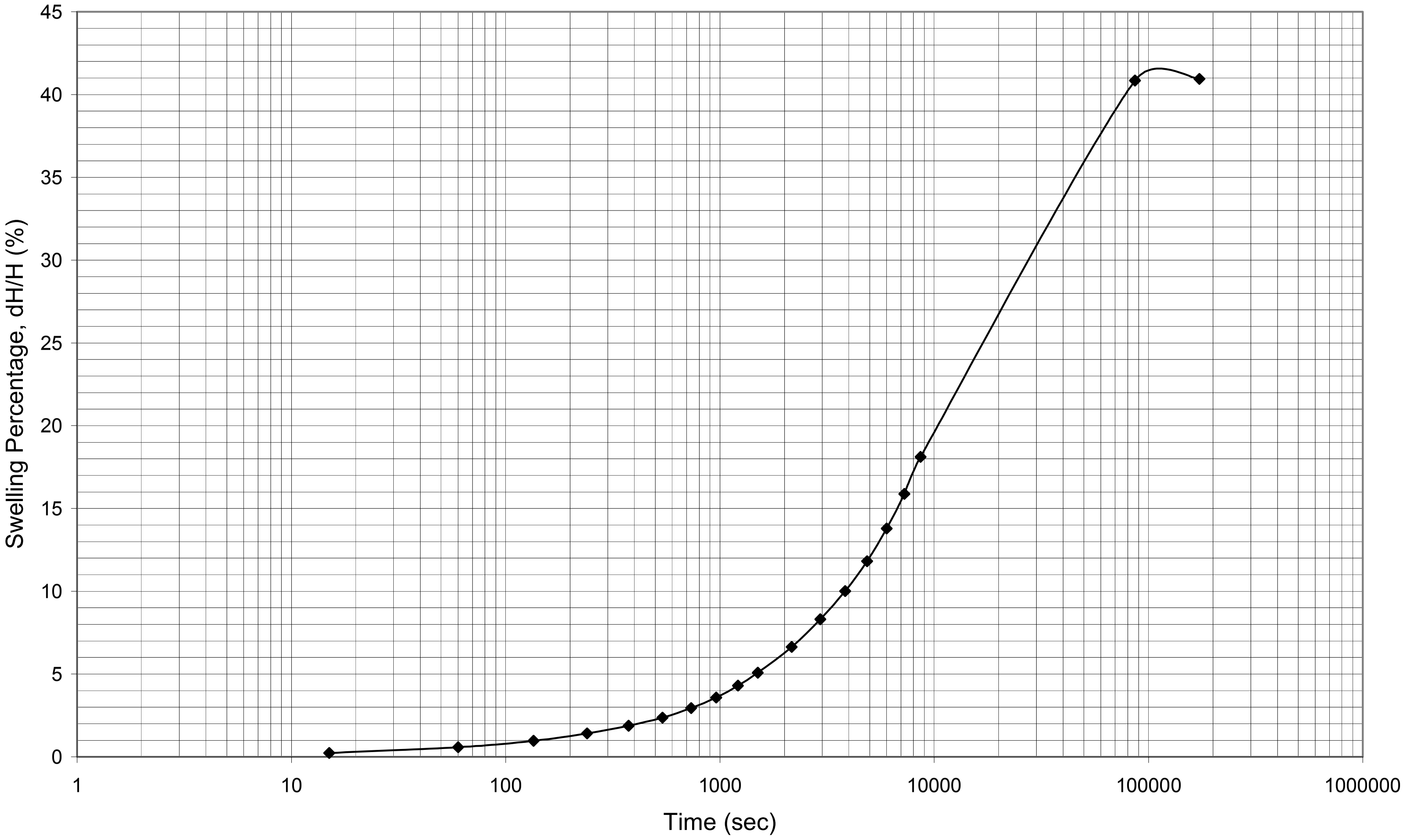

Figure 4.14. Swelling Percentage vs. Time Relationship for Sample A

There is no readily available method for measuring rate of swell. Therefore, for the evaluations of the results of this experimental study the rate of swell is defined as the time to reach 50 percent swell, t_{50}, i.e. the time elapsed to half the full swell.

The effects of the addition of fly ash, desulphogypsum, and lime on the rate of swell of expansive soil are presented in Figure 4.15.

Figures 4.16 and 4.17 show the effect of curing on the swelling percentages of fly ash, desulphogypsum, and lime added samples.

Figures 4.18 and 4.19 show the effect of curing on the rate of swell of fly ash, desulphogypsum, and lime added samples (corresponding swelling percentage vs. time graphs are given in Appendix B).

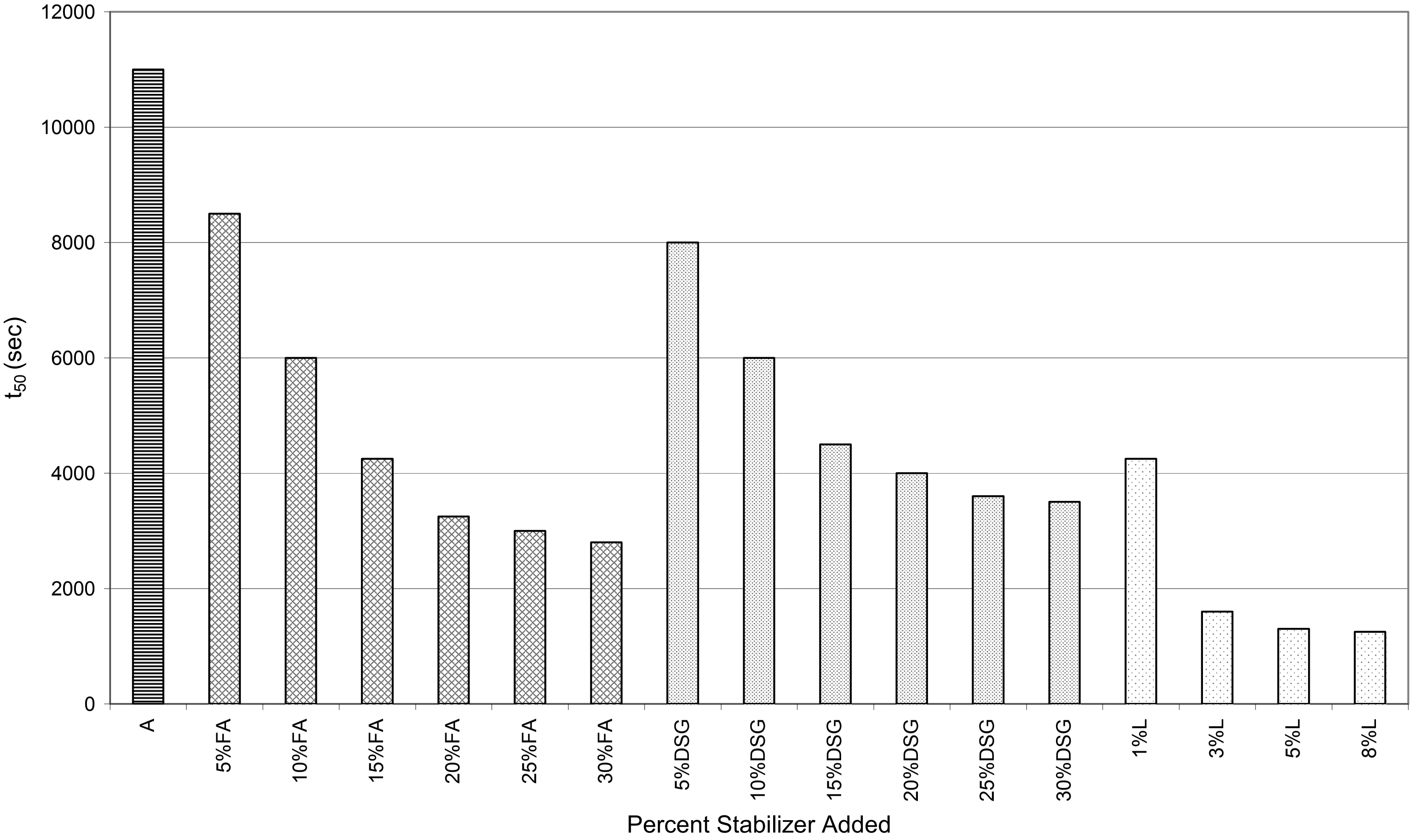

Figure 4.15. Variation of t_{50} with Type and Amount of Stabilizer Added

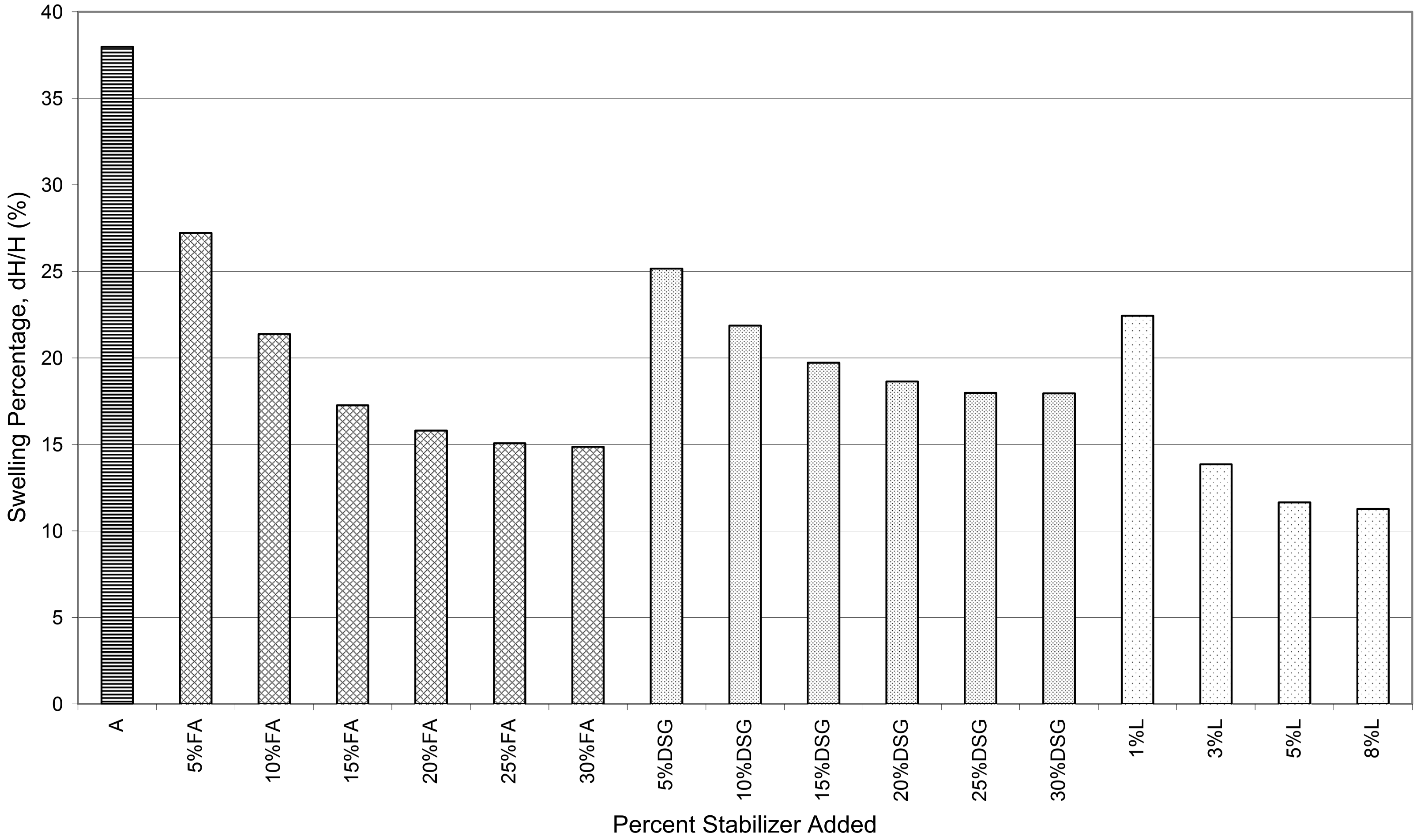

Figure 4.16. Variation of Swelling Percentage according to the Type and Amount of Stabilizer Added after 7 Days Curing

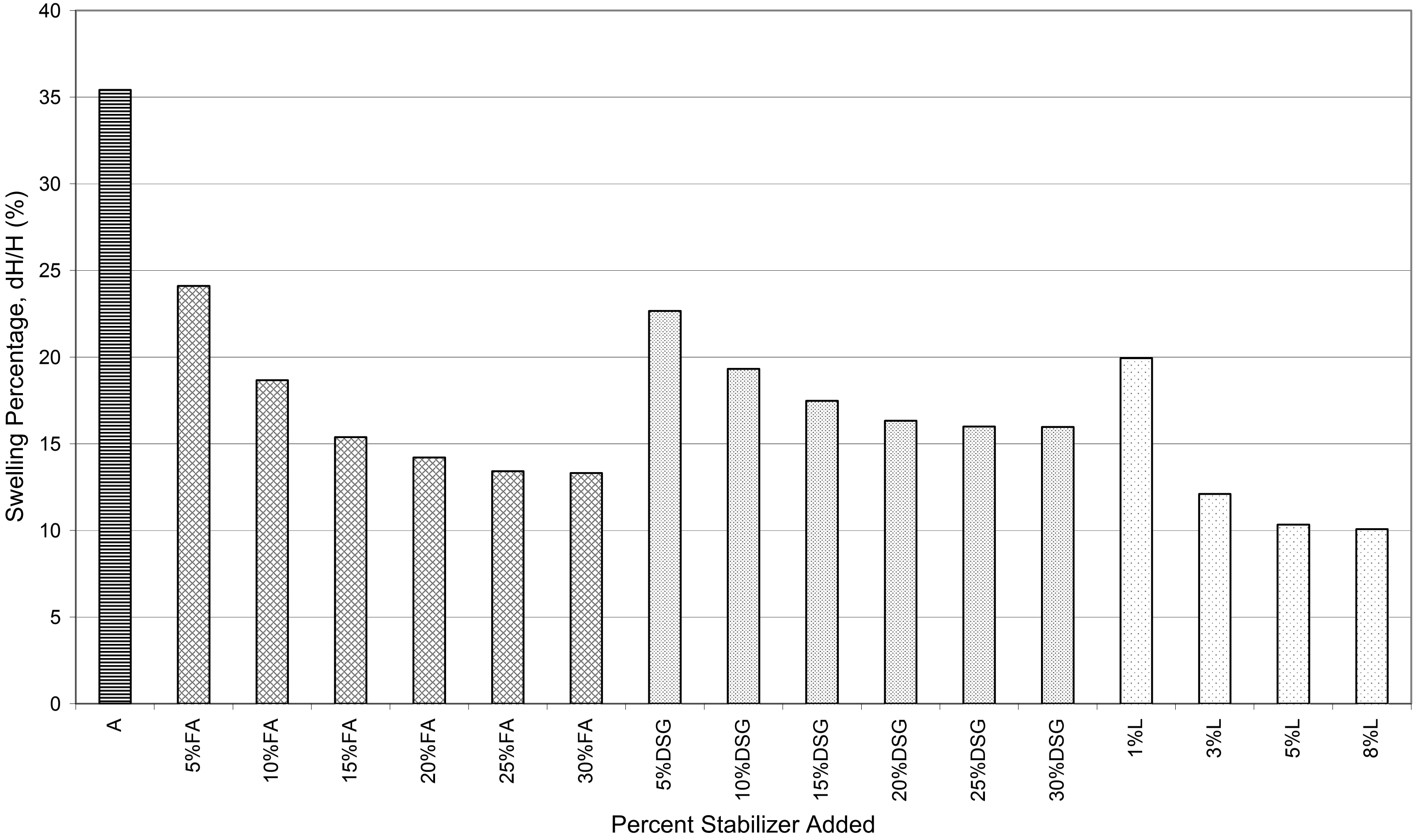

Figure 4.17. Variation of Swelling Percentage according to the Type and Amount of Stabilizer Added after 28 Days Curing

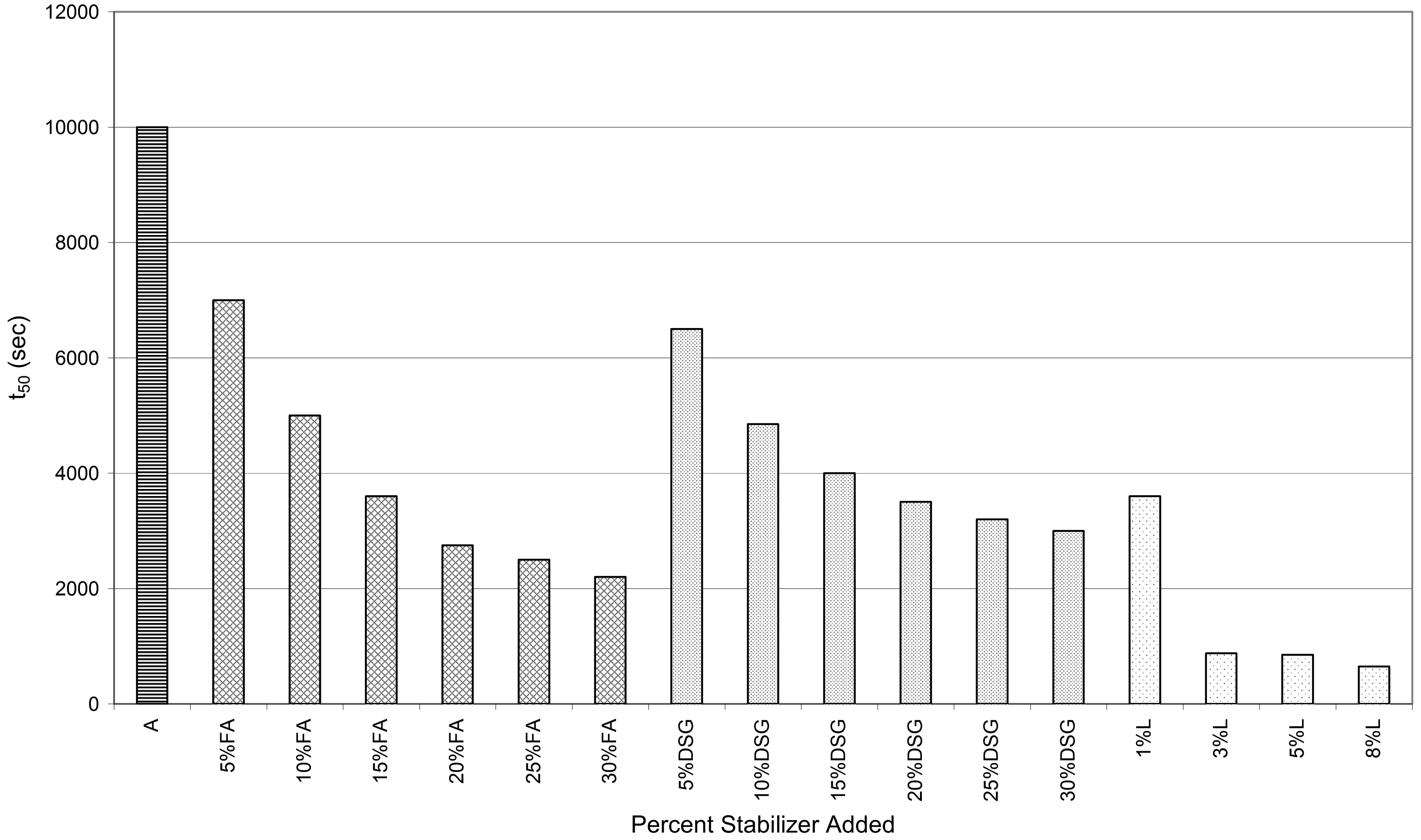

Figure 4.18. Variation of t_{50} with Type and Amount of Stabilizer Added after 7 Days Curing

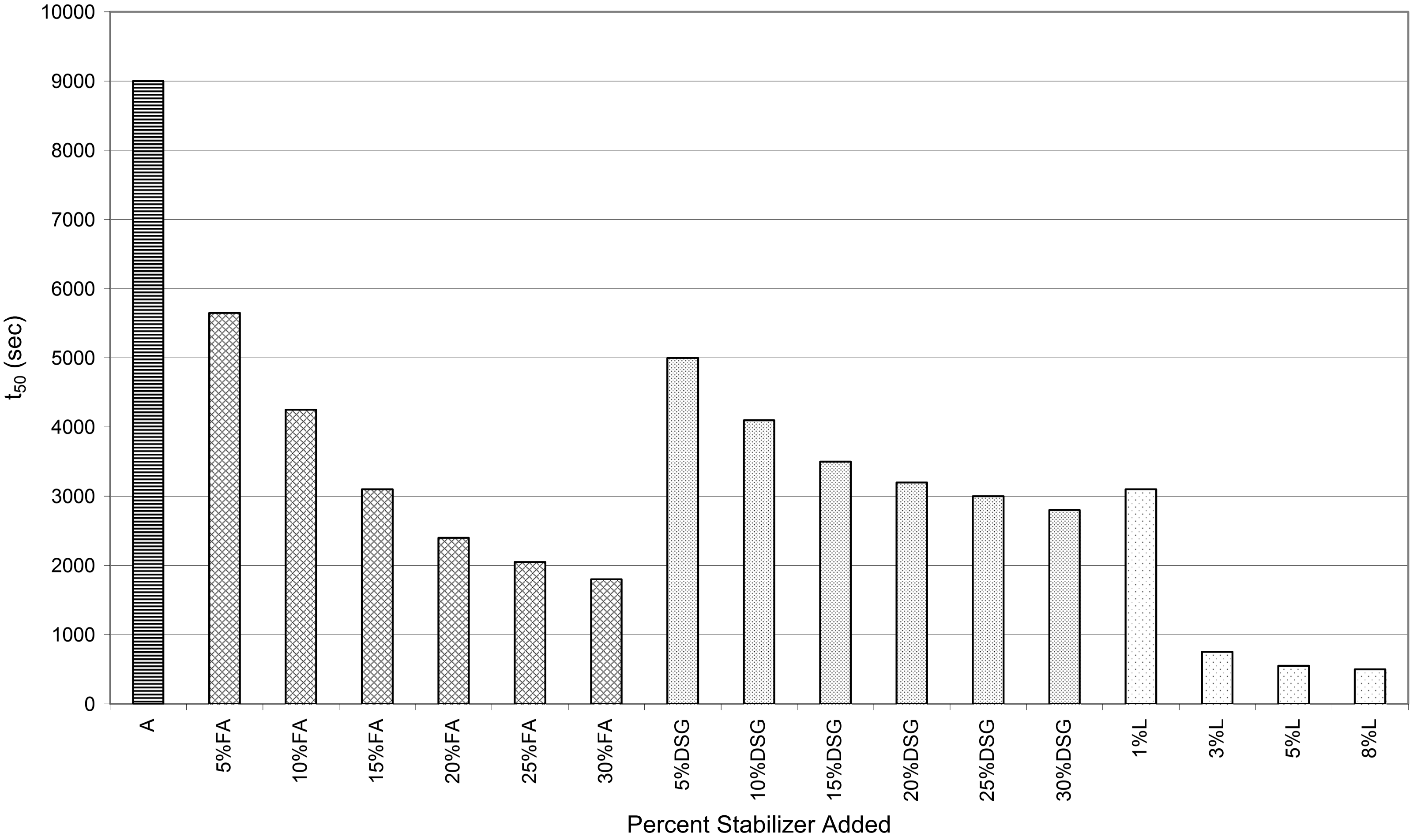

Figure 4.19. Variation of t_{50} with Type and Amount of Stabilizer Added after 28 Days Curing

CHAPTER 5

DISCUSSION OF TEST RESULTS

5.1. Effects of Fly Ash and Lime Addition on the Grain Size Distribution of Expansive Soil

Addition of both fly ash and lime shifted the grain size distribution curve of Sample A to the coarser side.

The grain size distribution curve of Sample A shifted significantly to the coarser side with the addition of fly ash (Figure 4.4). To be able to distinguish whether this shifting was due to the addition of silt sized particles to Sample A or due to some reactions between the ingredients of the samples, Figure C1 (Appendix C) was plotted for fly ash. Grain size distribution curves of Sample A, fly ash, measured 25%FA, and calculated 25%FA were shown in this figure (calculated 25%FA was obtained by considering 25% fly ash and 75% Sample A on the mass basis). As it can be seen from Figure C1 calculated 25%FA curve is slightly on the finer side of measured 25% FA curve, but these two curves are very close to each other. This shows that the shifting of the grain size distribution curve of Sample A with the addition of fly ash is mainly a consequence of the addition of silt sized particles, and measured 25%FA curve's being slightly on the coarser side of calculated 25%FA curve may be attributed to the cation exchange process and some pozzolanic reactions between the ingredients of the samples which cause the flocculation of clay particles.

Addition of 1% and 3% lime did not shift the grain size distribution curve of Sample A significantly. Addition of 5% and 8% lime caused noticeable shifting of the curve to the coarser side (Figure 4.5).

5.2. Effects of Fly Ash, Desulphogypsum, and Lime Addition on the Liquid Limit of Expansive Soil (Table 5.1)

Liquid limit values of the samples decreased with increasing stabilizer percentages.

Addition of 5% fly ash diminished the liquid limit of Sample A by 26.1%. Reduction continued with increasing fly ash percentages and the maximum fly ash addition (30%) resulted in a 42.4% reduction in the liquid limit of Sample A.

A similar trend was followed by the desulphogypsum added samples, with the liquid limit values varying in a narrower range. Addition of the minimum amount of desulphogypsum (5%) reduced the liquid limit of Sample A by 30.4% and the maximum desulphogypsum additive (30%) caused a reduction of 41.3%.

Liquid limit values varied in a wider range for the lime added samples with respect to the other stabilizer added samples. Addition of 1% lime resulted in the minimum reduction (10.9%) in the liquid limit of Sample A when compared with the other stabilizers. 3% lime caused an abrupt decrease (34.8%) in the liquid limit of Sample A. The maximum lime addition (8%) caused the maximum reduction (44.6%) in the liquid limit of Sample A among all the other stabilizer additions.

Table 5.1. Percent Changes in Liquid Limit, Plastic Limit, Plasticity Index, Shrinkage Limit, Shrinkage Index, Swelling Percentage, and Rate of Swell (No Curing)

Sample	% Change in LL	% Change in PL	% Change in PI	% Change in SL	% Change in SI	% Change in dH/H	% Change in t_{50}
A	0	0	0	0	0	0	0
5%FA	26.1	-4.8	35.2	-5.6	33.8	24.7	22.7
10%FA	27.2	-14.3	39.4	-11.1	36.5	39.5	45.5
15%FA	31.5	-14.3	45.1	-22.2	44.6	52.3	61.4
20%FA	35.9	-19.0	52.1	-27.8	51.4	57.1	70.5
25%FA	39.1	-28.6	59.2	-27.8	55.4	60.1	72.7
30%FA	42.4	-28.6	63.4	-38.9	62.2	60.6	74.5
5%DSG	30.4	-9.5	42.3	-11.1	40.5	31.2	27.3
10%DSG	31.5	-9.5	43.7	-16.7	43.2	40.1	45.5
15%DSG	35.9	-14.3	50.7	-16.7	48.6	47.8	59.1
20%DSG	37.0	-14.3	52.1	-16.7	50.0	52.0	63.6
25%DSG	38.0	-19.0	54.9	-27.8	54.1	55.0	67.3
30%DSG	41.3	-23.8	60.6	-27.8	58.1	55.1	68.2
1%L	10.9	-19.0	19.7	-11.1	16.2	37.9	61.4
3%L	34.8	-47.6	59.2	-38.9	52.7	60.0	85.5
5%L	40.2	-57.1	69.0	-50.0	62.2	67.1	88.2
8%L	44.6	-61.9	76.1	-66.7	71.6	68.1	88.6

‘+’ Decrease / ‘-’ Increase

5.3. Effects of Fly Ash, Desulphogypsum, and Lime Addition on the Plastic Limit of Expansive Soil (Table 5.1)

Addition of all the stabilizers increased the plastic limit of Sample A.

Plastic limits of the fly ash added samples generally increased with increasing fly ash percentages. Addition of 25% and 30% fly ash caused the same increment (28.6%) in the plastic limit of Sample A.

Desulphogypsum addition increased the plastic limit of Sample A by 23.8% at most.

Plastic limits of the lime added samples increased significantly with increasing lime percentages. Even 3% lime addition increased the plastic limit of Sample A more than 30% fly ash and 30% desulphogypsum additions (47.6%). The maximum lime addition increased the plastic limit of Sample A more than twice the maximum fly ash and desulphogypsum additions (61.9%).

5.4. Effects of Fly Ash, Desulphogypsum, and Lime Addition on the Plasticity Index of Expansive Soil (Table 5.1)

Plasticity indices of the samples decreased significantly with increasing stabilizer percentages.

The maximum amount of fly ash reduced the plasticity index of Sample A by 63.4%.

Addition of 5% desulphogypsum caused a sudden decrease in the plasticity index of Sample A by 42.3%. This is the maximum reduction obtained with the least amount of stabilizer. The maximum amount of desulphogypsum decreased the plasticity index of Sample A by 60.6% which is the least reduction obtained with the maximum amount of stabilizer.

Lime was found to be the most effective stabilizer to decrease the plasticity index of Sample A. 8% lime decreased the plasticity index by 76.1%.

5.5. Effects of Fly Ash, Desulphogypsum, and Lime Addition on the Shrinkage Limit of Expansive Soil (Table 5.1)

Addition of all the stabilizers increased the shrinkage limit of Sample A.

The maximum amount of fly ash increased the shrinkage limit of Sample A by 38.9%.

Addition of 10%, 15%, and 20% desulphogypsum increased the shrinkage limit of Sample A by the same amount (16.7%). The maximum desulphogypsum addition caused an increment of 27.8%.

The maximum lime addition increased the shrinkage limit of Sample A by 66.7%.

5.6. Effects of Fly Ash, Desulphogypsum, and Lime Addition on the Shrinkage Index of Expansive Soil (Table 5.1)

Shrinkage indices of the samples decreased significantly with increasing stabilizer percentages.

The maximum fly ash addition decreased the shrinkage index of Sample A by 62.2%.

Shrinkage indices varied in a narrower range for the desulphogypsum added samples with respect to the other stabilizer added samples. Addition of 5% desulphogypsum decreased the shrinkage index of Sample A by 40.5% which is the maximum reduction obtained with the least amount of stabilizer. Addition of 30% desulphogypsum caused a reduction of 58.1% which is the minimum reduction obtained with the maximum amount of stabilizer.

8% lime addition resulted in a reduction of 71.6% in the shrinkage index of Sample A.

5.7. Effects of Fly Ash, Desulphogypsum, and Lime Addition on the Specific Gravity of Expansive Soil

Fly ash and desulphogypsum additions decreased the specific gravity (G_s) of Sample A. Lime addition increased the specific gravity of Sample A (Table 4.2).

5.8. Effects of Fly Ash and Lime Addition on the Activity of Expansive Soil

Activity values decreased with increasing both fly ash and lime additives (Table 4.2). Reduction in activity indicates lower swelling potential.

5.9. Effects of Fly Ash, Desulphogypsum, and Lime Addition on the Swelling Percentage of Expansive Soil (Table 5.1)

Swelling percentages of the samples decreased considerably with increasing stabilizer percentages.

Addition of 5% fly ash diminished the swelling percentage of Sample A by 24.7%. Reduction continued with increasing fly ash percentages. The amount of reduction decreased as fly ash additions increased. Adding 25% and 30% fly ash decreased the swelling percentage of Sample A nearly by the same amount (60.1% and 60.6% respectively).

5% desulphogypsum addition caused a reduction of 31.2% in the swelling percentage of Sample A. A gradual reduction continued up to 25% desulphogypsum addition. Reduction nearly stopped thereafter. 25% desulphogypsum addition reduced the swelling percentage of Sample A by 55.0%, 30% desulphogypsum addition resulted in a reduction of 55.1%.

Adding 8% lime caused the highest reduction in the swelling percentage of Sample A (68.1%).

5.10. Effects of Fly Ash, Desulphogypsum, and Lime Addition on the Rate of Swell of Expansive Soil (Table 5.1)

Addition of all the stabilizers definitely affected the time to reach fifty percent swell and accordingly the rate of swell.

Fly ash was effective in reducing the time to reach fifty percent swell. The maximum fly ash addition decreased the t_{50} of Sample A by 74.5%.

The reduction in t_{50} was more gradual in the case of desulphogypsum addition. It was not as effective as fly ash, but significant changes took place. The maximum amount of desulphogypsum caused a reduction of 68.2% in the t_{50} of Sample A.

Lime addition greatly reduced the time to reach fifty percent swell. Even adding 1% lime decreased the t_{50} of Sample A by 61.4%, and adding 8% lime resulted in a reduction of 88.6%. It took only 20.8 minutes for the 8% lime added sample to reach the half swell, although it happened in 3.1 hours for Sample A (Figure 4.15).

Addition of all the stabilizers promotes flocculation of clay particles. This results in larger capillary canals in the samples, therefore permeability and rate of swell increase, i.e. t_{50} decreases.

5.11. Effect of Curing on Swelling Percentage

Both 7 days and 28 days curing provided considerable reductions in the swelling percentages of the samples. Reductions were gradual for each of the stabilizer additions (Table 5.2). dH/H value of each sample was the highest for 0 days curing and lowest for 28 days curing (Figure 5.1). That is each sample including Sample A followed the order below:

$$(dH/H)_{\text{without curing}} > (dH/H)_{\text{7 days curing}} > (dH/H)_{\text{28 days curing}}$$

Curing helps the moisture to be distributed uniformly in the mixture, and by providing the time necessary for the development of pozzolanic reactions it encourages self-hardening with time. The resulting cementitious soil matrix provides extra resistance against swelling and cause further reduction in swelling percentage.

5.12. Effect of Curing on Rate of Swell

Rate of swell of each sample increased, i.e. t_{50} decreased, with curing (Table5.2, Figure 5.2). The following order was kept by all the samples:

$$(t_{50})_{\text{without curing}} > (t_{50})_{\text{7 days curing}} > (t_{50})_{\text{28 days curing}}$$

Development of pozzolanic reactions with time contributes to flocculation by bonding adjacent soil grains together and results in further increase in permeability and rate of swell.

Table 5.2. Percent Changes in Swelling Percentage and Rate of Swell with No Curing, 7 Days Curing, and 28 Days Curing

	% Change in dH/H			% Change in t_{50}		
Sample	Without Curing	7 Days Curing	28 Days Curing	Without Curing	7 Days Curing	28 Days Curing
A	0	0	0	0	0	0
5%FA	24.7	28.3	31.9	22.7	30.0	37.2
10%FA	39.5	43.7	47.3	45.5	50.0	52.8
15%FA	52.3	54.6	56.6	61.4	64.2	65.6
20%FA	57.1	58.4	59.9	70.5	72.5	73.3
25%FA	60.1	60.3	62.1	72.7	75.1	77.2
30%FA	60.6	60.9	62.4	74.5	78.1	80.0
5%DSG	31.2	33.8	36.0	27.3	35.0	44.4
10%DSG	40.1	42.4	45.5	45.5	51.5	54.4
15%DSG	47.8	48.1	50.7	59.1	60.2	61.1
20%DSG	52.0	50.9	53.9	63.6	65.0	64.4
25%DSG	55.0	52.7	54.9	67.3	67.9	66.7
30%DSG	55.1	52.7	54.9	68.2	70.0	68.9
1%L	37.9	40.9	43.7	61.4	64.0	65.6
3%L	60.0	63.5	65.8	85.5	91.3	91.7
5%L	67.1	69.3	70.8	88.2	91.5	93.9
8%L	68.1	70.3	71.6	88.6	93.5	94.4

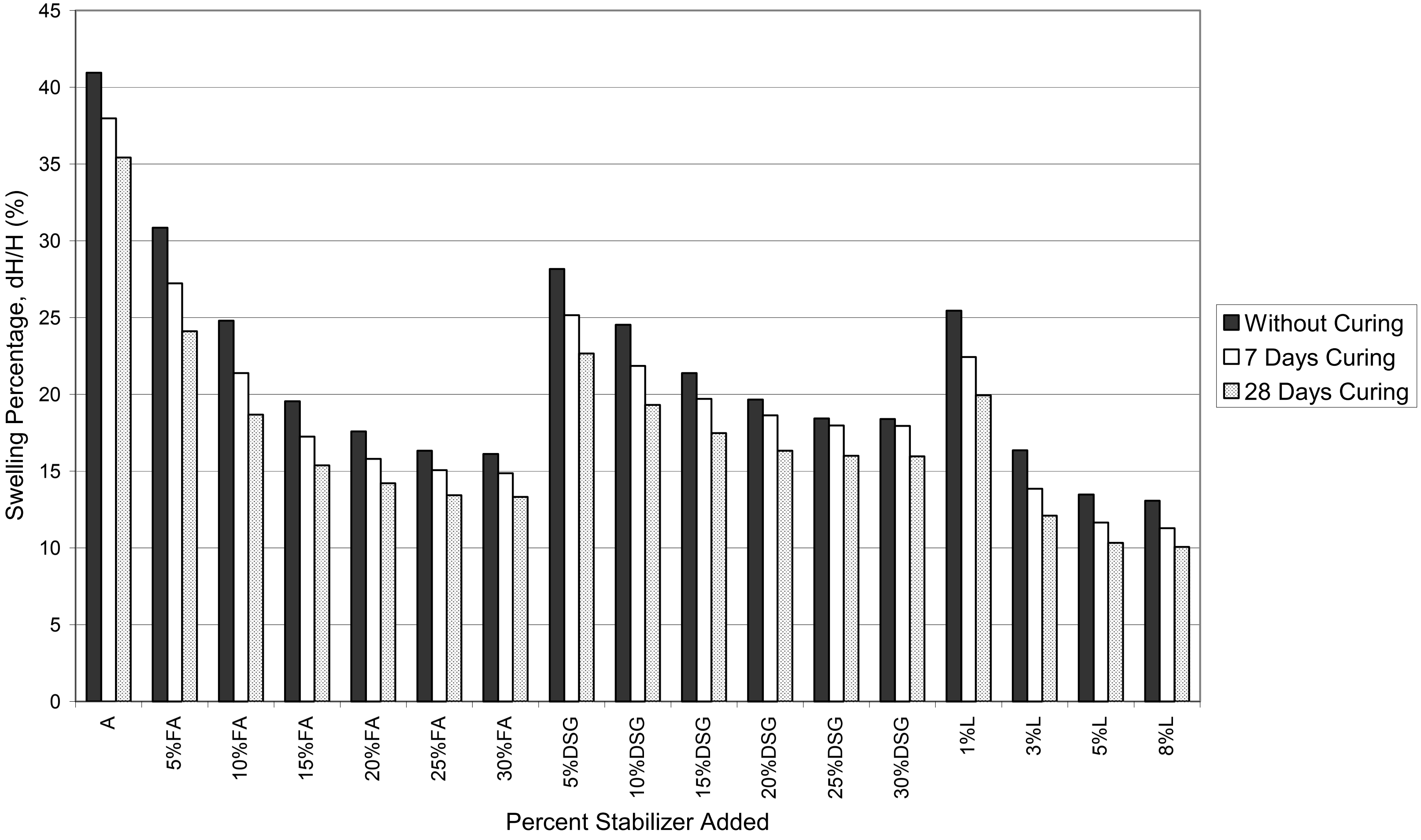

Figure 5.1. Effect of Curing on Swelling Percentage

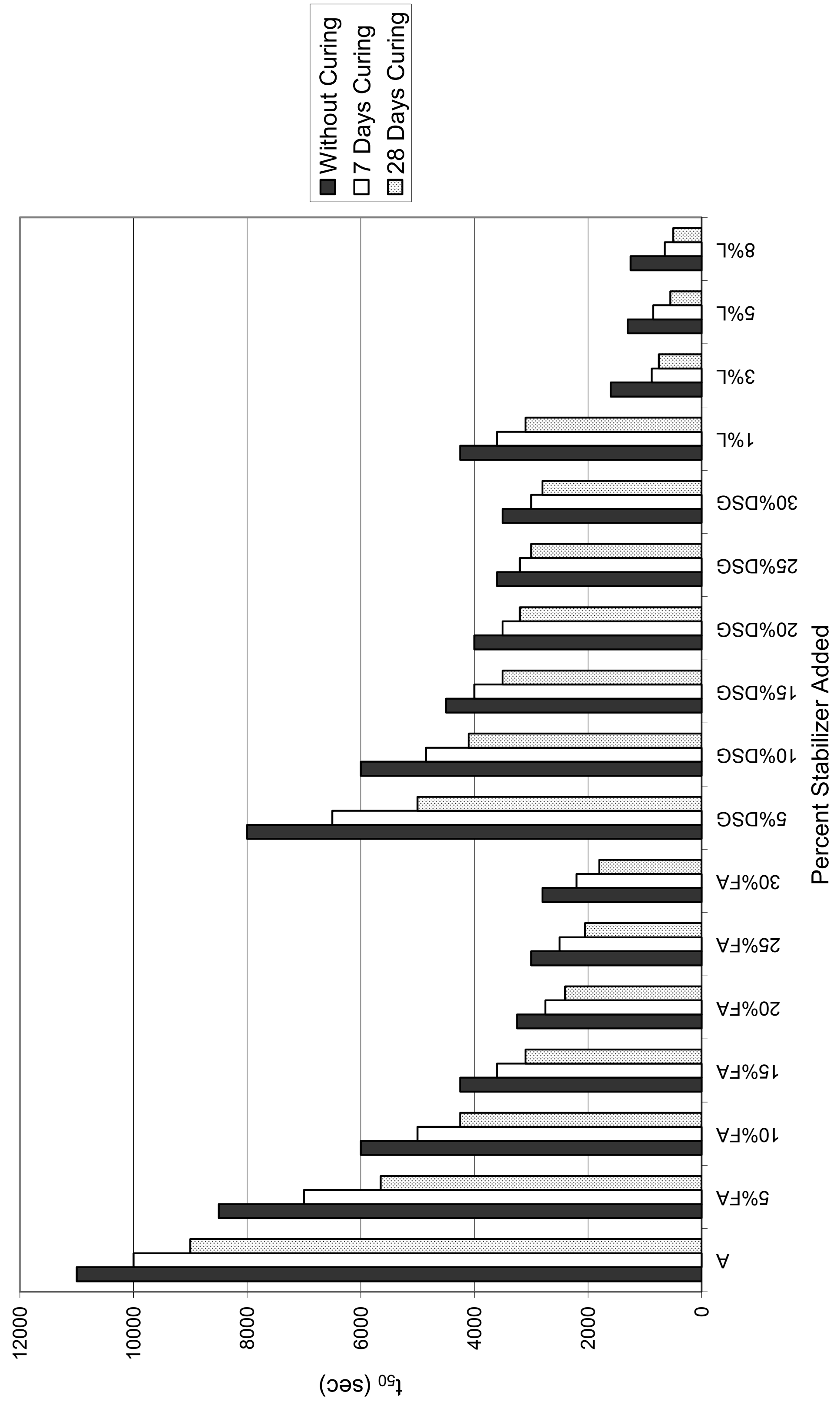

Figure 5.2. Effect of Curing on Rate of Swell

CHAPTER 6

CONCLUSIONS

The effect of fly ash, desulphogypsum, and lime addition on the volume change of an expansive soil sample was presented. Fly ash and desulphogypsum were introduced as admixtures up to a maximum of 30%, lime was introduced up to 8% by dry weight of soil. Due to the results of the experiments, the following conclusions are warranted:

1. Addition of fly ash and lime alters the grain size distribution of the expansive soil sample. The silt fractions increase whereas the clay fractions decrease with increasing amounts of fly ash and lime.

2. The alteration of the grain size distribution of the expansive soil sample with the addition of fly ash is mainly a result of the addition of silt sized particles. The cation exchange process and some pozzolanic reactions, which cause the flocculation of clay particles, also effect the grain size distributions of fly ash treated soil samples.

3. Addition of fly ash, desulphogypsum, and lime increases the plastic limit and shrinkage limit, and decreases the liquid limit, plasticity index, and shrinkage index of the expansive soil sample.

4. Fly ash, desulphogypsum, and lime addition shifts the expansive soil sample from CH towards MH according to the Unified Soil Classification System.

Addition of 3%, 5%, and 8% lime changes the classification of the expansive soil sample from CH to MH according to the Unified Soil Classification System.

5. Fly ash and lime addition decreases the activity of the expansive soil sample.

6. Fly ash, desulphogypsum, and lime addition decreases the swelling percentage of the expansive soil sample. The amount of reduction decreases with increasing stabilizer percentages.

7. Fly ash, desulphogypsum, and lime addition decreases the time to reach fifty percent swell (t_{50}). The amount of reduction decreases with increasing stabilizer percentages.

8. Curing results in further reduction in the swelling percentages, and t_{50} values.

9. Curing is more effective in reducing the t_{50} values than in reducing the swelling percentages.

10. Curing for the first increments of fly ash, desulphogypsum, and lime addition is generally more effective in reducing the swelling percentages and t_{50} values, with subsequent additions of the stabilizers being less beneficial.

11. With the 25% and 30% fly ash additions the swelling percentages become almost equal to the swelling percentage of 8% lime added sample.

12. There is only a slight change in the swelling percentage and rate of swell from 25 to 30 percent fly ash and desulphogypsum addition, therefore the optimum fly ash and desulphogypsum addition appears to be close to 25%.

Recommendation for Future Research

In this study, the geotechnical performances of fly ash and desulphogypsum in stabilizing the expansive soil were presented. However, past research has established that both fly ash and desulphogypsum consist of fine particles that contain leachable heavy metals such as arsenic, cobalt, copper, lead, nickel, and zinc, and are therefore classified as toxic wastes (Clark et al. 2001; Crews et al. 1998; Ferreira et al. 2003; Tao et al.2001). The risks imposed on the environment by possible geotechnical applications of fly ash and desulphogypsum must be carefully weighed against creating new pollution sources elsewhere. Therefore, to define more clearly the conditions for a safe application from an environmental point of view this research must be extended by performing leachate analyses of the samples used in this study.

REFERENCES

1. Abduljauwad, S.N., (1993), “Treatment of Calcareous Expansive Clays”, ASCE Geotechnical Publication, No. 36, pp. 100-115.

2. Alawaji, H.A., (1999), “Swelling and Compressibility Characteristics of Compacted Sand-Bentonite Mixtures”, Geotechnical Engineering Journal, Vol. 30, No.1, pp. 1-12.

3. Al-Rawas, A.A., Taha R., Nelson, J.D., Al-Shab, T.B. and Al-Siyabi, H., (2002), “A Comparative Evaluation of Various Additives Used in the Stabilization of Expansive Soils”, Geotechnical Testing Journal, Vol. 25, No. 2, pp. 199-209.

4. Arnold, M., (1984), “The Genesis, Mineralogy and Identification of Expansive Soils”, Fifth International Conference on Expansive Soils, Adelaide, South Australia, pp. 32-36.

5. ASTM, (1993), “Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete”, Annual Book of ASTM Standards, C 618-93, Vol. 04-02, pp. 310-312.

6. ASTM, (1993), “Standard Test Method for One-Dimensional Consolidation Properties of Soils”, Annual Book of ASTM Standards, D 2435-90, Vol. 04-08, pp. 313-323.

7. ASTM, (1993), “Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils”, Annual Book of ASTM Standards, D 4546-90, Vol. 04-08, pp. 853-859.

8. ASTM, (1993), “Standard Practice for Characterizing Fly Ash for Use in Soil Stabilization”, Annual Book of ASTM Standards, D 5239-92, Vol. 04-08, pp. 1277-1279.

9. Bell, F.G., (1993), “An Examination of the Use of Lime and Pulverized Fly Ash to Stabilize Clay Materials”, Bulletin of the Association of Engineering Geologists, No. 4, pp. 469-479.

10. Bell, F.G., (1996), "Lime Stabilization of Clay Minerals and Soils", Engineering Geology, Vol. 42, pp. 223-237.

11. Chen, F.H., (1975), Foundations on Expansive Soils, Elsevier Scientific Publishing Company, 280 pages.

12. Chen, W.F., (1995), The Civil Engineering Hand Book, CRC Press Inc., 2609 pages.

13. Clark, R.B., Ritchey, K.D. and Baligar, V.C., (2001), "Benefits and Constraints for Use of FGD Products on Agricultural Land", Fuel, Vol. 80, pp. 821-828.

14. Craig, R.F., (1993), Soil Mechanics, Chapman and Hall, 427 pages.

15. Çokça, E., (2001), "Use of Class C Fly Ashes for the Stabilization of an Expansive Soil", Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 7, pp. 568-573.

16. Das, B.M., (1990), Principles of Foundation Engineering, Mc Graw-Hill, 810 pages.

17. El-Sohby, M.A. and Rabba, E.A., (1981), "Some Factors Affecting Swelling of Clayey Soils", Geotechnical Engineering, Vol. 12, pp. 19-39.

18. El-Sohby, M.A. and Mazen, O., (1983), "Mineralogy and Swelling of Expansive Clayey Soils", Geotechnical Engineering, Vol. 14, pp. 79-87.

19. Ferreira, C., Riberio, A. and Ottosen, L., (2003), "Possible Applications for Municipal Solid Waste Fly Ash", Journal of Hazardous Materials, Vol. B96, pp. 201-216.

20. Fredlund, D.G. and Rahardjo, H., (1993), Soil Mechanics for Unsaturated Soils, John Wiley and Sons Inc., 517 pages.

21. Galos, K.A., Smakowski, T.S. and Szlugaj, J., (2003), "Flue-Gas Desulphurization Products from Polish Coal-Fired Power-Plants", Applied Energy, Vol. 75, pp.257-265.

22. Gens, A. and Alonso, E.E., (1992), "A Framework for the Behavior of Unsaturated Expansive Clays", Canadian Geotechnical Journal, Vol. 29, pp. 1013-1032.

23. Gomes, S., François, M., Pellisier, C. and Evrard, O., (1998), "Characterization and Comparative Study of Coal Combustion Residues

from a Primary and Additional Flue Gas Secondary Desulfurization Process", Cement and Concrete Research, Vol. 28, No. 11, pp. 1605-1619.

24. Grim, R.E., (1962), Applied Clay Mineralogy, Mc Graw-Hill Series in the Geotechnical Sciences, 422 pages.

25. Gromko, G.J., (1974), "Review of Expansive Soils", ASCE Journal of the Geotechnical Engineering, Vol. 100, No. GT6, pp.667-687.

26. İpek, T., (1998), Stabilization of Expansive Soils Using Lime, Cement and Fly Ash, M.S. Thesis, METU, Turkey, 119 pages.

27. Kehew, A.E., (1995), Geology for Engineers and Environmental Scientists, Prentice Hall, 574 pages.

28. Kinithua, J.M., Wild, S. and Jones, G.I., (1999), "Effects of Monovalent and Divalent Metal Sulphates on Consistency and Compaction of Lime-Stabilized Kaolinite", Applied Clay Science, Vol. 14, pp. 27-45.

29. Komine, H. and Ogata, N., (1996), "Prediction for Swelling Characteristics of Compacted Bentonite", Canadian Geotechnical Journal, Vol. 33, pp. 11-22.

30. Mathew, P.K. and Rao, S.N., (1997), "Effect of Lime on Cation Exchange Capacity of Marine Clay", Journal of Geotechnical and Geoenvironmental Engineering", Vol. 123, No. 2, pp. 183-185.

31. Mitchell, J.K., (1993), Fundamentals of Soil Behavior, John Wiley and Sons Inc., 422 pages.

32. Mohamed, A.M.O., (2000), "The Role of Clay Minerals in Marly Soils on its Stability", Engineering Geology, Vol. 57, pp. 193-203.

33. Nalbantoğlu, Z. and Tuncer, E.R., (2001), "Compressibility and Hydraulic Conductivity of a Chemically Treated Expansive Clay", Canadian Geotechnical Journal, Vol. 38, pp. 154-160.

34. Nalbantoğlu, Z. and Güçbilmez, E., (2002), "Utilization of an Industrial Waste in Calcareous Expansive Clay Stabilization", Geotechnical Testing Journal, Vol. 25, No. 1, pp. 78-84.

35. Nelson, J.D. and Miller, D.J., (1992), Expansive Soils, Problems and Practice in Foundation and Pavement Engineering, John Wiley and Sons Inc., 259 pages.

36. Oman, J., Dejanovic, B. and Tuma, M., (2002), "Solutions to the Problem of Waste Deposition at a Coal-Fired Power Plant", Waste Management, Vol. 22, pp. 617-623.

37. Özkul, M.H., (2000), "Utilization of Citro- and Desulphogypsum as Set Retarders in Portland Cement", Cement and Concrete Research, Vol. 30, pp. 1755-1758.

38. Payette, R.M., Wolfe, W.E. and Beeghly, J., (1997), "Use of Clean Coal Combustion By-Products in Highway Repairs", Fuel, Vol. 76, No. 8, pp. 749-753.

39. Popescu, M.E., (1986), "A Comparison between the Behaviour of Swelling and of Collapsing Soils", Engineering Geology, Vol. 23, pp. 145-163.

40. Rao, S.N. and Rajasekaran, G., (1996), "Reaction Products Formed in Lime-Stabilized Marine Clays", Journal of Geotechnical Engineering, Vol. 122, No. 5, pp. 329-336.

41. Rao, K.S.S. and Triphaty, S., (2003), "Effect of Aging on Swelling and Swell-Shrink Behavior of a Compacted Expansive Soil", Geotechnical Testing Journal, Vol. 26, No. 1, pp. 1-11.

42. Sahu, S., Brown, S.A. and Lee, R.J., (2002), "Thaumisite Formation in Stabilized Coal Combustion By-Products", Cement and Concrete Composites, Vol. 24, pp.385-391.

43. Shuai, F. and Fredlund, D.G., (1998), "Model for the Simulation of Swelling-Pressure Measurements on Expansive Soils", Canadian Geotechnical Journal, Vol. 35, pp. 96-114.

44. Sivapullaiah, P.V., Sridharan,A. and Stalin, V.K., (1996), "Swelling Behavior of Soil-Bentonite Mixtures", Canadian Geotechnical Journal, Vol. 33, pp. 808-814.

45. Sivapullaiah, P.V., Prashanth, J.P., Sridharan, A. and Narayana, B.V., (1998), "Reactive Silica and Strength of Fly Ashes", Geotechnical and Geological Engineering, Vol. 16, pp. 239-250.

46. Snethen, D.R., (1984), "Evaluation of Expedient Methods for Identification and Classification of Potentially Expansive Soils", Fifth International Conference on Expansive Soils, Adelaide, South Australia, pp. 22-26.

47. Tao, D., Chen, S., Parekh, B.K. and Hepworth, M.T., (2001), "An Investigation of a Thermochemical Process for Conversion of Gypsum and

Pyrite Solid Wastes into Useful Products", Advances in Environmental Research, Vol. 5, pp. 277-284.

48. TFHRC (Transportation and Federal Highways Research Council), (2003), "Coal Fly Ash-User Guideline-Embankment or Fill", http://www.tfhrc.gov/hnr20/recycle/waste/cfa54.htm.

49. Vassilev, S.V., Menendez, R., Alvarez, D., Somoano, M.D. and Tarazona, M.R.M., (2003), "Phase-Mineral and Chemical Composition of Coal Fly Ashes as a Basis for their Multicomponent Utilization. 1. Characterization of Feed Coals and Fly Ashes", Fuel, Vol. 82, pp. 1793-1811.

50. Wayne, A.C., Mohamed, A.O. and El-Fatih, M.A., (1984), "Construction on Expansive Soils in Sudan", Journal of Construction Engineering and Management, Vol. 110, No. 3, pp. 359-374.

APPENDIX A

Swelling percentage vs. time relationships of the fly ash, desulphogypsum, and lime added samples are given in Appendix A.

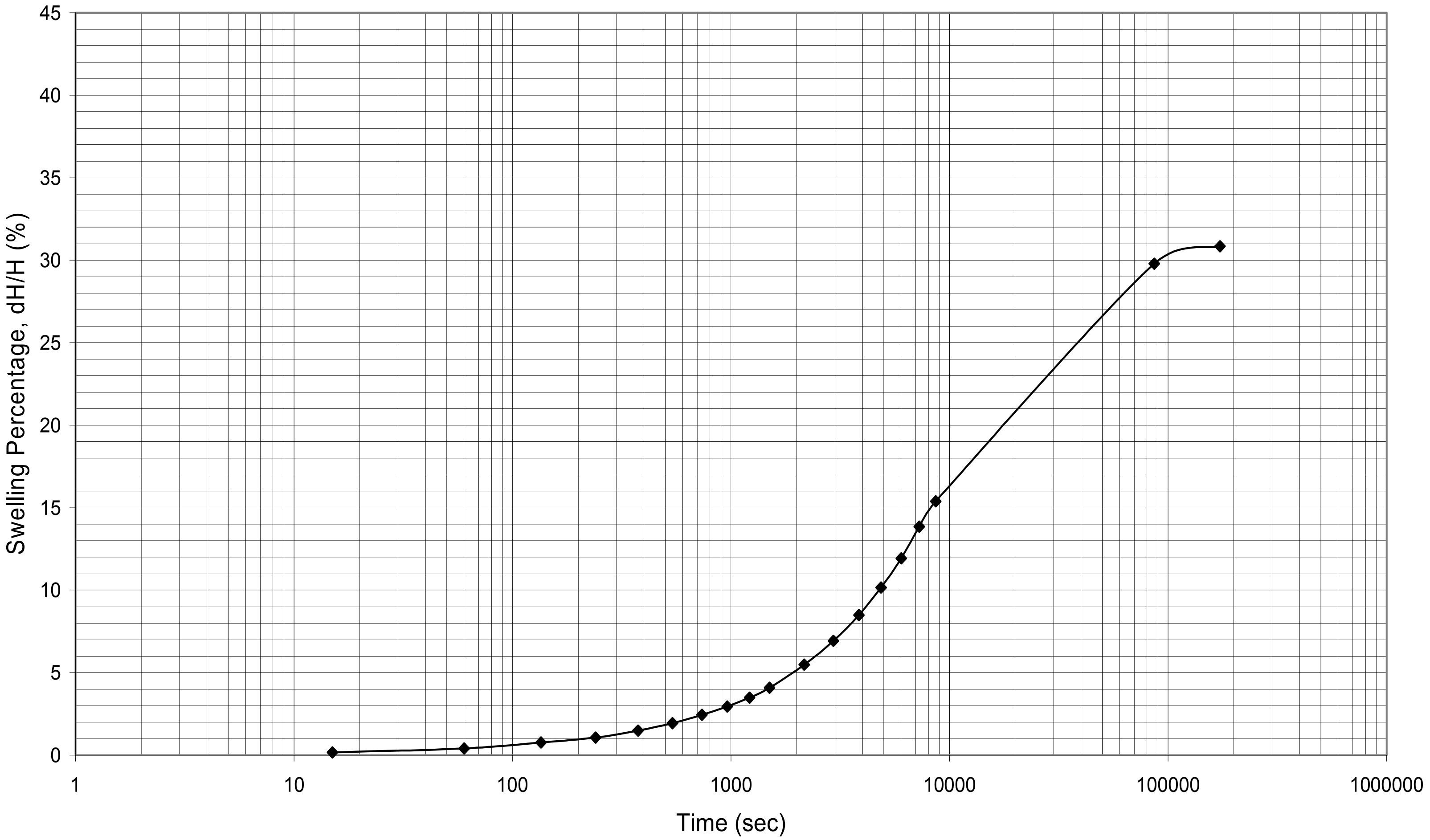

Figure A.1. Swelling Percentage vs. Time Relationship for Sample 5%FA

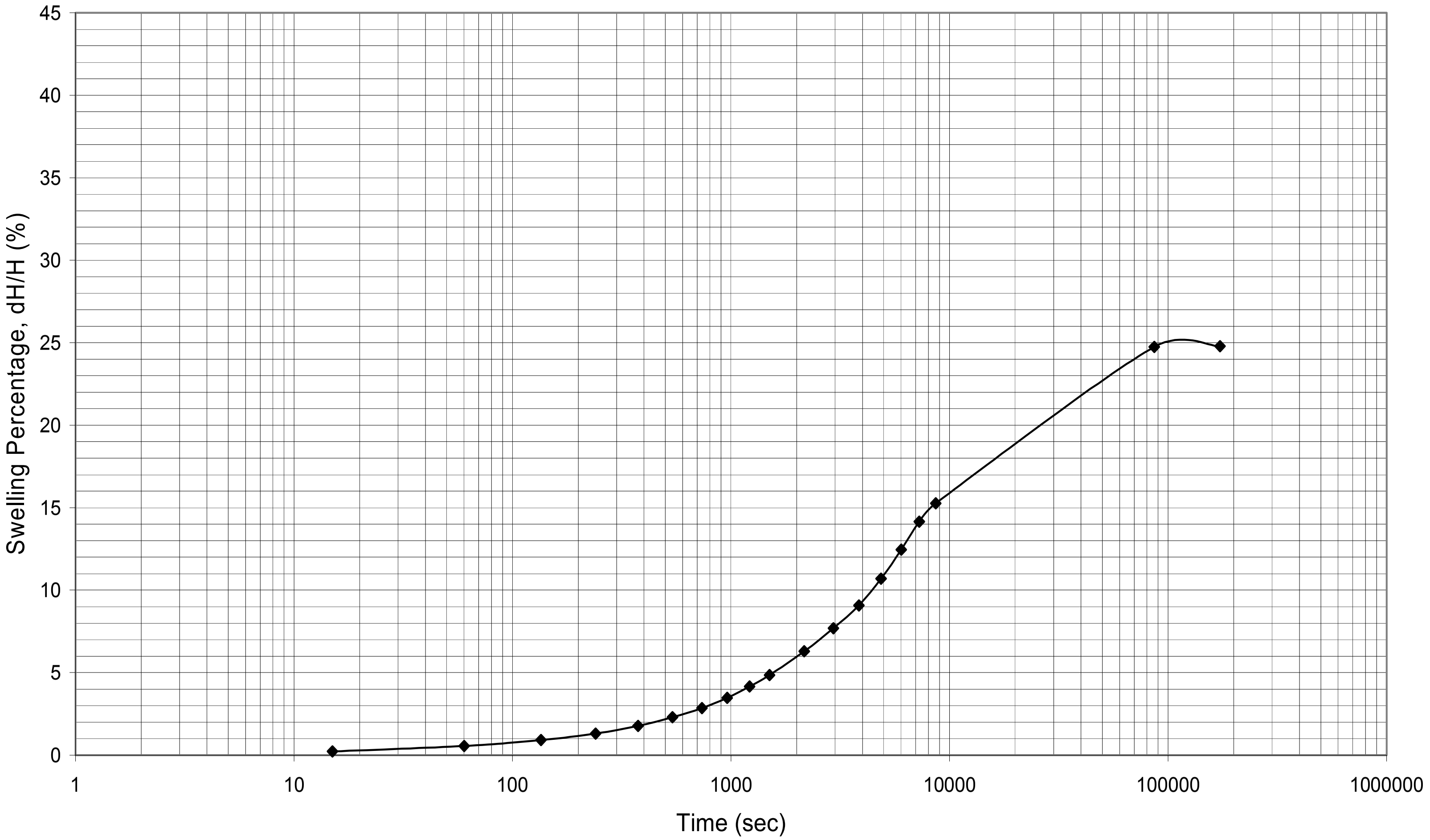

Figure A.2. Swelling Percentage vs. Time Relationship for Sample 10%FA

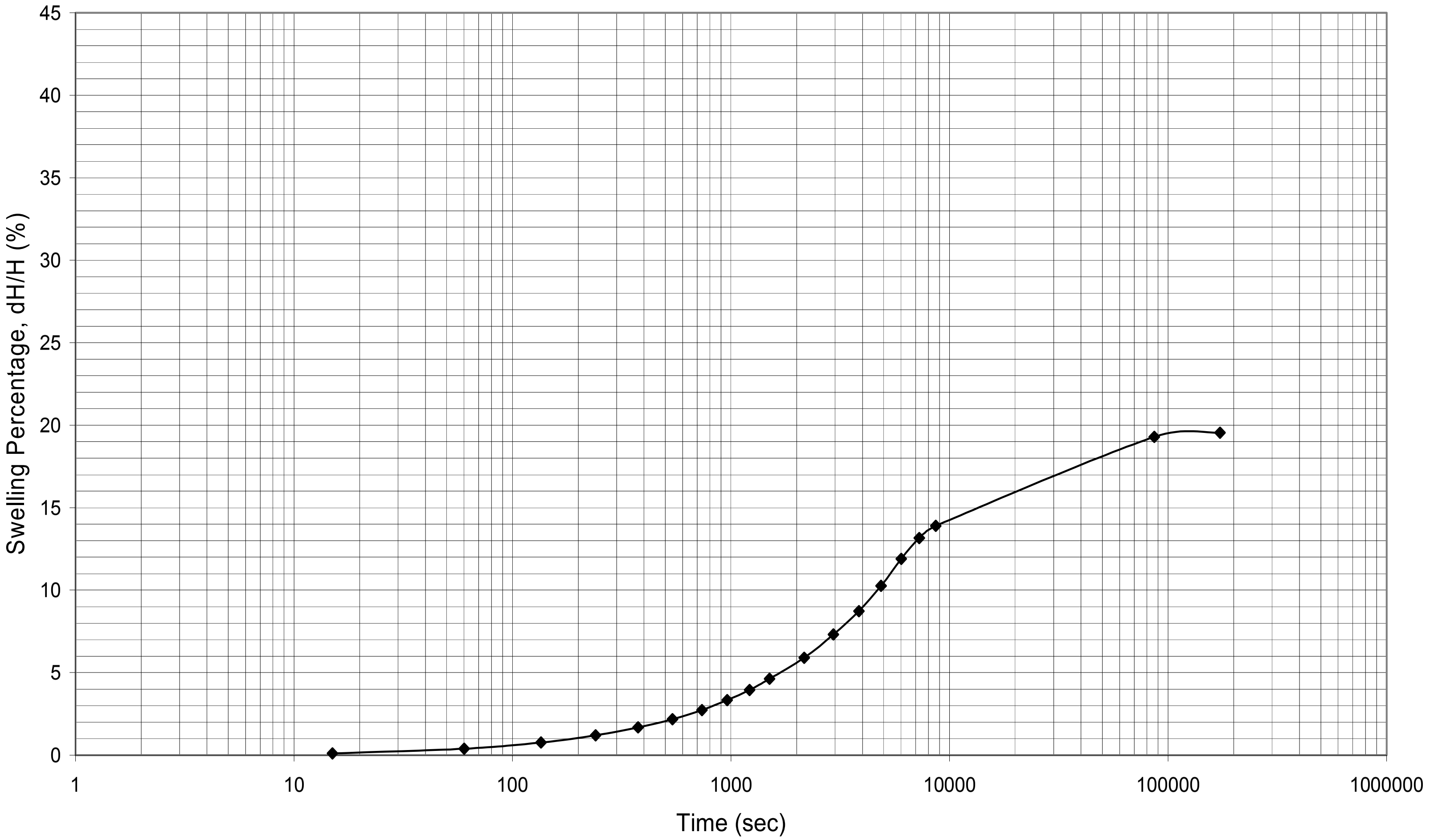

Figure A.3. Swelling Percentage vs. Time Relationship for Sample 15%FA

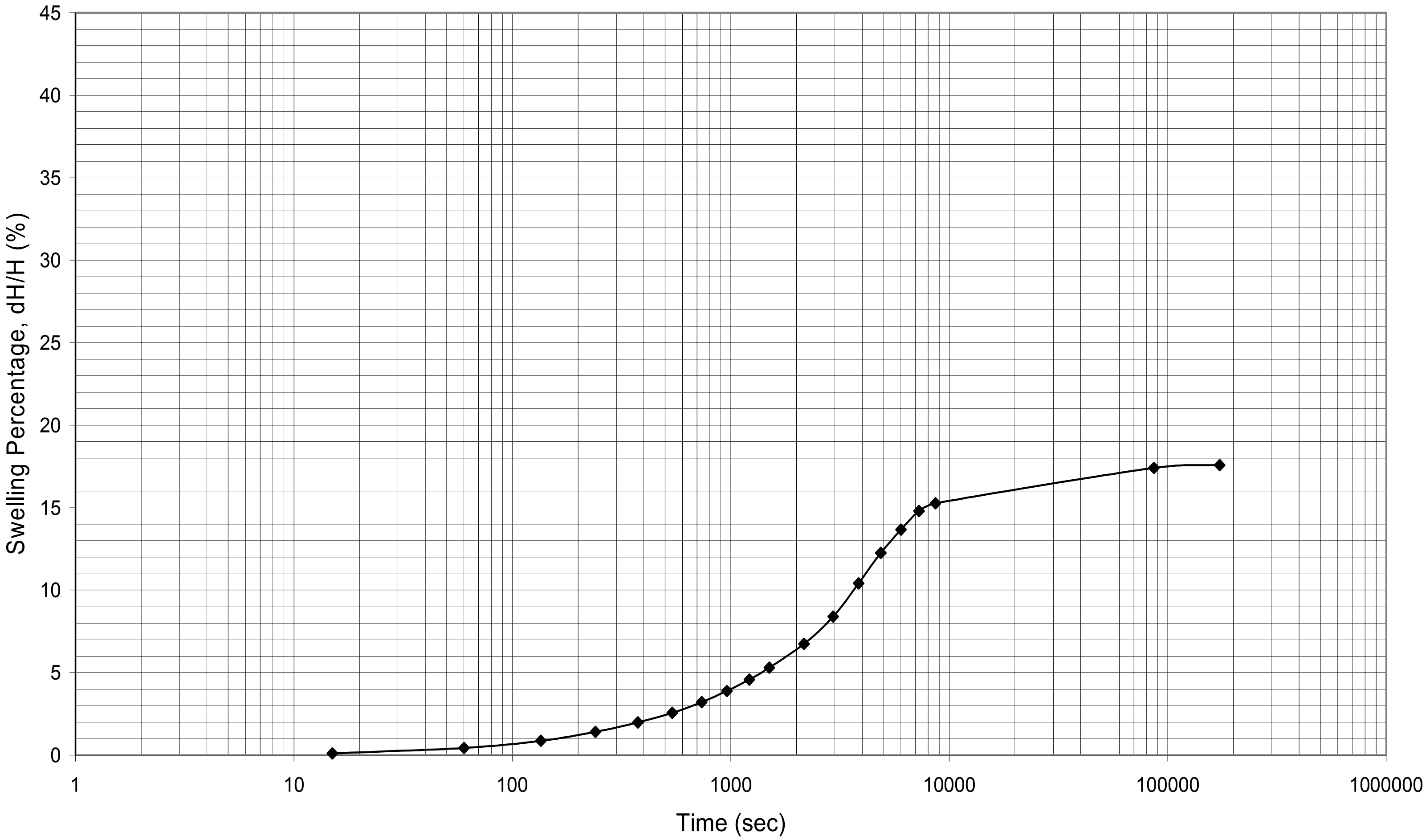

Figure A.4. Swelling Percentage vs. Time Relationship for Sample 20%FA

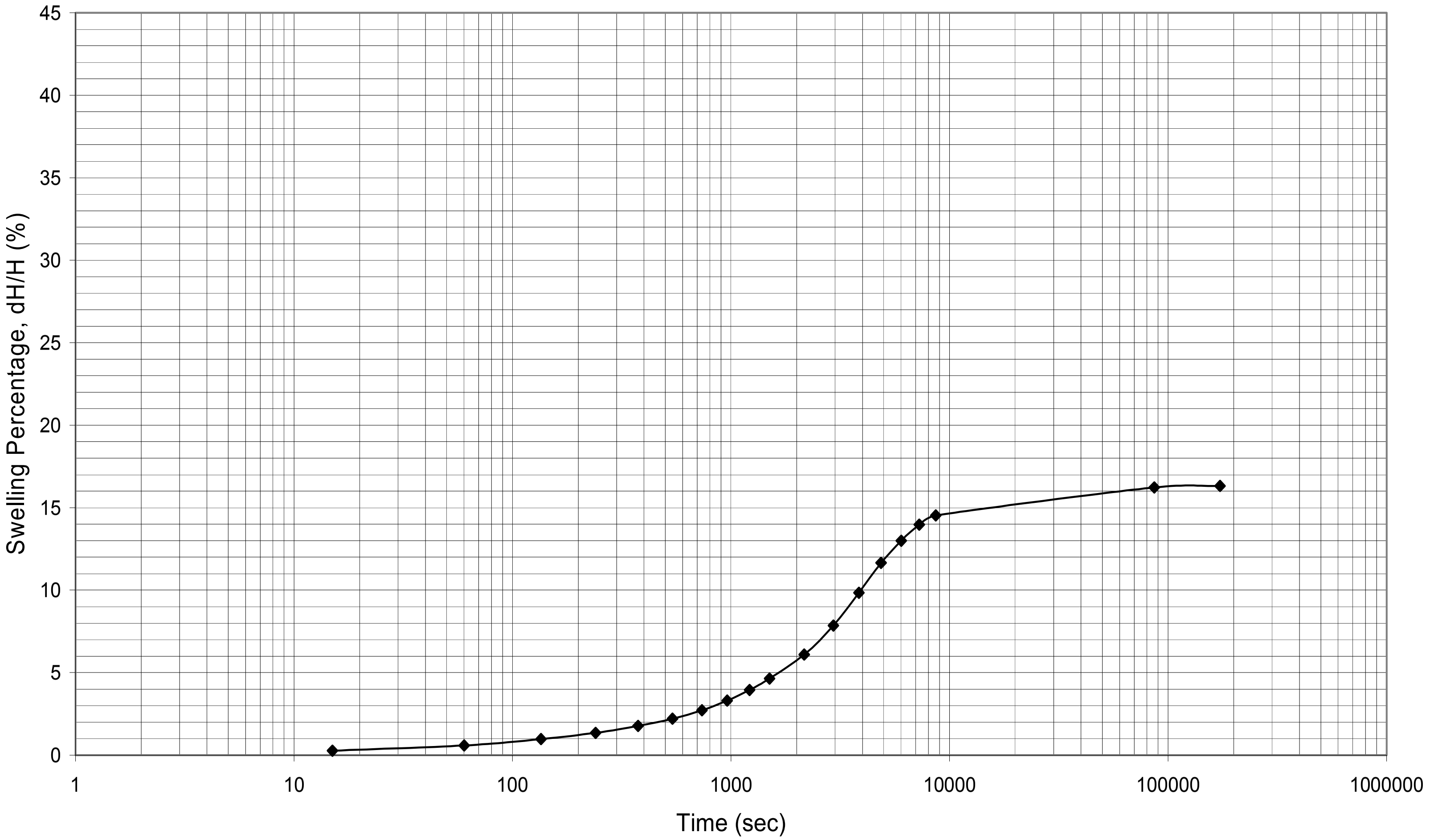

Figure A.5. Swelling Percentage vs. Time Relationship for Sample 25%FA

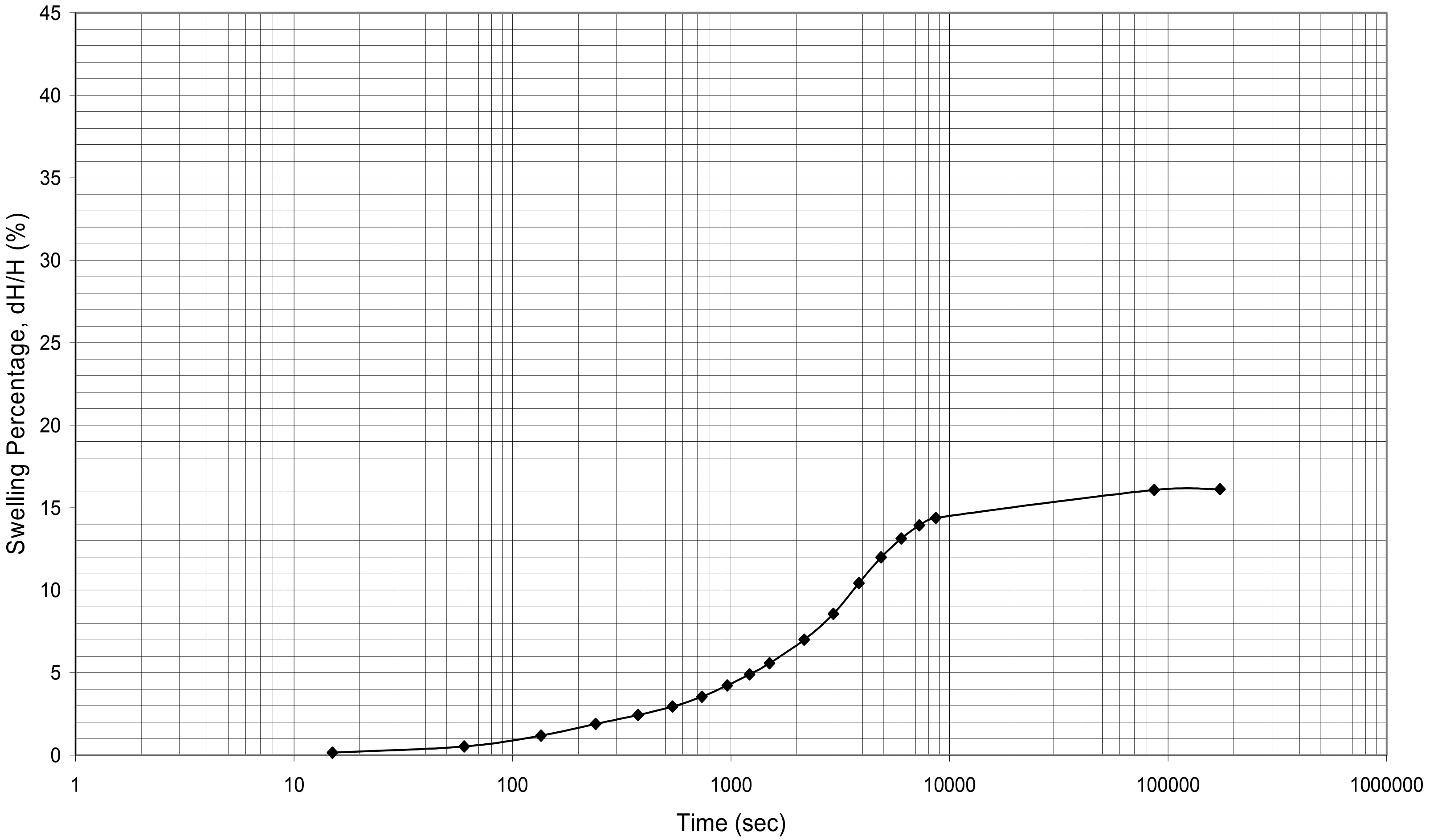

Figure A.6. Swelling Percentage vs. Time Relationship for Sample 30%FA

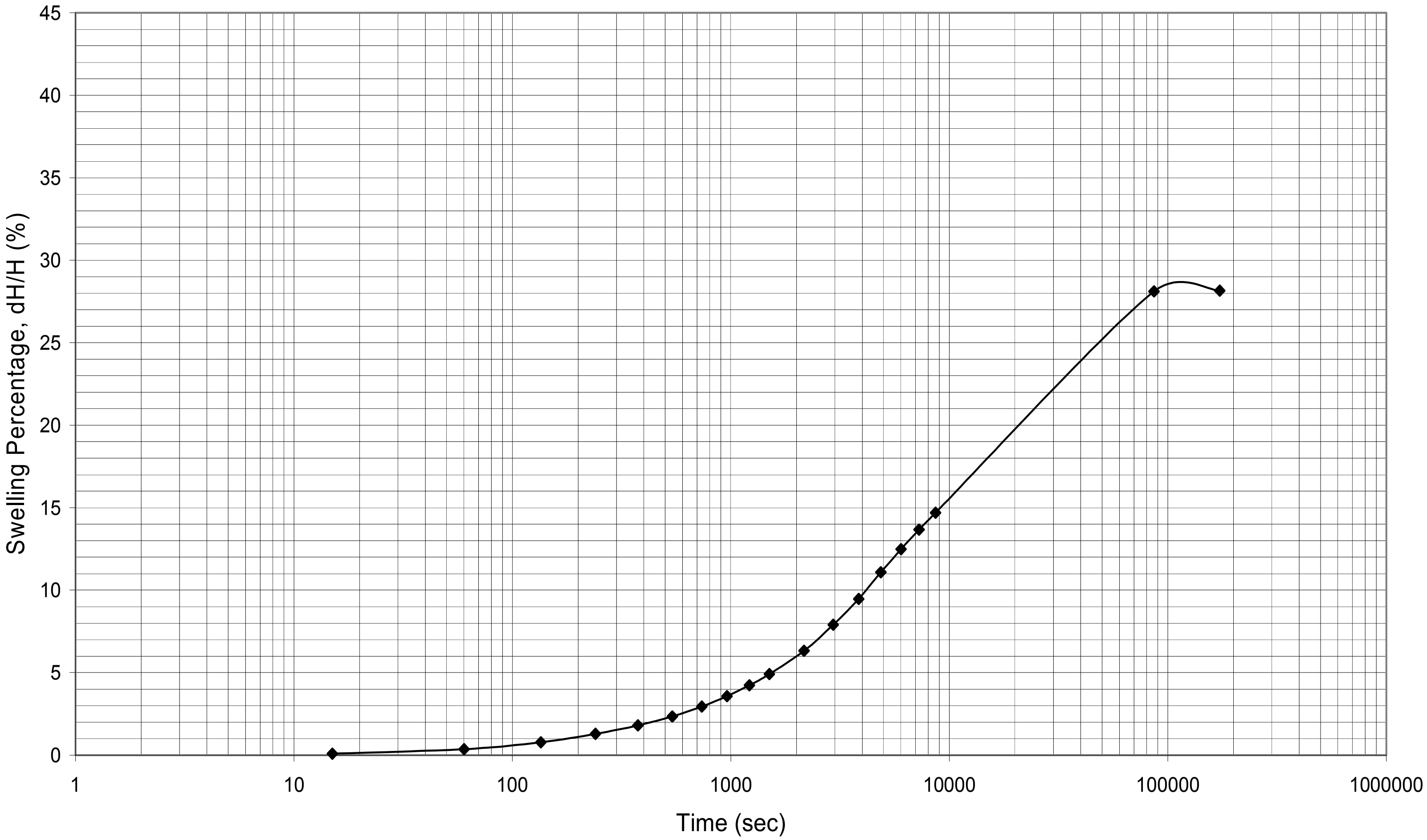

Figure A.7. Swelling Percentage vs. Time Relationship for Sample 5%DSG

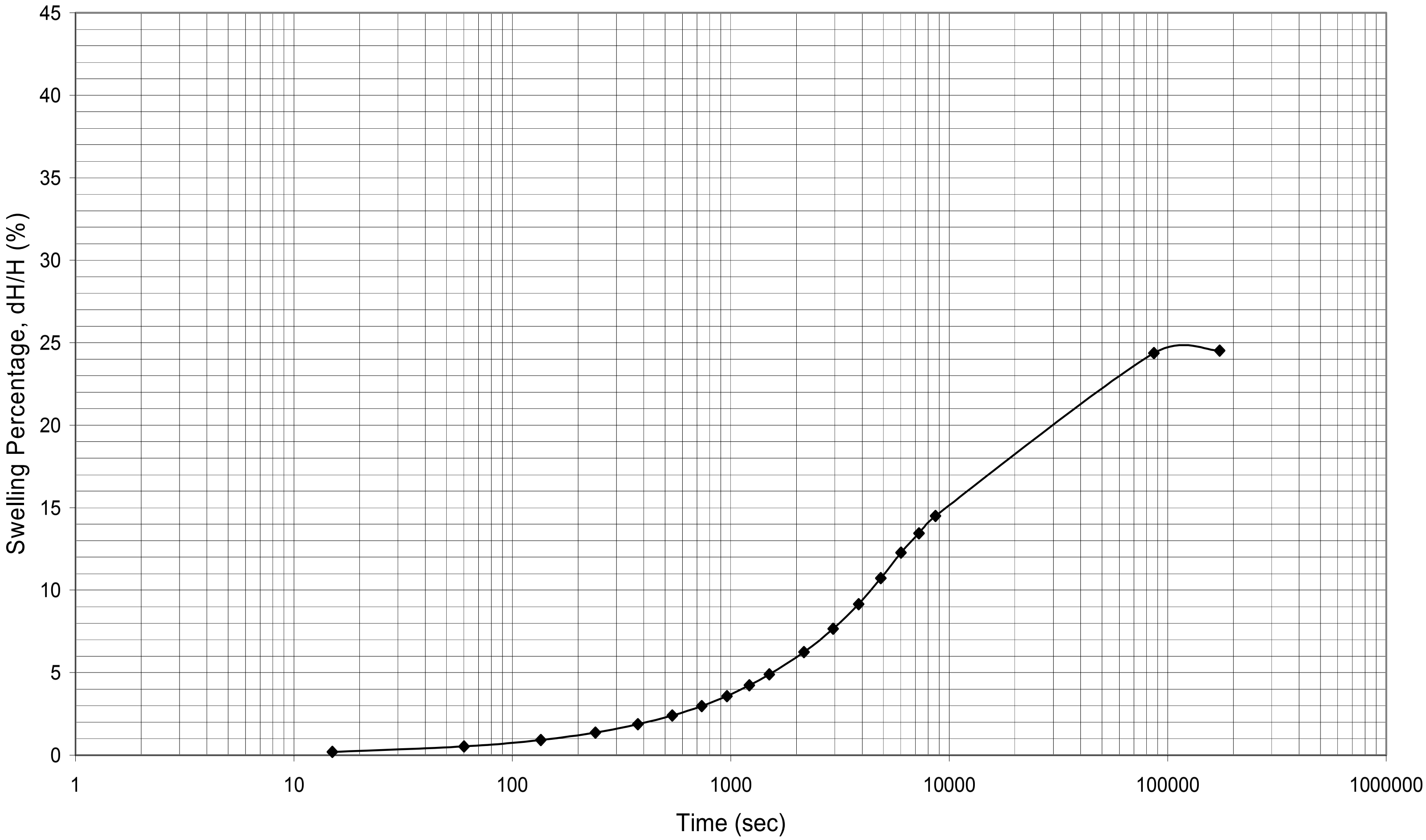

Figure A.8. Swelling Percentage vs. Time Relationship for Sample 10%DSG

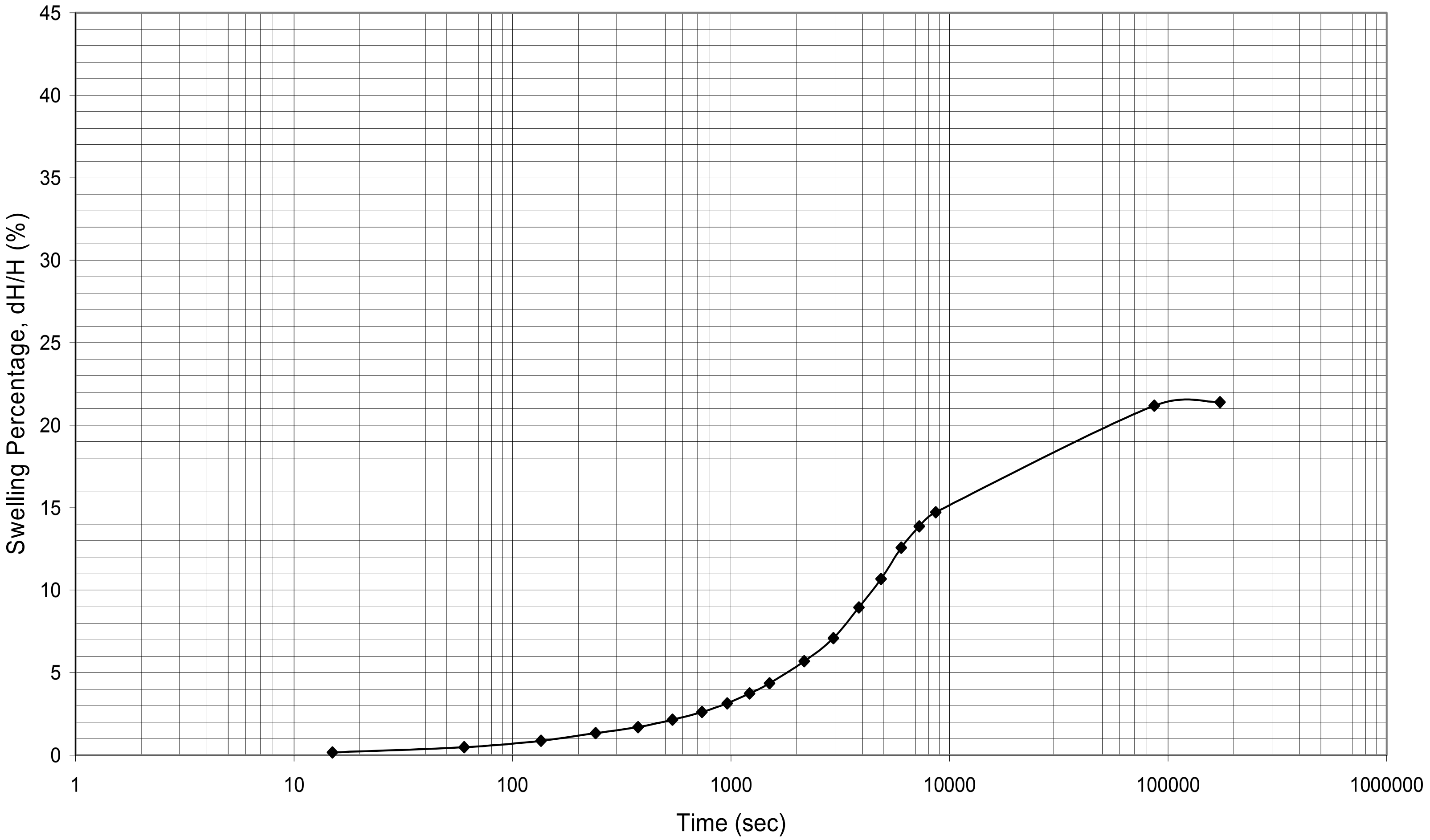

Figure A.9. Swelling Percentage vs. Time Relationship for Sample 15%DSG

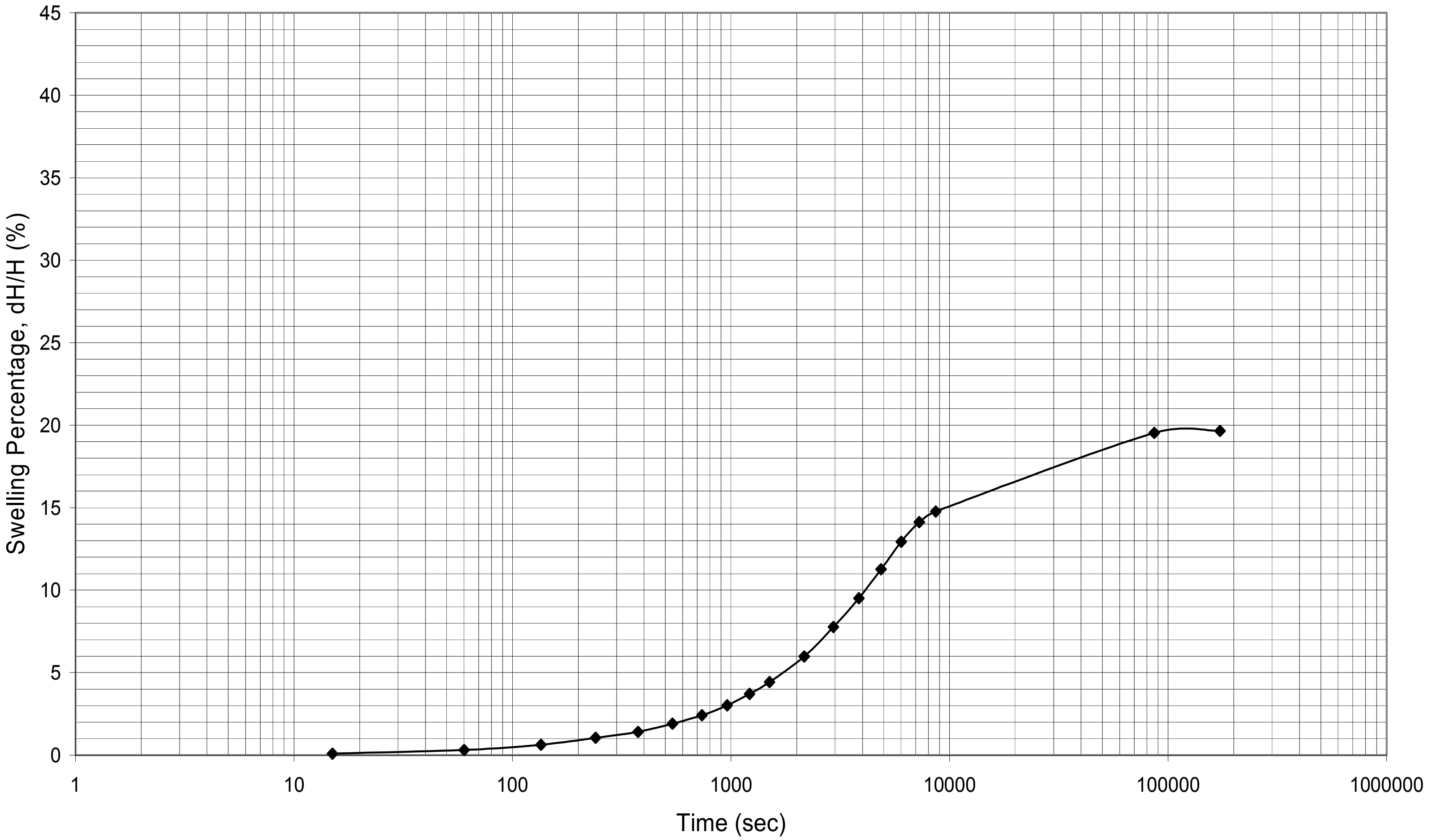

Figure A.10. Swelling Percentage vs. Time Relationship for Sample 20%DSG

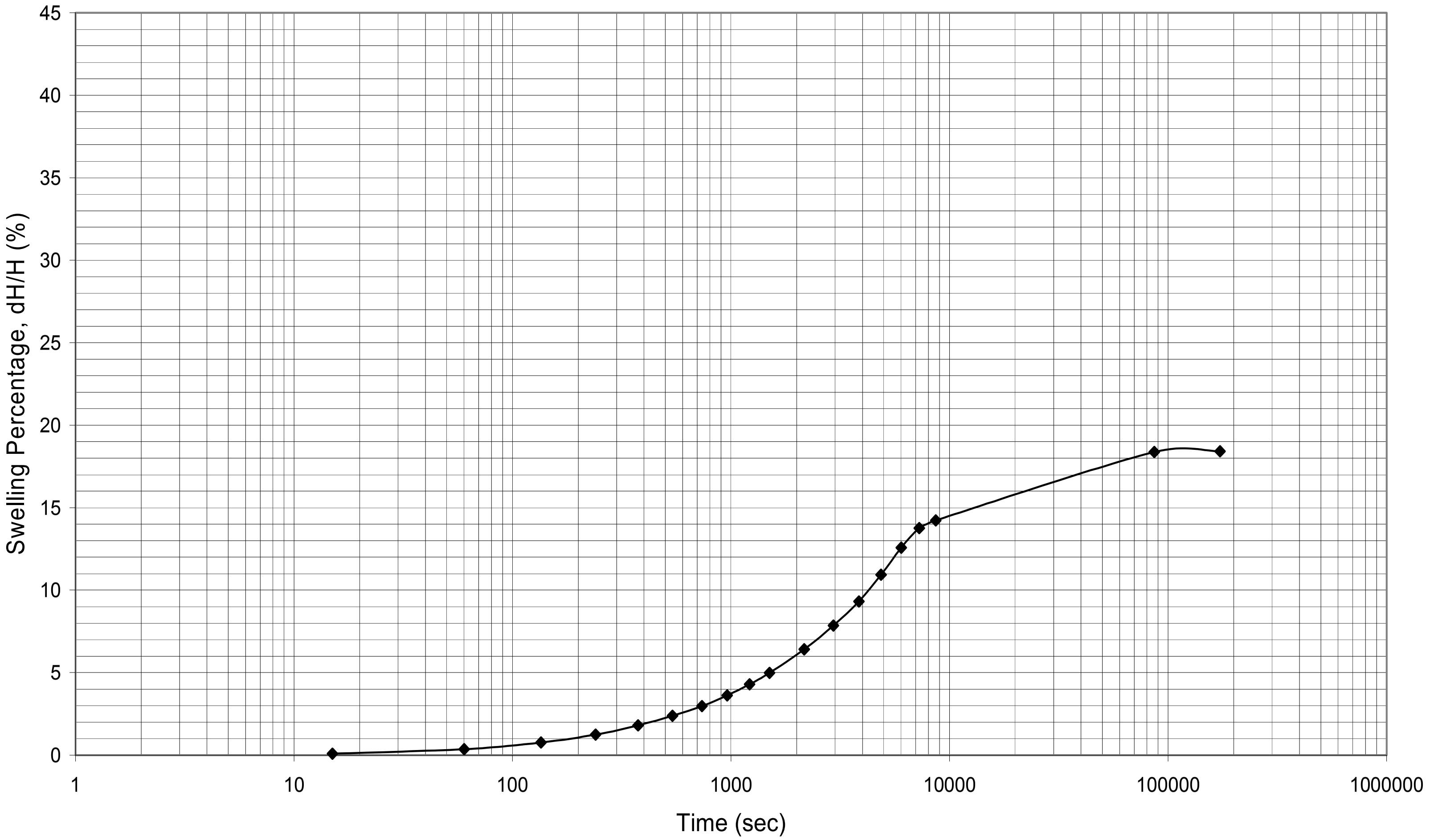

Figure A.11. Swelling Percentage vs. Time Relationship for Sample 25%DSG

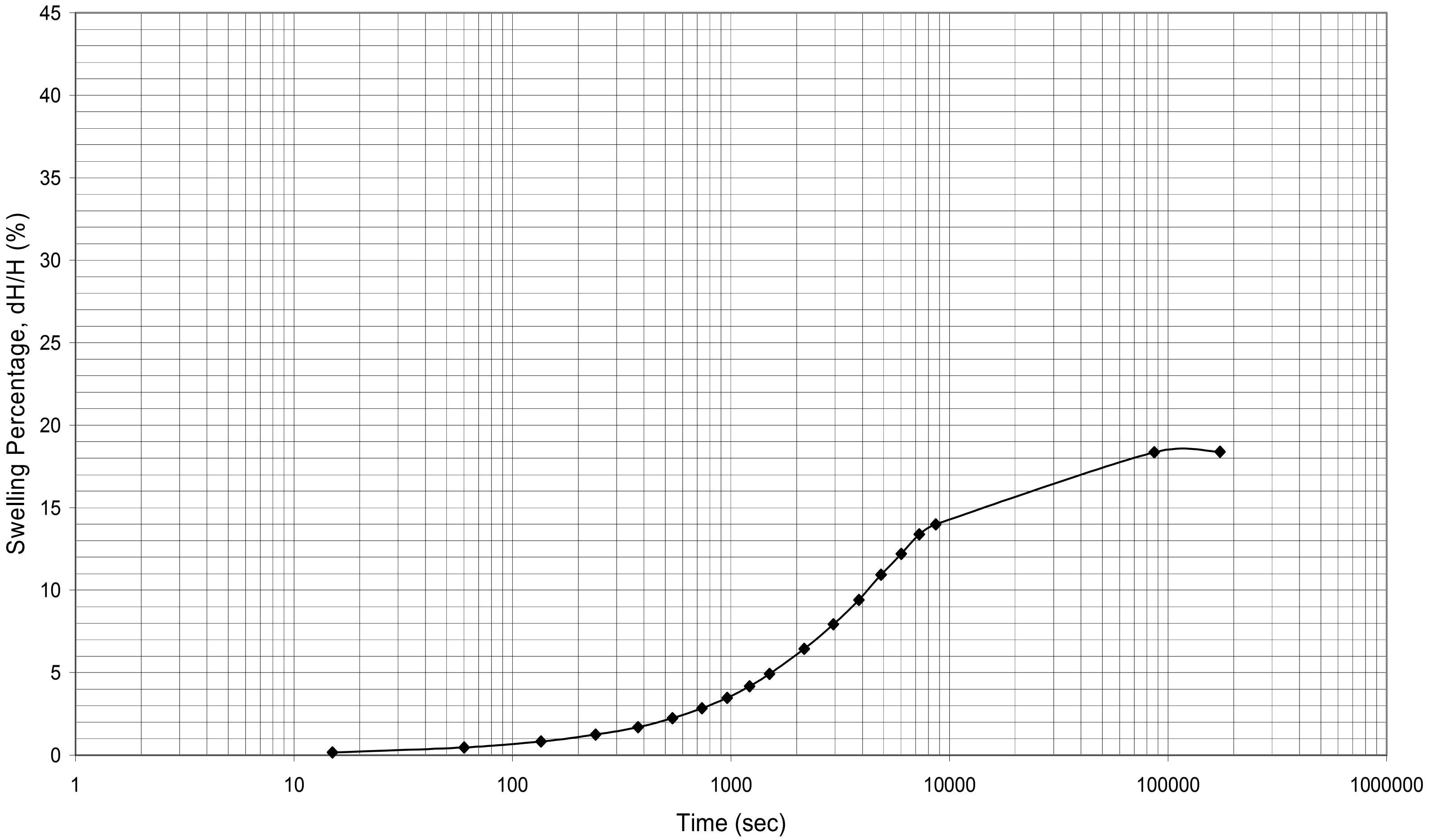

Figure A.12. Swelling Percentage vs. Time Relationship for Sample 30%DSG

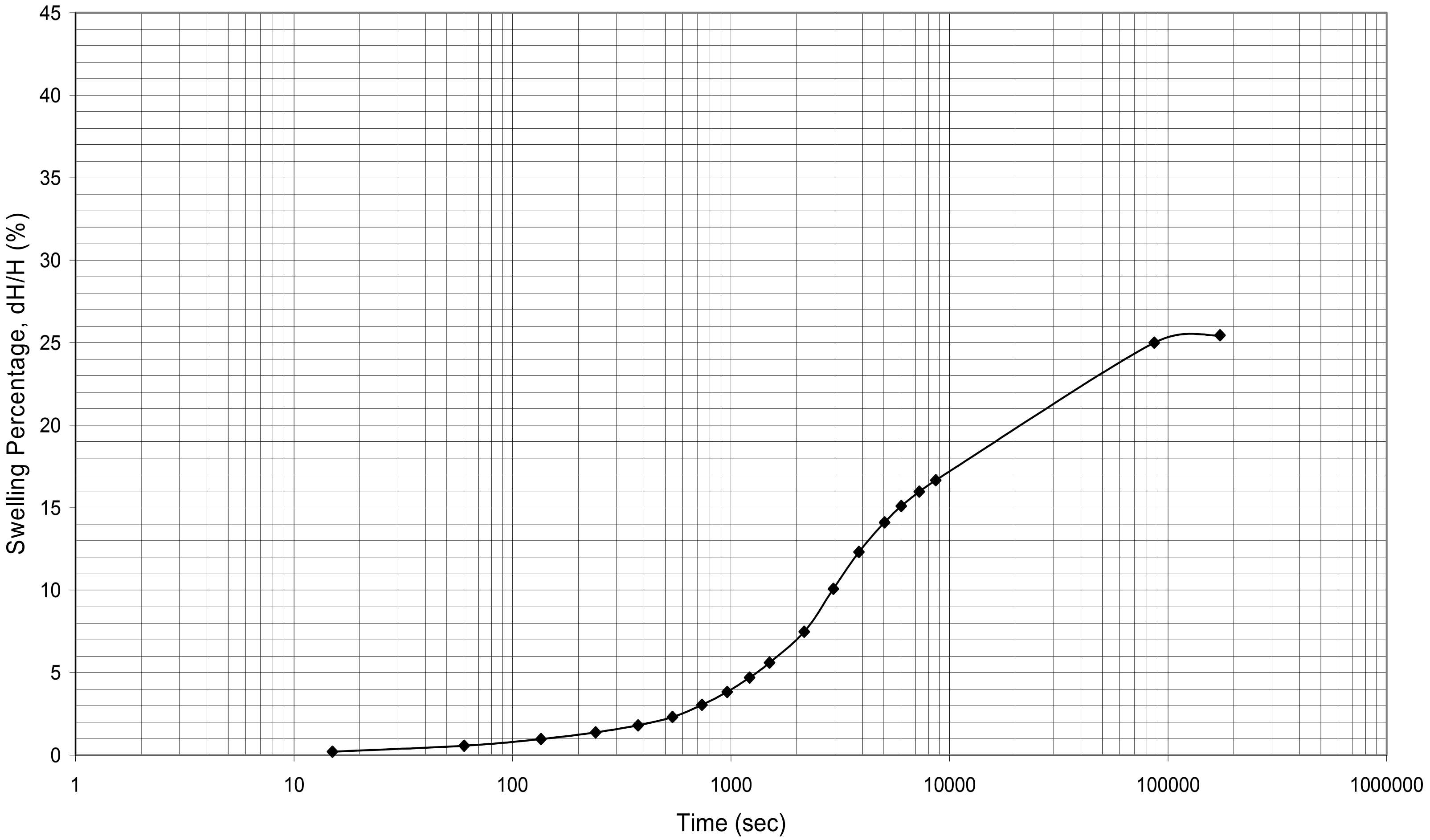

Figure A.13. Swelling Percentage vs. Time Relationship for Sample 1%L

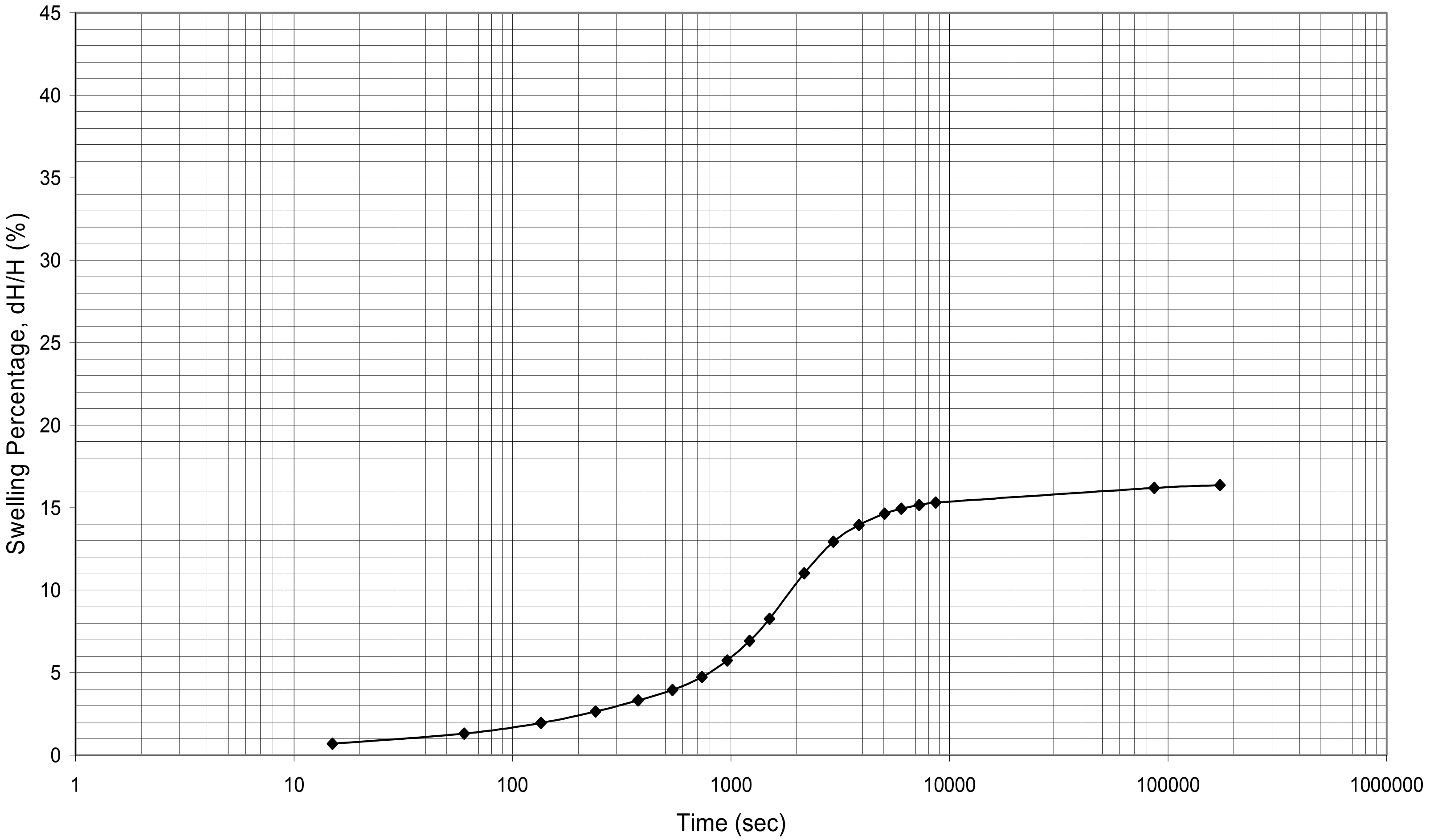

Figure A.14. Swelling Percentage vs. Time Relationship for Sample 3%L

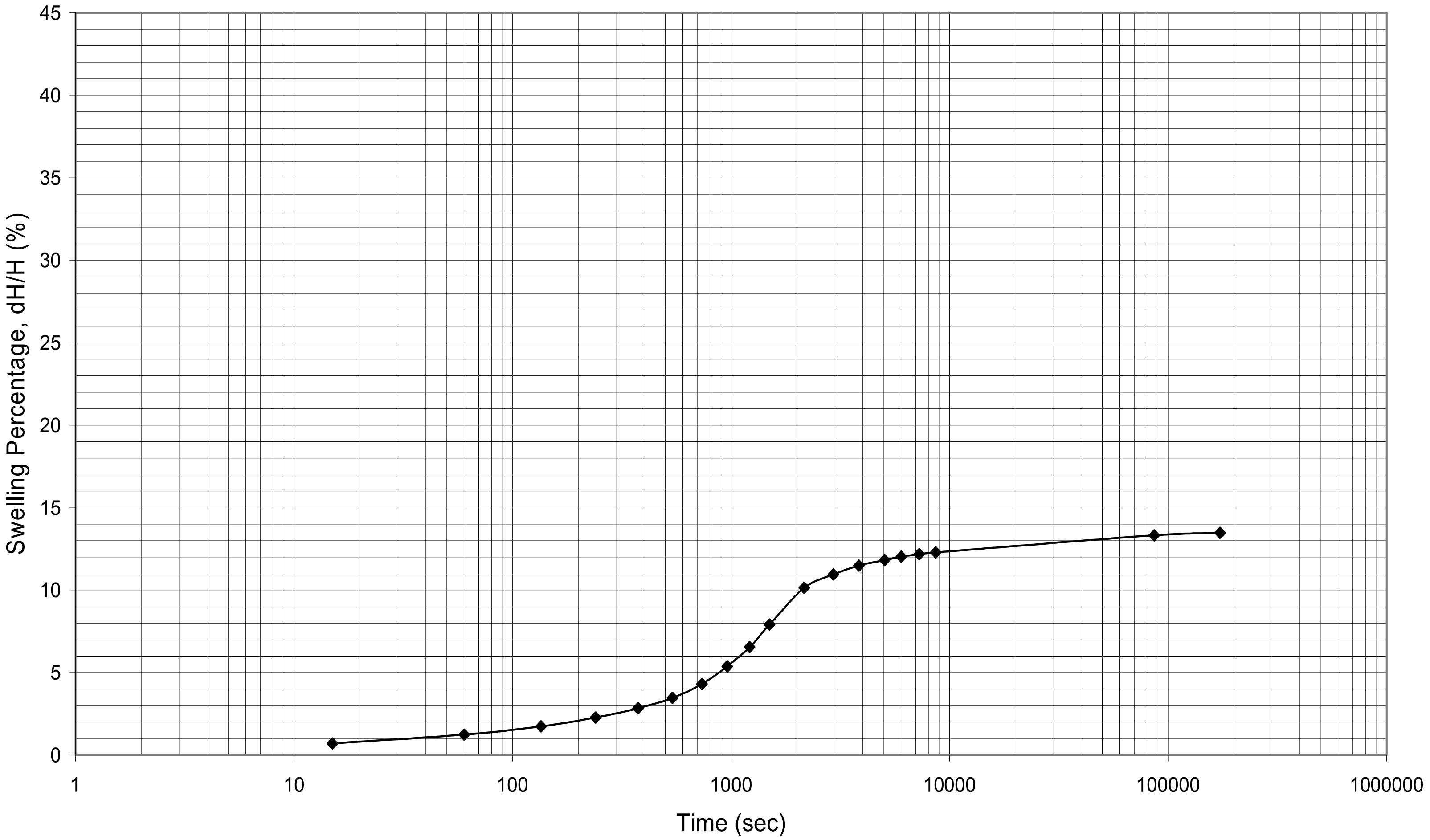

Figure A.15. Swelling Percentage vs. Time Relationship for Sample 5%L

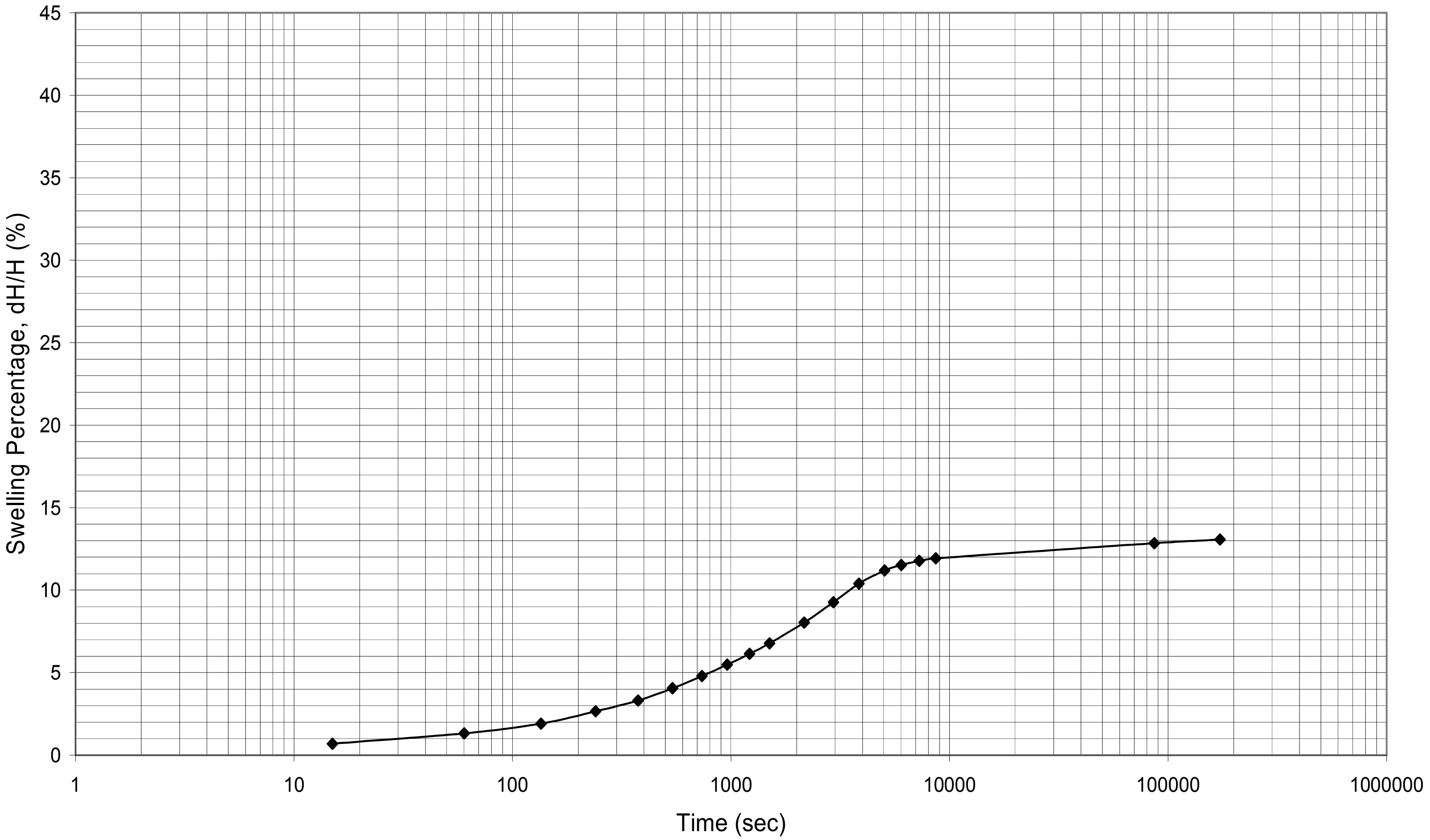

Figure A.16. Swelling Percentage vs. Time Relationship for Sample 8%L

APPENDIX B

Swelling percentage vs. time relationships of the 7 days and 28 days cured samples are given in Appendix B.

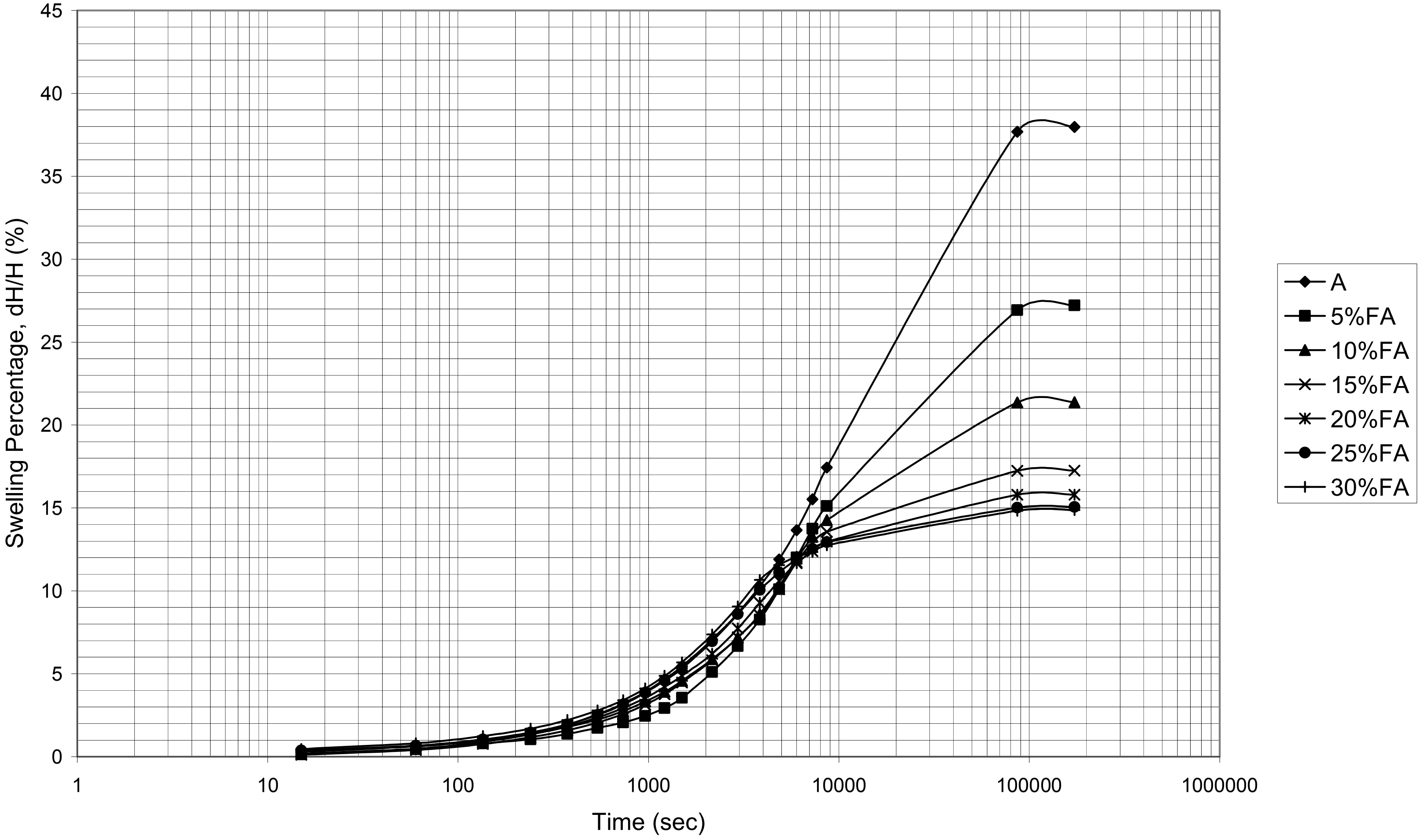

Figure B.1. Swelling Percentage vs. Time Relationships for the Fly Ash Added Samples after 7 Days Curing

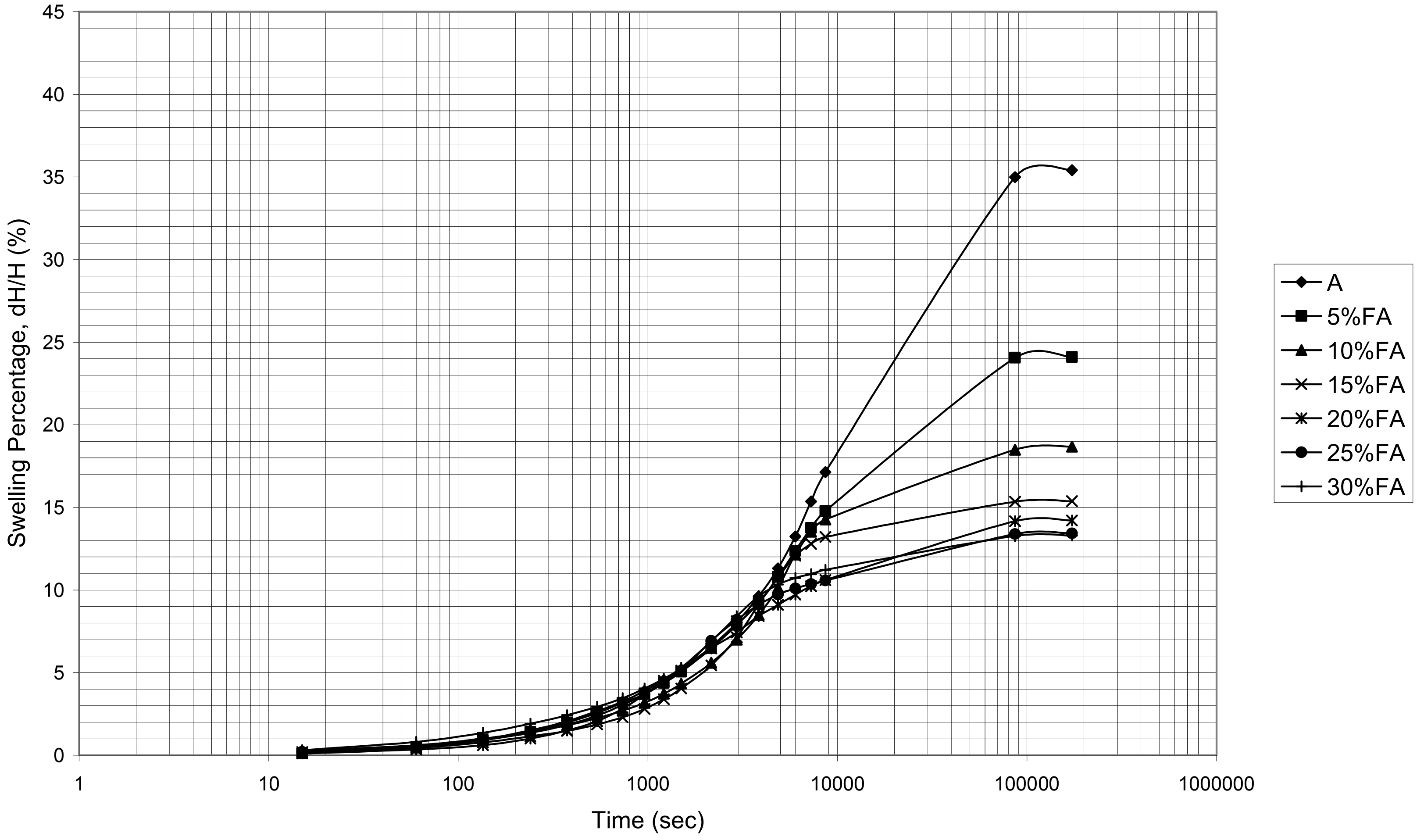

Figure B.2. Swelling Percentage vs. Time Relationships for the Fly Ash Added Samples after 28 Days Curing

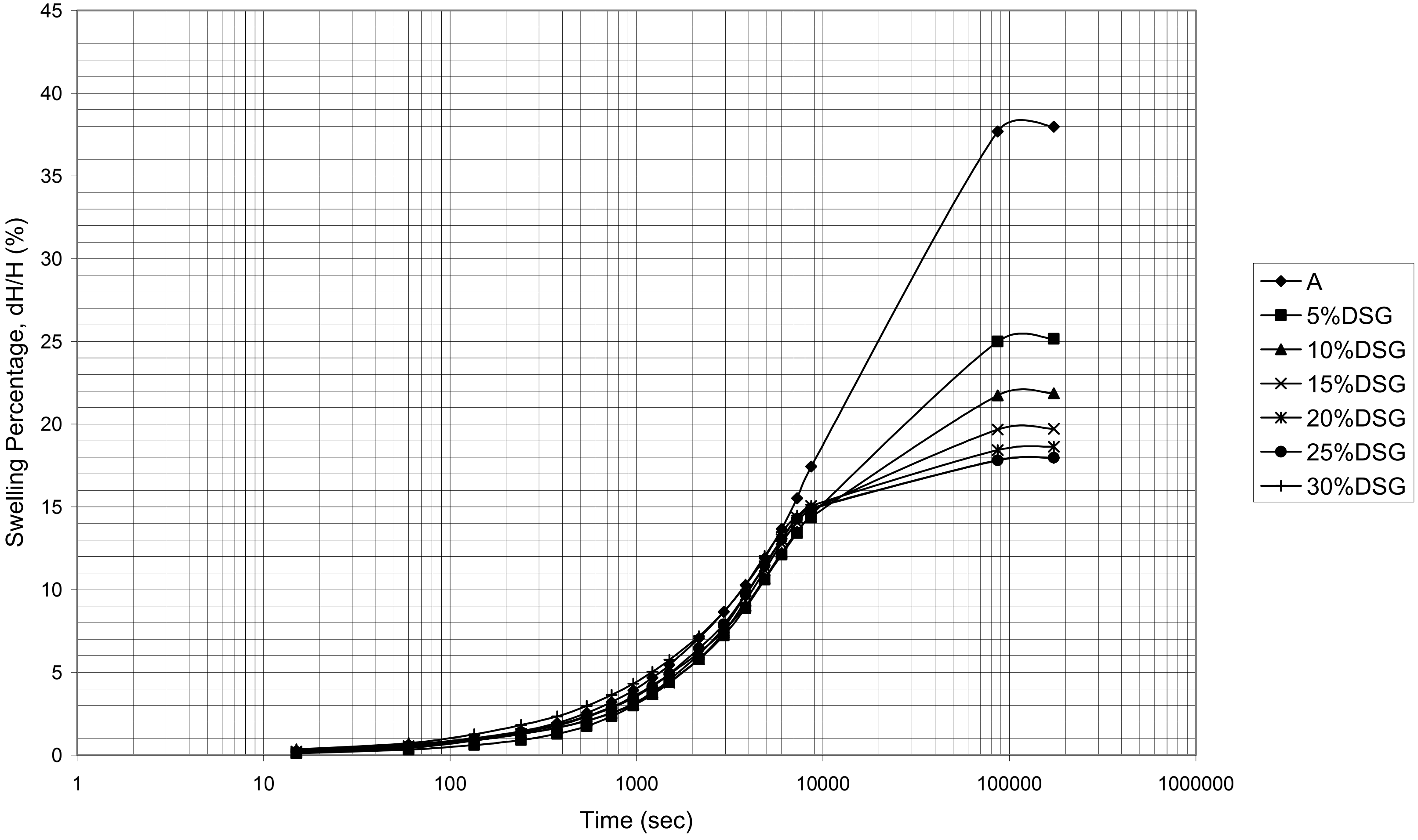

Figure B.3. Swelling Percentage vs. Time Relationships for the Desulphogypsum Added Samples after 7 Days Curing

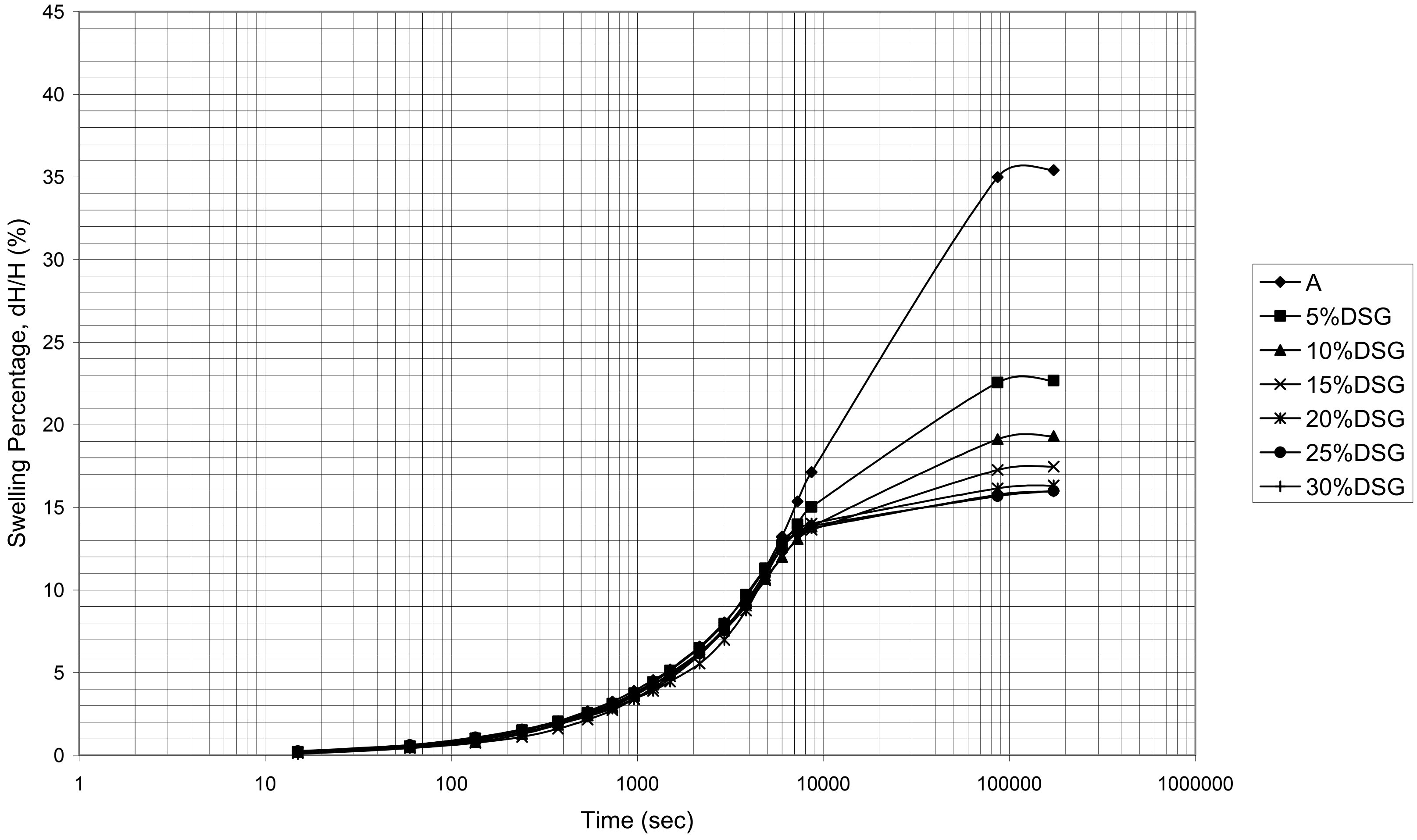

Figure B.4. Swelling Percentage vs. Time Relationships for the Desulphogypsum Added Samples after 28 Days Curing

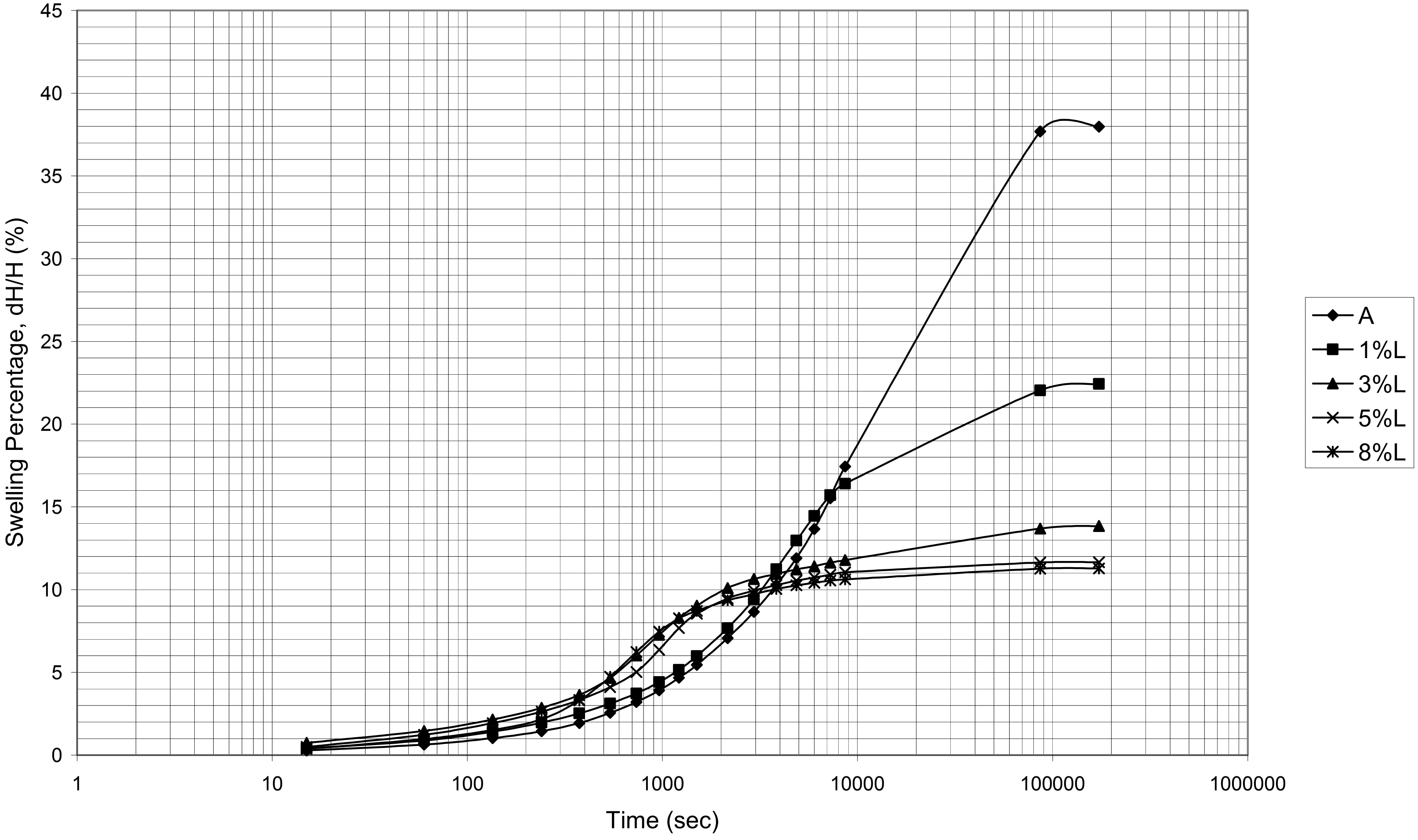

Figure B.5. Swelling Percentage vs. Time Relationships for the Lime Added Samples after 7 Days Curing

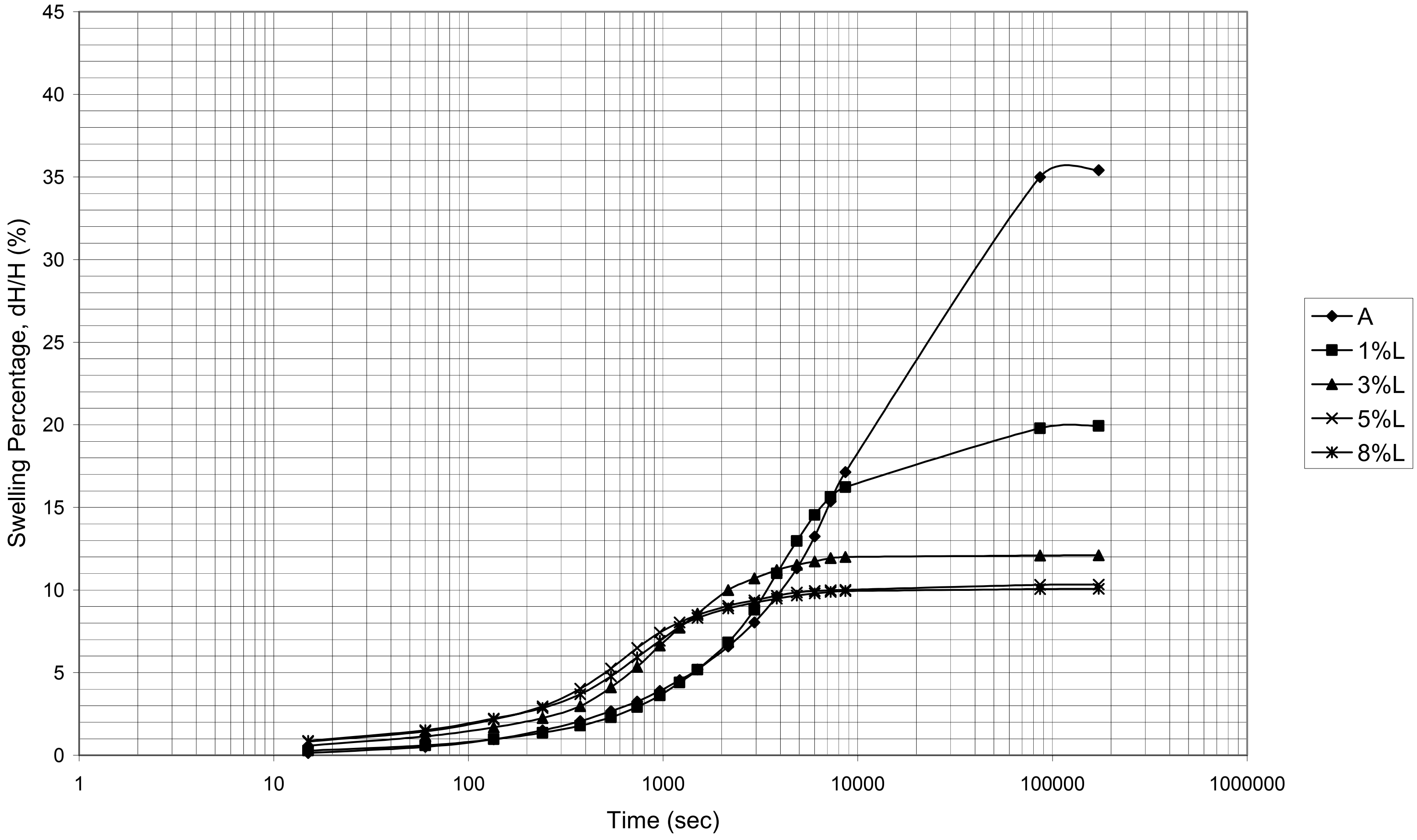

Figure B.6. Swelling Percentage vs. Time Relationships for the Lime Added Samples after 28 Days Curing

APPENDIX C

Grain Size distribution curves of Sample A, fly ash, measured 25%FA, and calculated 25%FA are given in Appendix C.

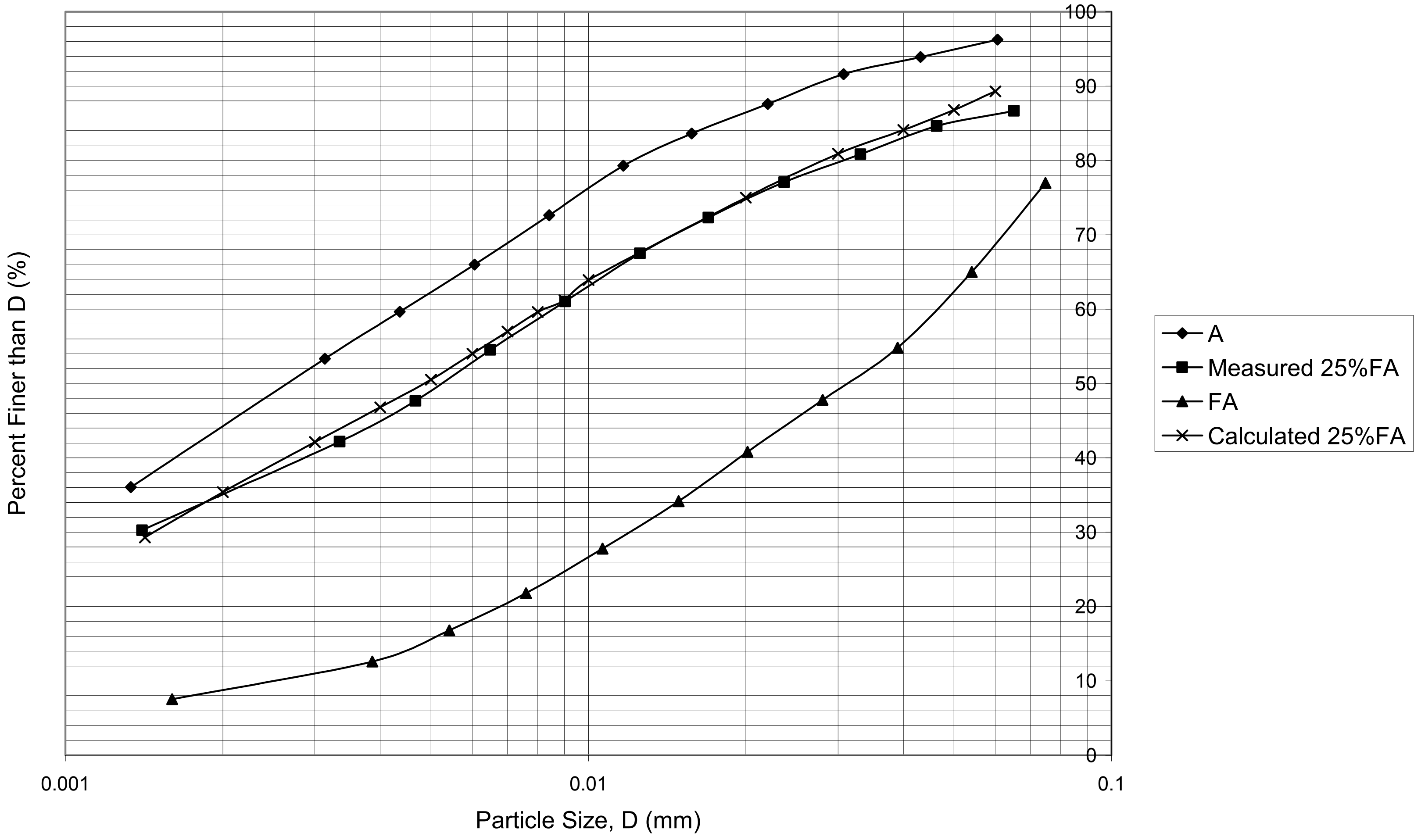

Figure C.1. Grain Size Distribution Curves of Sample A, Fly Ash, Measured 25%FA, and Calculated 25%FA