STABILIZATION OF EXPANSIVE SOILS USING AGGREGATE WASTE, ROCK POWDER AND LIME

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ABSTRACT

STABILIZATION OF EXPANSIVE SOILS USING AGGREGATE WASTE, ROCK POWDER AND LIME

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Expansive soils are a worldwide problem that poses several challenges for civil engineers. Such soils swell when given an access to water and shrink when they dry out. The most common and economical method for stabilizing these soils is using admixtures that prevent volume changes. In this study the effect of using rock powder and aggregate waste with lime in reducing the swelling potential is examined. The expansive soil used in this study is prepared in the laboratory by mixturing kaolinite and bentonite. Lime was added to the soil at 0 to 9 percent by weight. Aggregate waste and rock powder were added to the soil at 0 to 25 percent by weight. Grain size distribution, Atterberg limits and swell percent and rate of swell of the mixtures were determined. Specimens were cured for 7 and 28 days. This method of treatment caused a reduction in the swelling potential and the reduction was increased with increasing percent stabilizers.

KEYWORDS: Aggregate Waste, Curing, Expansive Soils, Lime, Soil Stabilization, Waste Rock Powder.

ÖZ

ŞİŞEN ZEMİNLERİN AGREGA ARTIĞI, TAŞ UNU VE KİREÇ KULLANILARAK STABİLİZASYONU

Yeşilbaş, Gülşah
Yüksek Lisans, İnşaat Mühendisliği
Tez Yöneticisi: Prof. Dr. Erdal ÇOKÇA

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Şişen zeminler dünya çapında bir problem olup, inşaat mühendislerine çeşitli sorunlar teşkil etmektedir. Bu tür zeminler suya maruz bırakıldıklarında şişip, kurutulduklarına büzüşürler. Bu zeminlerin stabilizasyonu için en yaygın ve ekonomik metod hacim değişikliklerini önleyici katkılar kullanmaktır. Bu çalışmada taş unu ile agrega artığının şişme potansiyeline etkisi incelenmiştir. Kullanılan şişen zemin numunesi laboratuarda kaolin ve bentonit karıştırılarak hazırlanmıştır. Kireç ağırlıkça %0 ila %9 arasında katılmıştır. Agrega artığı ve taş unu ise ağırlıkça %0 ila %25 oranında katılmıştır. Örneklerin dane çapı dağılımı, kıvam limitleri, şişme yüzdeleri ve şişme hızı tespit edilmiştir. Ayrıca 7 ve 28 gün kür uygulanmıştır. Bu metotla iyileştirme şişme potansiyelinde azalmaya neden olmuştur ve şişme yüzdesi katkı miktarı arttıkça azalmıştır.

ANAHTAR KELİMELER: Agrega Artığı, Atık Taş Unu, Kireç, Kürleme, Şişen Zeminler, Zemin Stabilizasyonu.

To My Grandma

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CHAPTER 1

INTRODUCTION

Some partially saturated clayey soils are very sensitive to variations in water content and show excessive volume changes. Such soils, when they increase in volume because of an increase in their water contents, are classified as expansive soils.

Problem of expansive soils has appeared as cracking and break-up of pavements, railways, highway embankments, roadways, building foundations, slab-on-grade members and, channel and reservoir linings, irrigation systems, water lines, sewer lines. (Gromko, 1974; Wayne et al. 1984; Mowafy et al. 1985; Kehew, 1995)

It is reported that damage to the structures due to expansive soils has been the most costly natural hazard in some countries. In the United States damage caused by expansive clays exceeds the combined average annual damage from floods, hurricanes, earthquakes, and tornadoes (Jones and Holtz, 1973). Documented evidence of the problems associated with expansive clays is worldwide, having occurred in such countries as the United States, China, Australia, India, Canada, and regions in Europe. (Popescu, 1986) It is reasonable that studies on the problem of expansive soils become more important day by day if the durative deficit of world resources and economy is taken into consideration. (Cited in Ipek, 1998)

When geotechnical engineers are faced with expansive soils, the engineering properties of those soils may need to be improved to make them suitable for construction. (Muntohar and Hantoro, 2002)

1.1. Review on Expansive Soils

Expansive soil term is used for the soils that have potential shrinking and swelling property under changing water content. Foundations constructed on expansive soils will be exposed to huge uplift forces due to swelling and these forces will result many structural problems.

Unsaturated expansive clays are a type of soils that, because of their mineralogical composition, usually experience large swelling strains when wetted. If the clay is in a saturated state, large swelling strains are also observed when the soil is unloaded. These soils have also been referred to in the literature as active clays and swelling clays. (Gens and Alonso, 1992)

Swelling clays are found in many parts of the world, particularly in semi-arid areas. In a review of Chen (1988), swelling clays are detected in Australia, Canada, China, Israel, Jordan, Saudi Arabia, India, South Africa, Sudan, Ethiopia, Spain, and the United States. (Wibawa, 2003) This is not to say that such soils do not exist elsewhere, for, indeed, they can be found almost everywhere.

It is obvious that the lightweight structures (damage to a lightweight structure is possible when very little volume expansion takes place in soil) that are designed and built by conventional techniques will be damaged in case of heaving. (Kehew, 1995) But although highway embankments and roadways are generally insensitive to vertical movements, high maintenance costs should be overcome if constructed on expansive soils. (Mowafy et al. 1985) The moisture may come from rain, flooding, leaking water or sewer lines or from reduction in surface evapotranspiration when an area is covered by a building or pavement. As it is mentioned above this problem is due to the mineralogical composition of these soils. Especially soils containing the clay mineral montmorillonite (a smectite) generally exhibit these properties. To understand and overcome these problems, expansive soils should be examined carefully and unsaturated soil mechanics should be taken into consideration.

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

1. Wide and deep shrinkage cracks occuring during dry periods,

2. Soil is rock-hard when dry, but very stiff and sticky when wet,

3. Damages on the surrounding structures due to expansion of soil.

1.2. Clay Mineralogy

There are many types of clays according to their physical and chemical properties. To understand behavior of clays, it should be first concentrated on their micro structures.

The basic structural units of most clay minerals consist of a silica tetrahedron and an alumina octahedron. Silicon and aluminium may be partially replaced by other elements in these units, this being known as isomorphous sustitution. The silicon-oxygen tetrahedron (silica tetrahedron) consists of four oxygens nestled around a silicon atom. (Figure 1.1.a) Alumina octahedron consists of an aluminium atom surrounded octahedrally by six oxygen ions. (Figure 1.1.b)

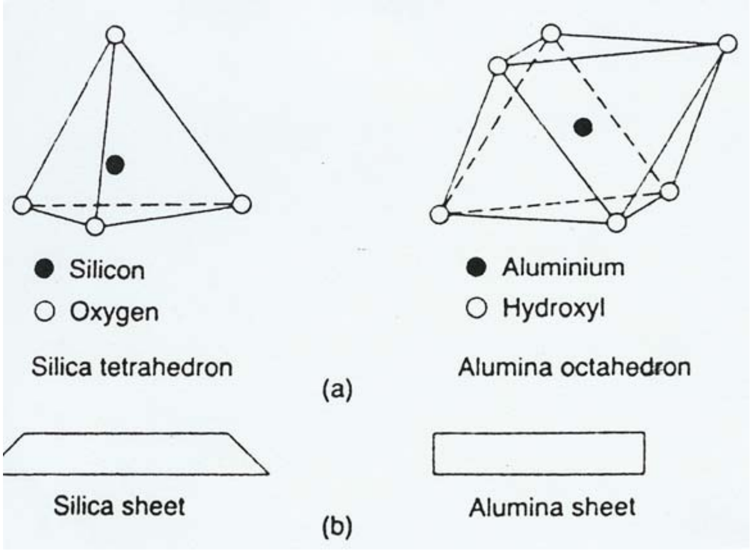

Figure 1.1Clay Minerals: Basic Units. (Craig, 1994)

The basic units combine to form sheet structures. Silica sheet is the combination of silicon-oxygen tetrahedrons. Alumina sheet is formed by combination of alumina octahedrons. Diagrammatic sketches of the kaolinite and montmorillonite structures are shown on Figures 1.2 and 1.3.

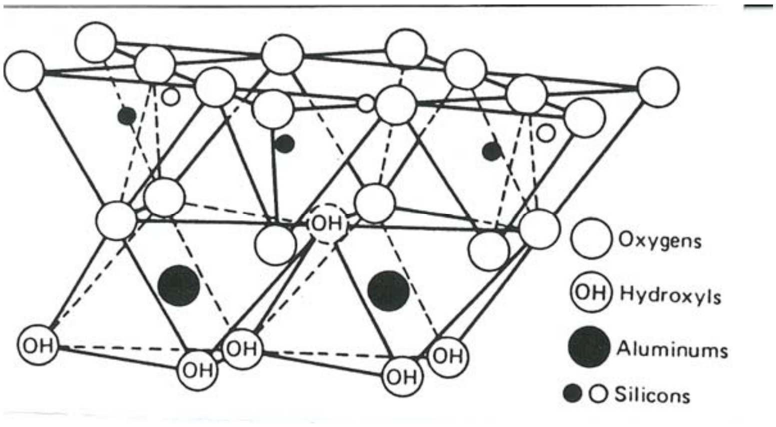

Figure 1.2 Diagrammatic Sketch of the Kaolinite Structure. (Craig, 1994)

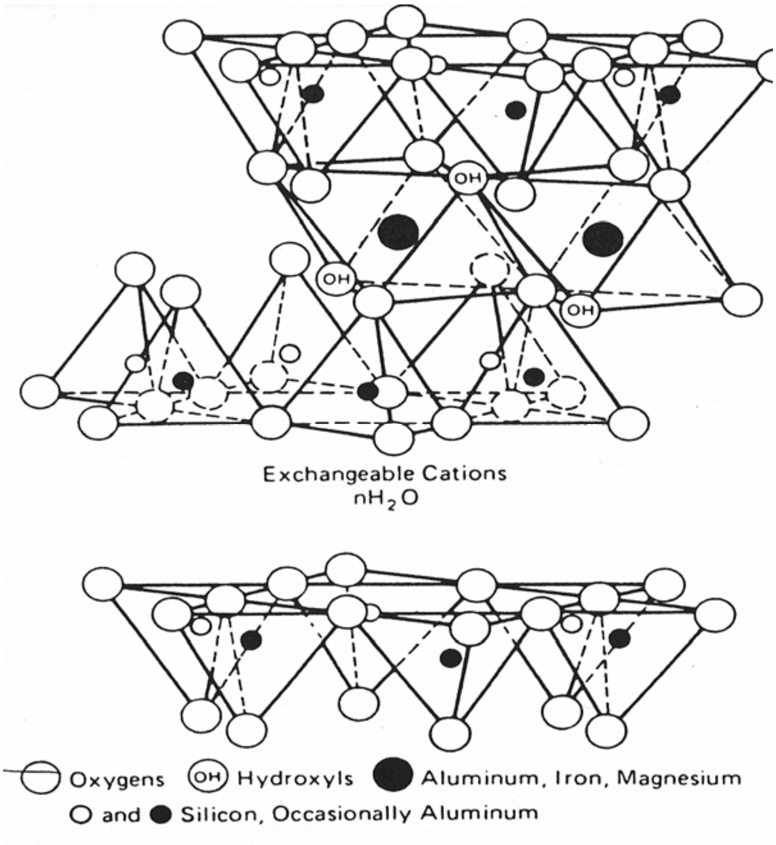

Figure 1.3 Diagrammatic Sketch of the Montmorillonite Structure. (Craig, 1994)

The various clay minerals are formed by the stacking of combinations of the basic sheet structures with different forms of bonding between the combined sheets.

Kaolinite consists of a structure based on a single sheet of silica tetrahedrons combined with a single sheet of alumina octahedrons. There is very limited isomorphous substitution. The combined silica-alumina sheets are held together fairly tightly by hydrogen bonding: a kaolinite particle may consist of over one hundred stacks (Figure 1.4).

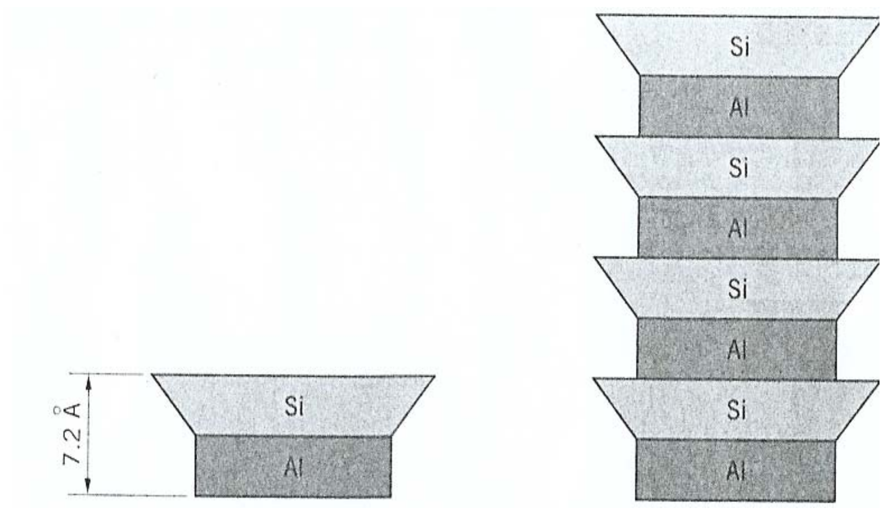

Figure 1.4 The Kaolinite Mineral. (Cernica, 1995)

Illite has a basic structure consisting of a sheet of alumina octahedrons between and combined with two sheets of silica tetrahedrons. The combined sheets are linked together by fairly weak bonding due to potassium ions held between them (Figure 1.5).

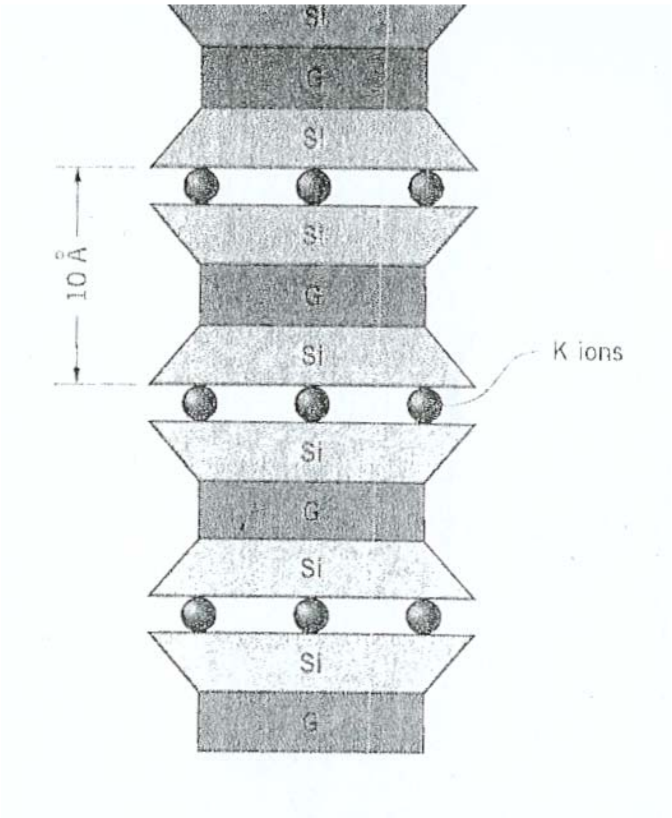

Figure 1.5 The Illite Clay Mineral. (Cernica, 1995)

Montmorillonite has the same basic structure as illite. In the octahedral sheet there is a partial substitution of aluminium by magnesium. The space between the combined sheets is occupied by water molecules and (exchangeable) cations other than potassium (Figure 1.6). There is a weak bond between the combined sheets due to these ions. Considerable swelling of montmorillonite can occur due to additional water being adsorbed between the combined sheets. (Craig, 1994) The large swelling capacity of montmorillonites, particularly sodium montmorillonites, marks these minerals as the most troublesome ones with respect to engineering design and construction. (Popescu, 1986)

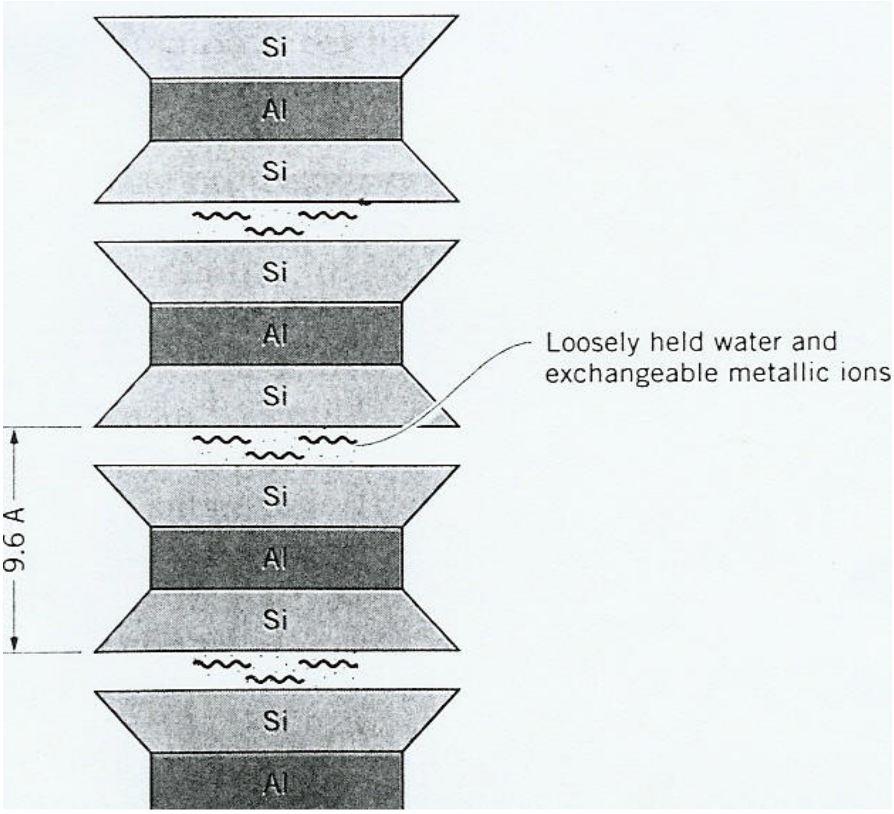

Figure 1.6 The Montmorillonite Clay Minerals. (Cernica, 1995)

Bentonite is part of the montmorillonite clay family, usually formed from the weathering of volcanic ash. It is noted for its expansive properties in the presence of water. As such, it was found to have beneficial uses as a general grout in preventing leakage from reservoirs, for plugging leaks in tunnel construction, and as a drilling mud in connection with soil borings and oil and gas wells. It prevents flocculation and facilitates the removal of the drill cuttings of the rotary drill. Also, it is sometimes used as a backfill for slurry trench walls, for clarification of beer and wine, and for other special applications. It has a liquid limit of 500% or more. (Cernica, 1995)

1.3. Mechanism of Swelling

There are two basic mechanisms involved in swelling phenomena:

1. Interparticle or intercrystalline swelling, shown diagrammatically in Figure 1.7, 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 effect is the same whether the clay has the form of particles as shown in the upper part of the figure or of crystals as shown in the middle part. The short dashes in the figure which link the layers of the clay crystals imply that the layers are strongly bonded by molecular forces.

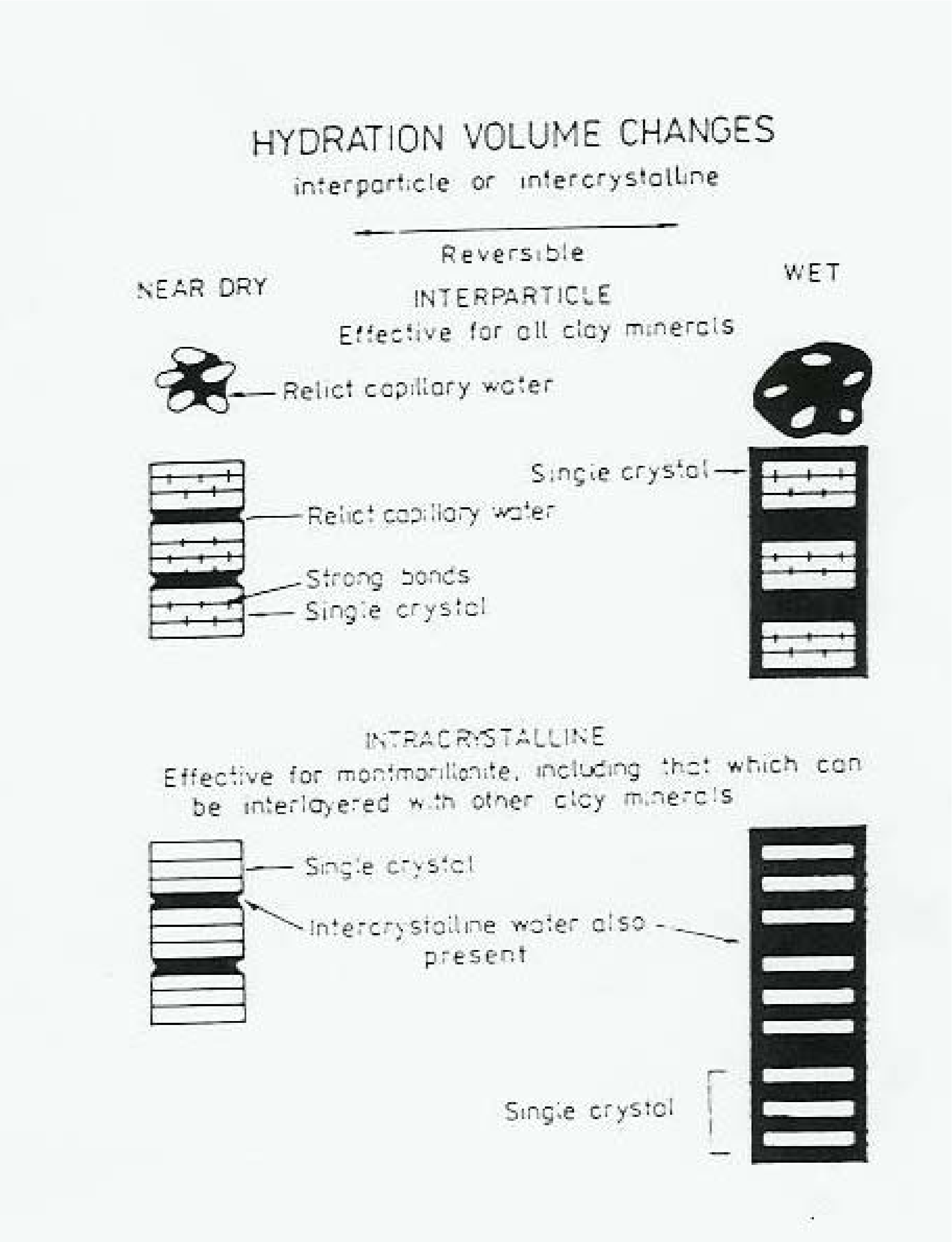

Figure 1.7 Mechanism of Swelling (After Popescu, 1986).

2. Intracrystalline swelling 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)

In montmorillonites the interlayer cations become hydrated, and the large hydration energy involved is able to overcome the attractive forces between the unit layers. Since in the prototype minerals interlayer cations are absent, there is no cation hydration energy available to separate the layers. (Cernica, 1995))

There can be two reasons of intracrystalline swelling: Clay particles are generally platelets having negative charges on their surfaces and positively charged edges (Figure 1.8). Cations in the soil water attach to the surfaces of the platelets and the negative charges on the surfaces of clay particles. The unbalanced electrostatic charges on clay-particle surfaces draw water molecules into the area between silicate sheets and force the sheets apart.

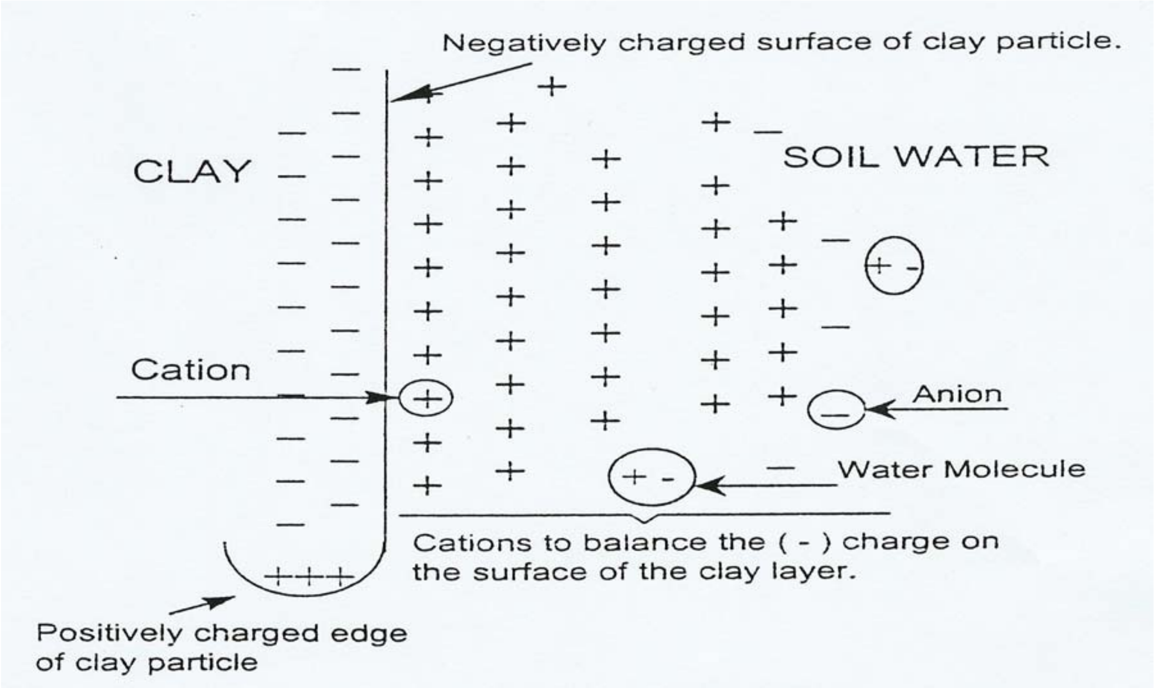

Figure 1.8 Internal Electrochemical System of Soil. (Mitchell, 1976)

The other factor is provided by cations attracted to the clay surfaces. Due to the attraction of negatively charged clay surfaces for the cations, the concentration of cations between the clay-particle surfaces is higher than the concentration of cations in the pore fluid. This creates an osmotic potential difference between the pore fluid and clay-mineral surfaces. In the actual case cations should migrate from the intracrystalline spacing (higher potential) to the intracrystalline spacing (lower potential) to equalize the cation concentration. But due to the attraction of clay surfaces, cations can not move and water moves into the area between clay-mineral surfaces. Due to this condition a repulsive force is exerted on the clay-mineral surfaces and the volume of clay soil increases (Figure 1.9). (Kehew, 1995)

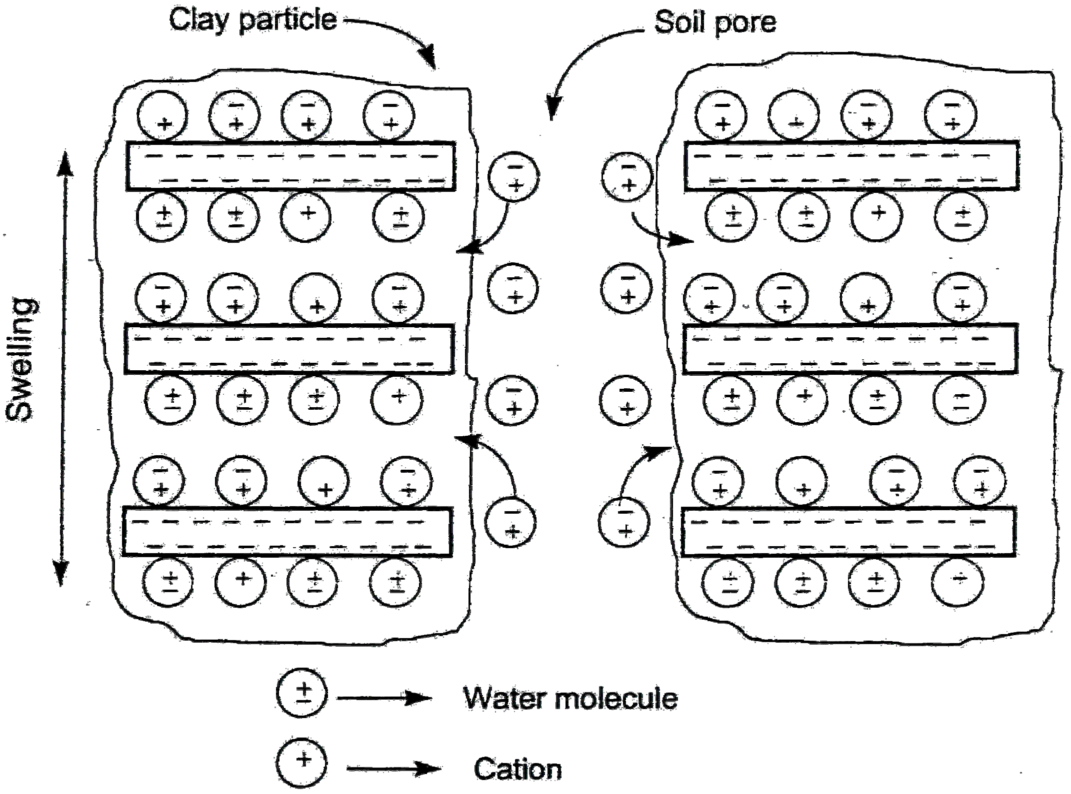

Figure 1.9 Swelling of Clay Soils (After Kehew, 1995).

1.4. Factors Affecting Swelling

Many attempts have been made in the past to understand the swelling behavior of soils. El-Sohby and Rabba (1981) stated that the primary factors which affect the swelling of soils are as follows: the initial water content, the type and amount of clay mineral, the initial dry density and percentage of coarse-grained fraction. El-Sohby and Mazen (1983) studied the effect of mineralogical composition (clay content, clay mineral type and exchangeable ions) on the swelling behavior of expansive soils and grouped soils according to the activity of the clay minerals present.

As it is previously mentioned the swell of soil is due to the presence of expanding clay minerals, hydration of cations on clay surfaces and the release of intrinsic stresses caused by overconsolidation or dessication of soils. Soil properties, including the composition of soil (mineral constituents), pore fluid, dry density and soil structure, primarily determine the potential for swell, whereas environmental conditions such as climate, groundwater, drainage, vegetation cover, confinement, and field permeability determine the actual amount and rate of swell.

Soil properties and environmental conditions, which influence the swell potential, are summarized in Table 1.1 and Table 1.2 (Nelson and Miller, 1992).

Table 1.1 Soil Properties that Influence the Swell Potential. (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
INITIAL WATER CONTENT	As the initial water content increases, initial degree of saturation will increase, affinity of soil to absorb water will decrease and the amount of swelling will decrease.
INITIAL DRY DENSITY	The higher values of initial dry densities cause closer particle spacings, greater repulsive force between particles, larger swelling potential and pressure.
CLAY CONTENT & MINERALOGY	Clay minerals which have swell potential are montmorillonites, vermiculates, and some mixed layer minerals. As the percentage of clay increases swelling potential increases.
COARSE GRAINED FRACTION	The more the amount of fine particles the higher the swell potential and swell percent.
COMPOSITION OF SOLIDS	Active clay minerals are composed of montmorillonites and mixed layer combinations of montmorillonites and other clay minerals.
CONCENTRATION OF PORE FLUID SALTS	High concentration of cations in the pore fluid tends to reduce magnitude of volume change.
COMPOSITION OF PORE FLUID	Prevelance of monovolent cations increase shrink-swell potential; divalent and trivalent cations inhibit shrink swell.
SOIL STRUCTURE & FABRIC	Flocculated clays are more expansive than dispersive clays. By compacting at higher water content or by remolding, fabric and structure are changed. Kneading compaction create dispersive structures with lower swell potential than statically compacted soils.

Table 1.2 Environmental Conditions that Influence the Swell Potential. (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
CLIMATE	Arid climates promote desiccation while humid climates promote wet soil properties.
GROUNDWATER	Fluctuating and shallow water tables provide a source of moisture for heave.
DRAINAGE	Poor surface drainage leads to moisture accumulation or ponding.
VEGETATIVE COVER	Vegetation (trees, shrubs, etc.) deplete moisture from soil through transpiration and cause accumulation of moisture areas denuded of vegetation.
CONFINEMENT	Larger confining pressures reduce swell; cut areas are more likely to swell; lateral pressures may not equal vertical overburden pressures.
FIELD PERMEABILITY	Fissures can significantly increase permeability and promote faster rates of swell.

1.5. Oedometer Methods to Measure Swelling Properties

Many tables and charts are available in the literature that can be used to obtain approximate values of swell parameters namely free swell and swell pressure. Oedometer methods are the easiest and most widely used methods in paractice. (El Fatih and Muawia, 1984)

To study the swell behaviour simple laboratory oedometer tests may be performed. These test procedures are described in Annual Book of ASTM Standards V04.08 with designation number D4546-90, three alternative test methods are proposed for the determination of the magnitude of swell for soil samples. Percent heave is defined as the increase or decrease in the ratio of the change in vertical height, ΔH, to the original height, H, of column of soil; ΔH/H×100.

The three methods to determine the swell pressure or percent heave are as follows (ASTM D4546-90, 1993):

Method A (Figure 1.10):

The specimen is inundated and allowed to swell vertically at the seating pressure applied by the loading plate and the top porous stone until primary swell is complete. (Step 3-4) After primary swell has occurred, the specimen is loaded until its initial void ratio/height is obtained. (Step 4-6)

Method A can be modified to place an initial vertical stress 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. (Step 1-2) Then the vertical stress is removed, except for the seating pressure. Deformation is recorded within 5 minutes after removal of the vertical stress. (Step 2-3), the specimen is inundated, and the test is continued as explained in the preceding paragraph. This method measures 1. The primary swell, 2. Percent heave for vertical confining pressures up to the swell pressure, and 3. The swell pressure

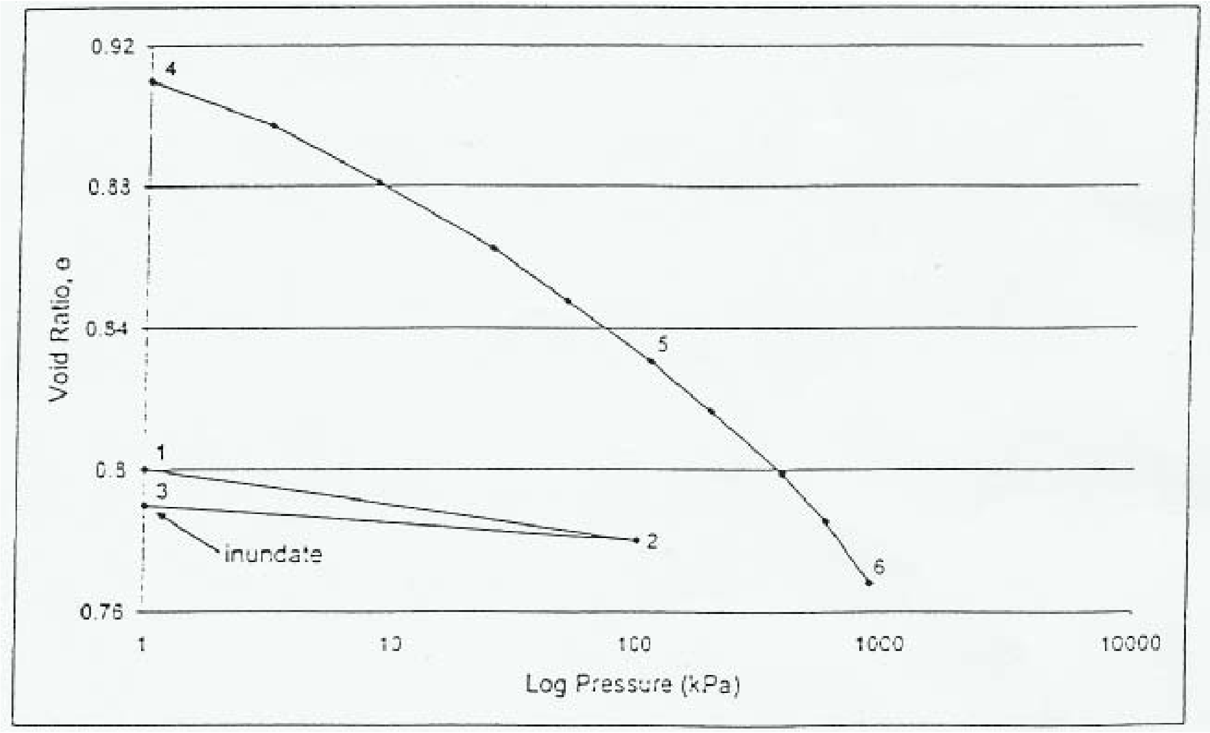

Figure 1.10 Void Ratio-Log Pressure Curve for Method A

Method B (Figure 1.11):

Before introducing free water into the consolidometer, a vertical pressure exceeding the seating pressure is applied to the specimen. The specimen is inundated. The specimen may swell; swell and then contract, contract then swell consequently. After the movement becomes negligible, the amount of swell or settlement is measured. This method measures 1. 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 2. The swell pressure

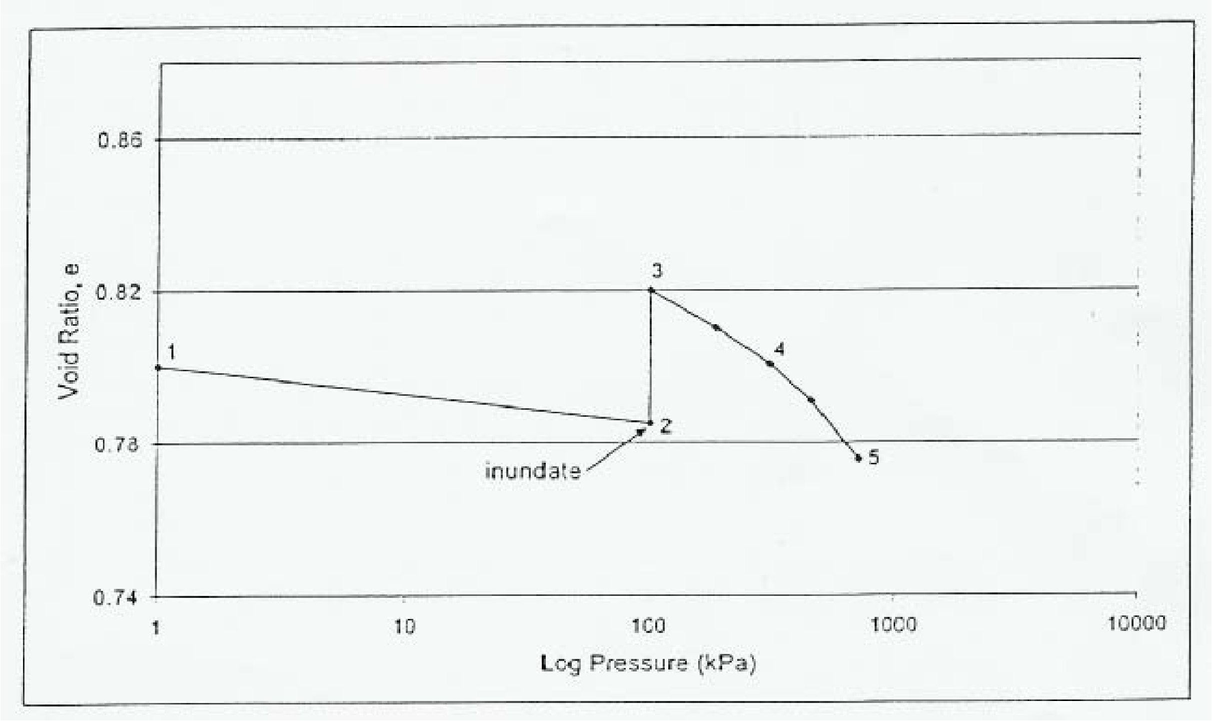

Figure 1.11 Void Ratio-Log Pressure Curve for Method B

Method C (Figure 1.12):

The specimen is inundated by giving access to free water. By making adjustments in vertical pressure, the specimen is maintained at constant height. The rebound curve following consolidation is determined. This method measures 1. The swell pressure, 2. Preconsolidation pressure, and 3. Percent heave or settlement within the range of applied vertical pressures.

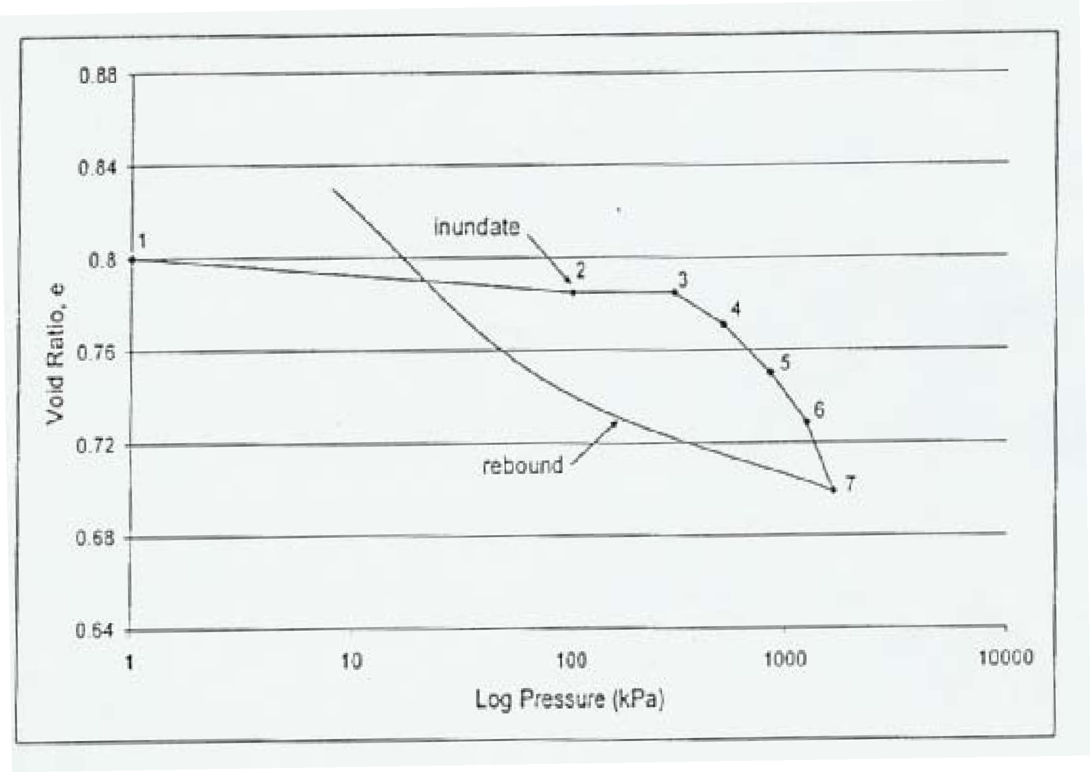

Figure 1.12 Void Ratio-Log Pressure Curve for Method C

1.6. Determination of Rate of Swell (t_{50})

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. (Basma and Tuncer, 1991)

CHAPTER 2

SOIL STABILIZATION

In geotechnical engineering practice the soils at a given site are often less than ideal for the intended purpose. It would seem reasonable in such instances to simply relocate the structure or facility. However, considerations other than geotechnical often govern the location of a structure, and the engineer is forced to design for the site at hand. One possibility is to adapt the foundation to the geotechnical conditions at the site. Another possibility is to try to stabilize or improve the engineering properties of the soils at the site. Depending on the circumstances, this second approach may be the most economical solution for the problem. Stabilization is usually mechanical or chemical, but even thermal and electrical stabilization have occasionally been used or considered. (Craig, 1994)

One method to improve expansive soils is chemical stabilization. Chemical stabilization includes the mixing or injecting of chemical substances into the soil. Portland cement, lime, asphalt, calcium chloride, sodium chloride, and paper mill wastes are common chemical stabilization agents. 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.

In recent years, researchers from many fields have attempted to solve the problems posed by industrial wastes. Finding a way for the utilization of these wastes would be an advantageous way of getting free of them. Recent projects illustrated that successful waste utilization could result in considerable savings in construction costs. (Kamon and Nontananandh, 1991)

The stabilization of an expansive soil by using lime, rock powder and the waste of aggregates is considered in this thesis study.

2.1. Lime Stabilization

It is an age-old practice to use lime in one form or the other to improve the engineering behavior of clayey soils. Because of the proven success of lime stabilization in the field of highways and air-field pavements, this technique is now being extended for deep in-situ treatment of clayey soils to improve their strength and reduce compressibility. The improvements in the properties of soil are attributed to the soil-lime reactions (Clare and Cruchley, 1957; Ormsby and Kinter, 1973; Locat et al. 1990).

Lime stabilization is covered extensively in the literature (Rogers and Glendinning, 2000; Quaint et al. 2000; Little et al. 1987; Mitchell, 1986; NLA, 1985; Armani and Moonfish, 1972; Stocker, 1972; Thompson, 1969). Lime will primarily react with medium, moderately fine, and fine-grained soils to produce decreased elasticity, increased workability, reduced swell, and increased strength. Such improved soil properties are the result of three basic chemical reactions (Fang, 1991): 1.Cation exchange and flocculation-agglomeration 2.Cementation (pozzolanic reaction); and 3. Carbonation

The cation exchange process involves an agglomeration of the fine clay particles into coarse particles. The cementation process develops from the reaction between calcium present in lime and silica and alumina in the soil, forming calcium-silicate and calcium-aluminate or calcium-aluminate-silicates. The cementitious compounds produced are characterized by their high strength and low-volume change. Previous researchers reported that small lime additions (from 2% to 8%) significantly decrease the liquid limit, plasticity index, maximum dry density, and swell, and increase plastic limit, the optimum moisture content, and strength of expansive soils (Croft, 1967; Abduljauwad, 1995; Basma et al., 1998). It was reported by Sivapullaiah et al., (1997) that lime added in excess of the amount required for cation exchange could only produce cementitious compounds, which blind the flocculated particles and develop extra strength. (Al-Rawas et al., 2002)

The most commonly used products are hydrated high calcium lime Ca $(OH)_2$, MgO, calcitic quick lime CaO, and dolomitic quick lime CaO.MgO. Quick lime is used widely for soil stabilization (TRC180, 1982). Hydrated lime is a fine powder, whereas quicklime is a more granular substance. Quick lime is more caustic than hydrated lime, so additional safety procedures are required with this material. The type of the lime used as a stabilizing agent varies from country to country. Although using quick lime is more popular in Europe, hydrated lime is used mainly for stabilization but proportion of quick lime that is used increased to about 25% in 1987 from about 15% in 1976 (Rollings and Rollings, 1996). According to

McCallister and Petry (1988) both calcium hydroxide [Ca (OH_2)] and quick lime (CaO) are common and effective for the physicochemical treatment of expansive clays.

2.1.1. Lime-Soil Reactions

When lime is added to the soil, hydration of the lime causes an immediate drying of the soil. Anhydraous quicklime will have a more pronounced drying effect than hydrated lime. Consequently, lime can prove to be an effective construction expedient for drying out wet sites.

If lime is added to a plastic soil, plasticity drops, and texture changes. The chemical changes occurring in the soil are usually explained with the help of some established mechanisms suggesting cation exchange, flocculation, and aggregation. The first two reactions are known to occur immediately after lime is either added or allowed to diffuse into the soil whereas the third reaction is time bound and temperature dependent and can be considered as a long term reaction. Cation exchange is an important reaction and is believed to be mainly responsible for the changes occurring in the plasticity characteristics of the soil. Depending on the availability of various types of cations in the pore fluid, cation replacement can take place. In general, the cations are arranged in the order of their replacing power according to the Iyotropic series, $Li^+ < Na^+ < H^+ < K^+ < NH_4^+ < Mg^{2+} < Ca^{2+} < Al^{3+}$, i.e., any cation will tend to replace the left of it and monovalent cations are generally replaced by multivalent cations. The replacement of sodium or potassium ions with calcium will significantly reduce the plasticity index of a clay mineral.

The addition of lime increases the soil pH, which also increases the cation exchange capacity. Consequently, even calcium-rich soils may respond to lime treatment with a reduction in the soil's plasticity. A reduction in plasticity is usually accompanied by reduced potential for shrinking or swelling.

Due to the addition of lime to the soil the texture of the soil is also changed. As a result of particle agglomeration clayey soils become more silty and sandy in behavior. The amount of clay-sized particles (2µm) decreases as the amount of lime in the soil lime mixtures increases. Verhasselt (1990) scrutinized various possible bonding mechanisms during the experimental research. According to his conclusions the mechanisms that cause particle agglomeration are most probably the hydrogen (H^+) and hydroxyl (OH^-) bonding by the calcium hydroxyl ($Ca\ (OH_2)$) functions on the clay particle surface. Stable and larger particles are formed by clay particles linked together by the relatively weak bonds effectively coarsening the texture of the clay soils. (Cited in Ipek, 1998)

Stabilization occurs when the proper amount of lime is added to reactive soil. Ingles and Metcalf (1972) recommended the criteria of lime mixture as shown in Table 2.1.

Table 2.1. Suggested Lime Contents (Ingles, 1972)

Soil Type	Content for Modification	Content for Stabilization

Fine crushed rock	2 – 4 percent	Not recommended
Well graded clay gravels	1 – 3 percent	~3 percent
Sands	Not recommended	Not recommended
Sandy clay	Not recommended	~5 percent
Silty clay	1 – 3 percent	2 – 4 percent
Heavy clay	1 – 3 percent	3 – 8 percent
Very heavy clay	1 – 3 percent	3 – 8 percent
Organic soils	Not recommended	Not recommended

Stabilization differs from modification in that significant level of long-term strength gain is developed through a long-term pozzolonic reaction. This pozzolonic reaction is the formation of calcium silicate hydrates and calcium aluminate hydrates as the calcium from the lime reacts with the aluminates and silicates solubilized from the clay mineral surface. This reaction can begin quickly and is responsible for some of the effects of modification. However, research has shown that the full term pozzolonic reaction can continue for a very long period of time- even many years- as long as enough lime is present and the pH remains high (Above about 10). As a result of this long-term pozzolonic reaction, some soils can produce very high strength gains when lime treated. The key to pozzolonic reactivity and stabilization is a reactive soil and a good mix design protocol. The results of stabilization can be very substantial increase in resilient modulus values (by a factor of 10 or more in many cases), very substantial improvements in shear strength (by a factor of 20 or more in some cases), continued strength gain with time even after periods of environmental or load damage (autogenously healing) and long-term durability over decades of service even under severe environmental conditions. (Wibawa, 2003) The change after adding lime to the soil is shown in Figure 2.1.

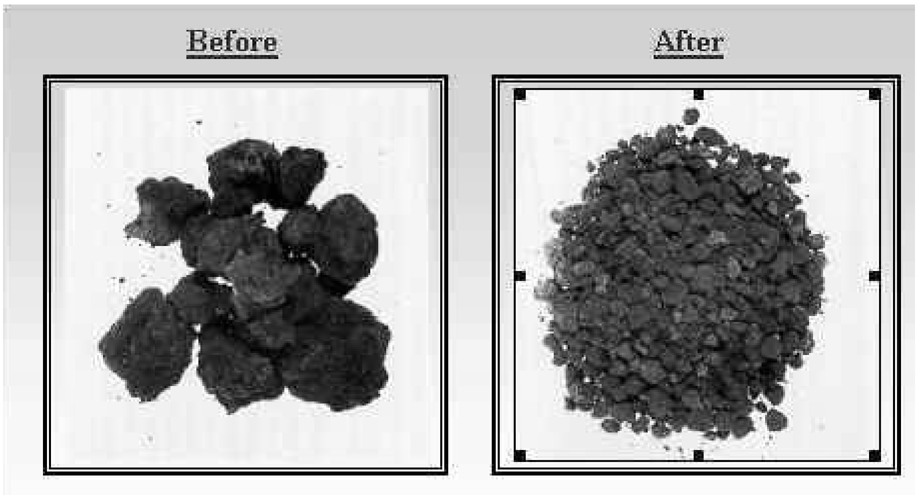

Figure 2.1.The Visual Effect of Lime Addition (Wibawa, 2003)

2.2 Stabilization by Waste Materials

When lime is not adequate to achieve the desired strength and improvement, lime in combination with some waste materials may provide the needed improvement. Recent research has demonstrated that moderate levels of lime and fly ash or rice husk etc. can achieve significant strength improvements in reclaimed soil and aggregate systems without producing extremely rigid and shrinkage sensitive systems. Generally, target strength can be achieved through a sound mixture design process which identifies a lime and a waste material combination which will achieve desired strength and resilient modulus properties. (Muntohar and Hantoro, 2002)

Recently, how to utilize resources and how to preserve natural environment have become more serious problems in the world. In considering of increasing amount of the various kinds of industrial waste matter which are by-products from the industrial activity, it is necessary to dispose or utilize them for construction materials. The requirements for utilizing by-products for construction materials are as follows; 1. Production of large amount is possible for a long period of time. 2. The materials are available everywhere. 3. Feasibility of quality control. 4. The materials do not cause environmental pollutions.

The crusher stones are required to use instead of natural gravels, because they are difficult to collect from the reason of environment preservation. Aggregate waste and rock powder are industrial by-products from crusher plants.

The materials used in this study as it is mentioned before are rock powder and the waste of aggregates which are used for concrete. These materials are inactive and silt size materials and use to stabilize expansive soils. Both materials may help to reduce swelling potential. Some characteristics of these materials are given in Chapter 3.

CHAPTER 3

EXPERIMENTAL WORK

3.1. Purpose

The purpose of the experimental work is to investigate the effects of addition of rock powder and aggregate waste 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 lime, rock powder and aggregate waste.

3.2. Material

Kaolinite: Kaolinite was taken from Eczacıbaşi Minerals Factory on Eskişehir Road (Ankara) in the form of gravel sized grains. These grains were crushed and passed through No. 40 sieve before usage.

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

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

Aggregate Waste: Aggregate waste is the waste of a quarry in Elmadağ (Ankara). It is the waste powder of the aggregate used in the process of concrete. Aggregate waste was passed through No. 40 sieve before usage. The specific gravity of aggregate waste is 2.38.

Rock Powder: Rock powder is the powder of the rock taken from the energy tunnels of 'Deriner Dam' in Artvin. The specific gravity of rock powder is 2.43. This material is used as an additive to concrete to improve workability and to increase compressive strength. Also it has been found out to be suitable as a supplement or replacement for cement in concrete. Using these materials in concrete or in stabilization gives an economical solution for most problems engineers face with.

Chemical and Mineralogical analyses to determine the chemical and mineralogical contents of Bentonite, Kaolinite and Lime were done by 'Cement Producers Association of Türkiye'.

The results of the chemical analysis are tabulated on Table 3.1.

Table 3.1 Results of Chemical Analysis of Kaolinite, Bentonite and Lime.

Chemical Composition (%)	Type of the Materials Used		
	Kaolinite	**Bentonite**	**Lime**
MgO	0.03	1.28	1.26
Al_2O_3	33.03	14.98	0.00
SiO_2	49.89	56.19	1.54
CaO	0.42	2.25	67.08
Fe_2O_3	1.78	9.45	0.03
SO_3	0.13	0.33	1.09
K_2O	1.69	1.19	0.05
Na_2O	0.08	2.41	0.02
TiO_2	1.33	1.11	0.32
Loss of Ignition	11.10	1.10	28.50

The results of mineralogical analyses are tabulated on Table 3.2.

Table 3.2 Results of Mineralogical Analysis of Kaolinite, Bentonite and Lime.

Mineralogical Constituents of the Materials Used	Type of the Materials Used		
	Kaolinite	**Bentonite**	**Lime**
	Kaolinite- $Al_2Si_2O_5\,(OH)_4$	Albite- $NaAlSi_3O_8$	Portlandite- $Ca(OH)_2$
	Illite- K-Na-Mg-Fe-Al-Si-O- H_2O	Illite- K-Na-Mg-Fe-Al-Si-O- H_2O	Calcite- $CaCO_3$
	Quartz- SiO_2	Quartz- SiO_2	Quartz- SiO_2
	Feldspar-$(Na_2K)AlSi_3O_8$		

Chemical analyses of aggregate waste and rock powder materials were done at ‘Materials and Construction Laboratory of METU’. (Table 3.3)

Table 3.3 Results of Chemical Analysis of Aggregate Waste and Rock Powder

Composition	Type of Materials Used	
	Aggregate Waste (AW) %	Rock Powder (RP) %
SiO_2	60.1	54.8
Al_2O_3	14.6	15.6
Fe_2O_3	4.6	6.3
CaO	6.7	7.7
MgO	3.1	4.2
SO_3	0.1	0.1
Loss of Ignition	5.7	7.9
Insoluble Residue	83.3	79.6

3.3. Preparation of Specimens

In order to eliminate the effects that come out as a result of using undisturbed samples that contribute to the results of the testing, a potentially expansive soil was prepared in the laboratory. Bentonite and Kaolinite were used to prepare a potentially expansive soil. Bentonite was Karakaya Bentonite. Kaolinite was taken from Eczacıbaşı Minerals Factory on Eskişehir Road. According to the preliminary studies; a soil sample composed of 15% Bentonite and 85% Kaolinite was designated as 'Sample A'.

In the beginning of the preliminary studies the waste materials to be used as stabilizing agents were pre-tested and the results of the tests showed that both materials are capable of reducing swell percent of the expansive soil made in the laboratory.

Twenty samples were prepared by adding lime, lime and aggregate waste, and lime and rock powder with different percentages. The purpose of preparing samples including only lime and Sample A was to see the effect of waste materials when used with the same amounts of lime.

Sample preparation technique was the same as done in Ipek’s thesis study (1992). Firstly, all materials used in this study were ground so that they could pass through No.40 sieve and oven-dried for 24 hours at 50 °C. To prepare Sample A, 15% Bentonite and 85% kaolinite were mixed roughly using a trowel. Then, to be able to mix such fine grained soil samples thoroughly; the constituents were sieved two times through No. 30 sieve. Each time only 150 gr of each sample was mixed. Because during the preliminary studies it was observed that mixing higher amounts prevented the particles distribute uniformly in the mixture, and this way affected the results. After mixing was complete 10% water was added.

Samples other than Sample A were obtained by mixing a calculated amount of stabilizer with Sample A to obtain a sample with predetermined percentage of stabilizer which varied from 0 to 9 percent for lime, 0 to 25 percent for aggregate waste and 0 to 25 percent for rock powder by dry weight of the soil which are tabulated in Table 3.4.

The same mixing procedure was applied for the experiment on cured samples. The samples prepared according to the procedure explained above were tightly encased by a plastic bag to prevent loss of moisture and were set to cure at 22 °C and 70 percent moisture for 7 days and 28 days.

Table 3.4 Specimens Used in the Experimental Study.

No	Sample A (%)	Lime (%)	Aggregate Waste (%)	Rock Powder (%)
1	100	-	-	-
2	99	1	-	-
3	97	3	-	-
4	95	5	-	-
5	93	7	-	-
6	91	9	-	-
7	80	-	20	-
8	80	1	19	-
9	80	3	17	-
10	80	5	15	-
11	80	7	13	-
12	80	9	11	-
13	75	-	25	-
14	80	-	-	20
15	80	1	-	19
16	80	3	-	17
17	80	5	-	15
18	80	7	-	13
19	80	9	-	11
20	75	-	-	25

3.4. Sample Properties

Hydrometer tests, Atterberg limit tests, specific gravity tests were performed according to the tests methods specified in ASTM Standards with designation number D2435 and particle size distribution, consistency limits, specific gravity of samples were determined. According to the grain size distribution curves, percentages of clay-sized and silt-sized particles were determined.

The sample properties are tabulated in Table 3.5.

Soil classification is done according to the Unified Soil Classification System. (Figure 3.1)

Table 3.5 Sample Properties

Sample[+]	Clay Percent	Silt Percent	Gs	LL (%)	PL (%)	PI (%)	SL (%)	Activity (PI/%Clay)
100% A	69.81	30.19	2.64	98.84	24.12	74.72	17.37	1.07
99% A + 1%L	67.90	32.10	2.66	88.77	18.94	69.83	16.43	1.03
97% A + 3%L	66.80	33.20	2.67	83.89	20.74	63.15	18.58	0.95
95% A + 5%L	61.70	38.30	2.69	82.45	24.05	58.40	19.22	0.95
93% A + 7%L	54.90	45.10	2.70	77.52	24.75	52.77	22.15	0.96
91% A + 9%L	53.60	46.40	2.72	72.54	25.86	46.59	25.56	0.87
80% A + 20%AW	59.59	40.41	2.53	83.65	21.08	62.57	13.50	1.05
80% A + 19%AW+1%L	61.30	38.70	2.55	82.50	24.22	58.28	14.21	0.95
80% A + 17%AW+3%L	57.89	42.11	2.56	81.60	28.30	53.31	19.69	0.92
80% A + 15%AW+5%L	61.03	38.97	2.57	76.78	30.88	45.90	22.03	0.75
80% A + 13%AW+7%L	60.89	39.11	2.58	73.64	29.53	44.11	23.27	0.72
80% A + 11%AW+9%L	58.42	41.58	2.59	72.01	28.50	43.51	24.82	0.74
75% A + 25%AW	54.49	45.51	2.55	80.69	19.56	61.13	13.48	1.12
80% A + 20%RP	59.59	40.41	2.56	87.69	21.60	66.09	15.51	1.11
80% A + 19%RP+1%L	63.00	37.00	2.57	83.35	19.70	63.65	17.57	1.01
80% A + 17%RP+3%L	61.98	38.02	2.58	81.85	26.30	55.55	19.59	0.90
80% A + 15%RP+5%L	63.00	37.00	2.59	74.54	28.93	45.61	24.25	0.72
80% A + 13%RP+7%L	61.30	38.70	2.60	72.30	31.34	40.96	26.39	0.67
80% A + 11%RP+9%L	57.89	42.11	2.62	71.57	31.88	39.69	27.52	0.69
75% A + 25%RP	56.19	43.81	2.57	78.77	21.14	57.63	15.33	1.03

[+] A: Expansive Soil Sample (85% Kaolinite+15% Bentonite), L: Lime, AW: Aggregate waste, RP: Rock Powder.

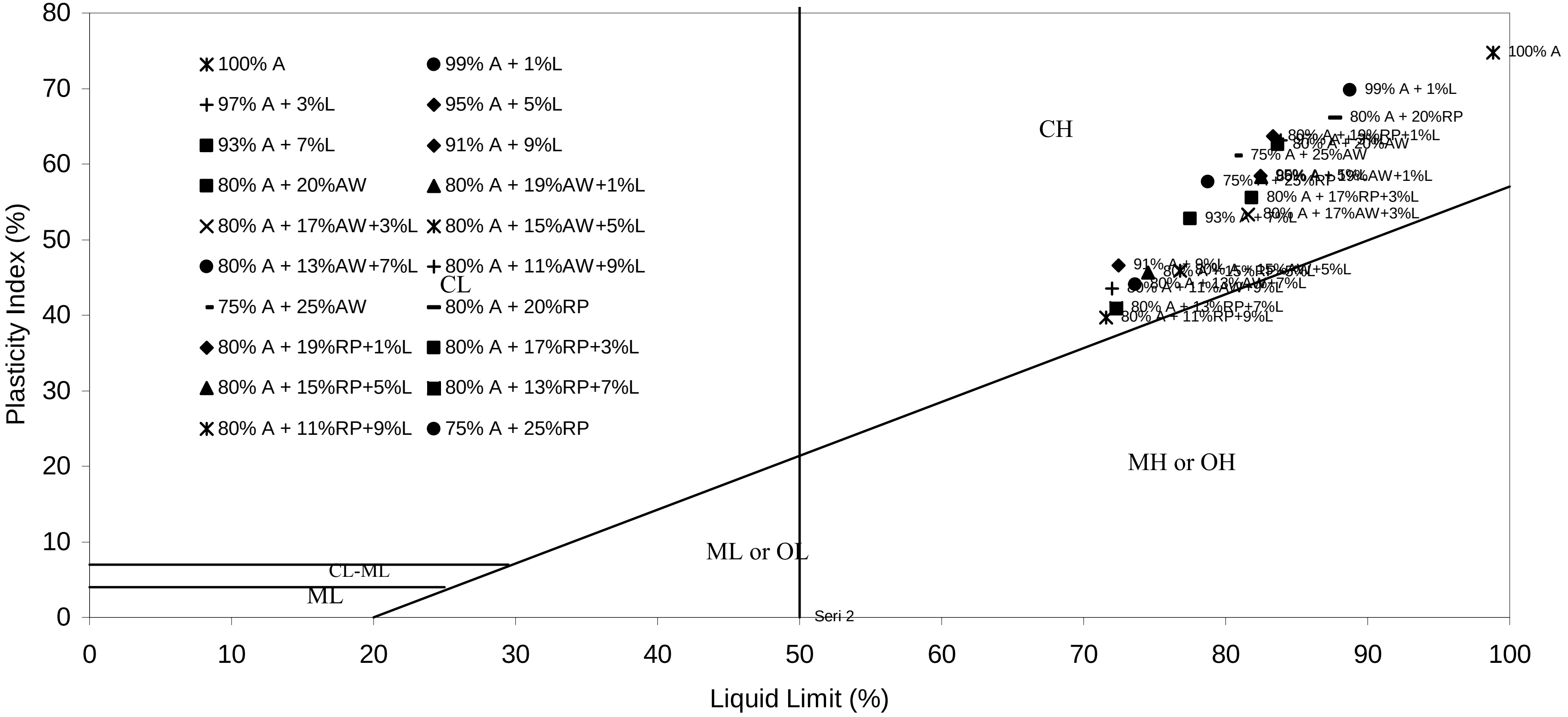

Figure 3.1 Plasticity Chart: Unified System

Swelling potential of each sample was calculated according to the PI values and clay percentages listed on Table 3.6 and the chart of Seed et al. was used to determine swelling potential degrees of the samples (Figure 3.2 and Table 3.6).

Grain size distribution curves of the samples were grouped according to the type of additive and plotted on the same graph, plotting the grain size distribution curve of Sample A on each graph to be able to determine whether there was a shifting due to the addition of stabilizer or not (Figures 3.3, 3.4, 3.5).

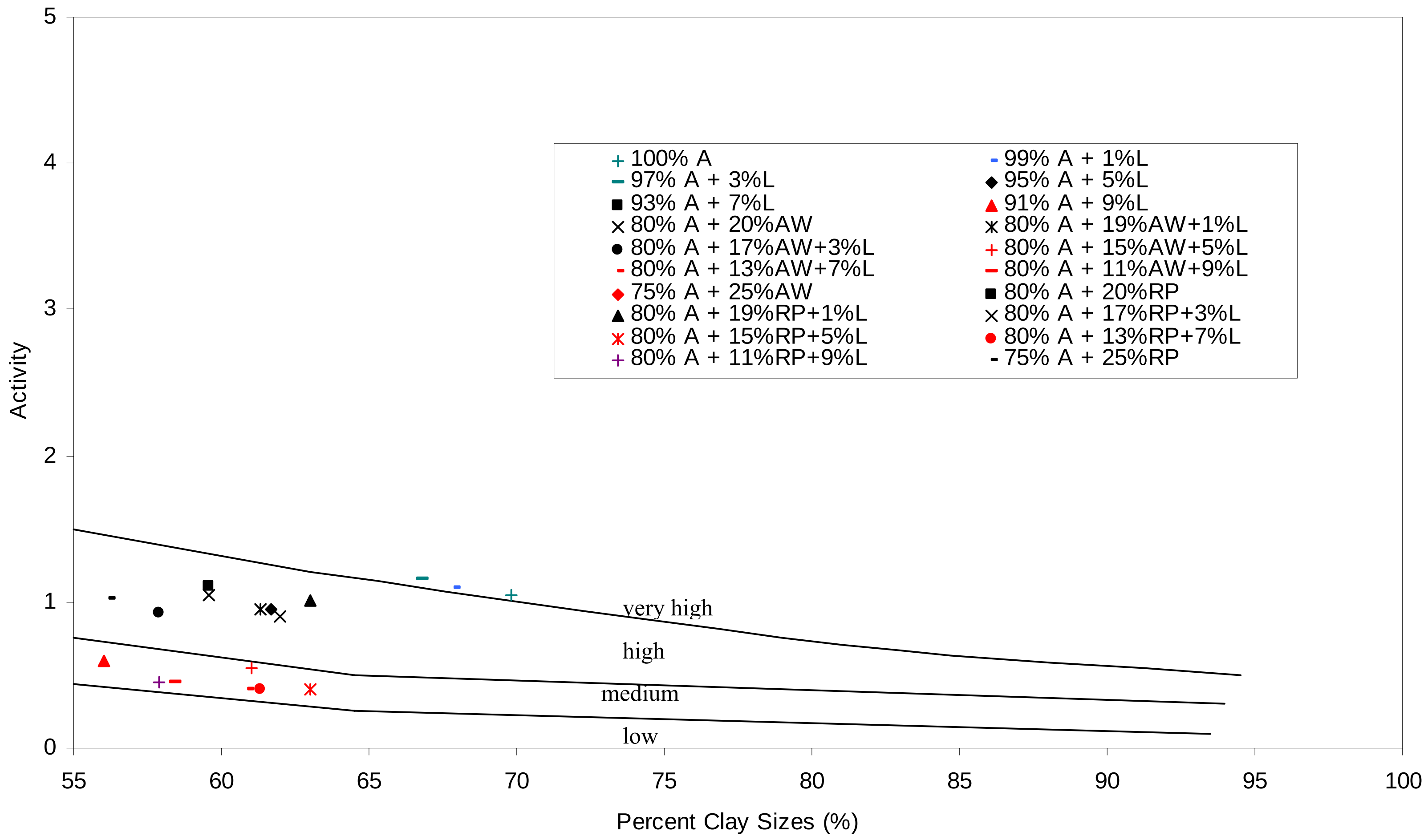

Figure 3.2 Swell Potential Classification with Clay Fraction and Activity (Seed et al. 1962)

Table 3.6 Swelling Potential of the Samples According to Seed et al. (1962)

Sample	Soil Class.	Swelling* Potential
100% A	CH	very high
99% A + 1%L	CH	very high
97% A + 3%L	CH	very high
95% A + 5%L	CH	High
93% A + 7%L	CH	High
91% A + 9%L	CH	Medium
80% A + 20%AW	CH	High
80% A + 19%AW+1%L	CH	High
80% A + 17%AW+3%L	CH	High
80% A + 15%AW+5%L	CH	Medium
80% A + 13%AW+7%L	CH	Medium
80% A + 11%AW+9%L	CH	Medium
75% A + 25%AW	CH	High
80% A + 20%RP	CH	High
80% A + 19%RP+1%L	CH	High
80% A + 17%RP+3%L	CH	High
80% A + 15%RP+5%L	CH	Medium
80% A + 13%RP+7%L	CH	Medium
80% A + 11%RP+9%L	CH	Medium
75% A + 25%RP	CH	High

*The chart of Seed et al. 1962 (Figure 3.1) was used to classify swelling potential by using clay fraction and activity values

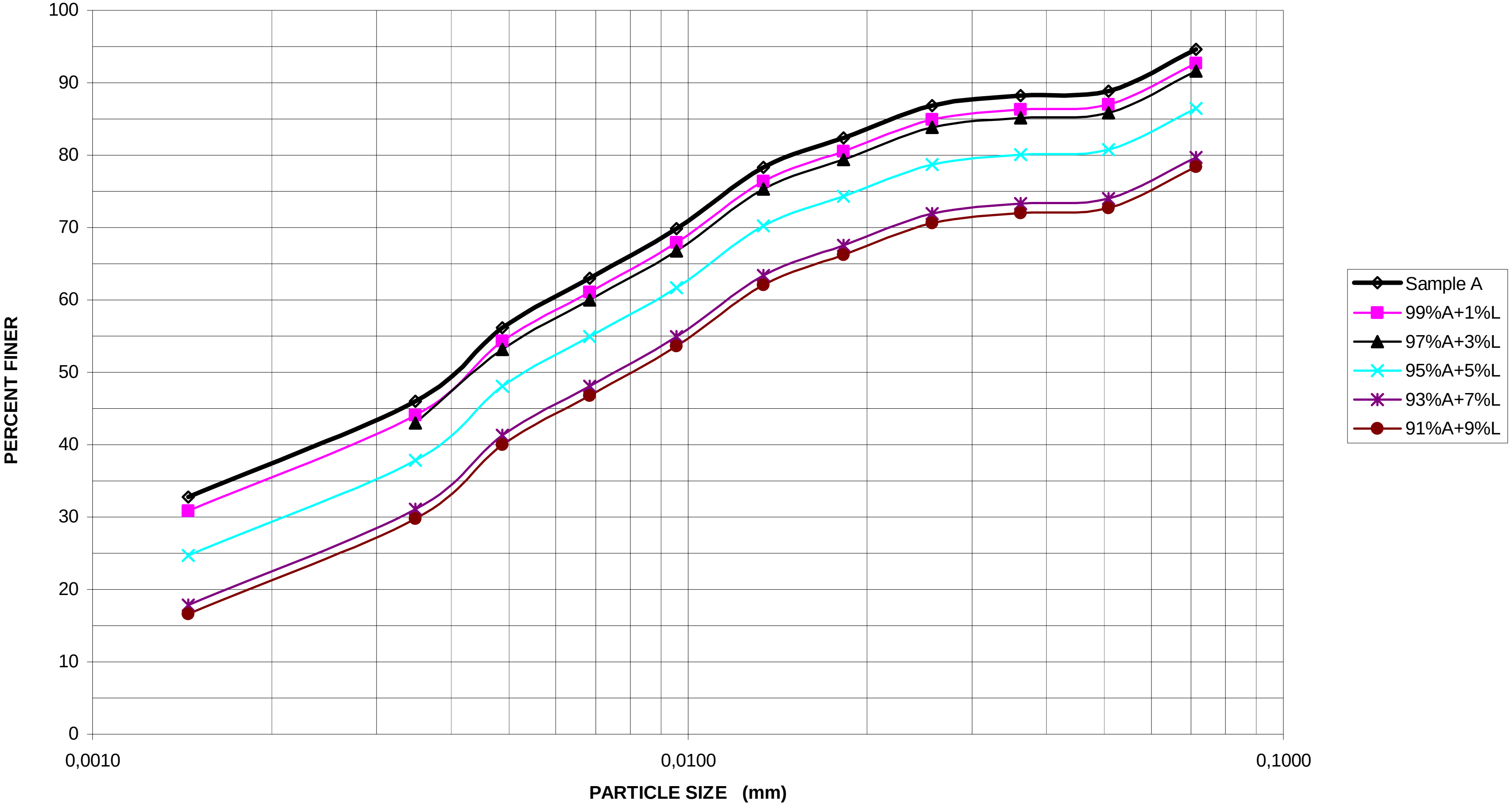

Figure 3.3 Grain size Distribution Curves of Lime Added Samples.

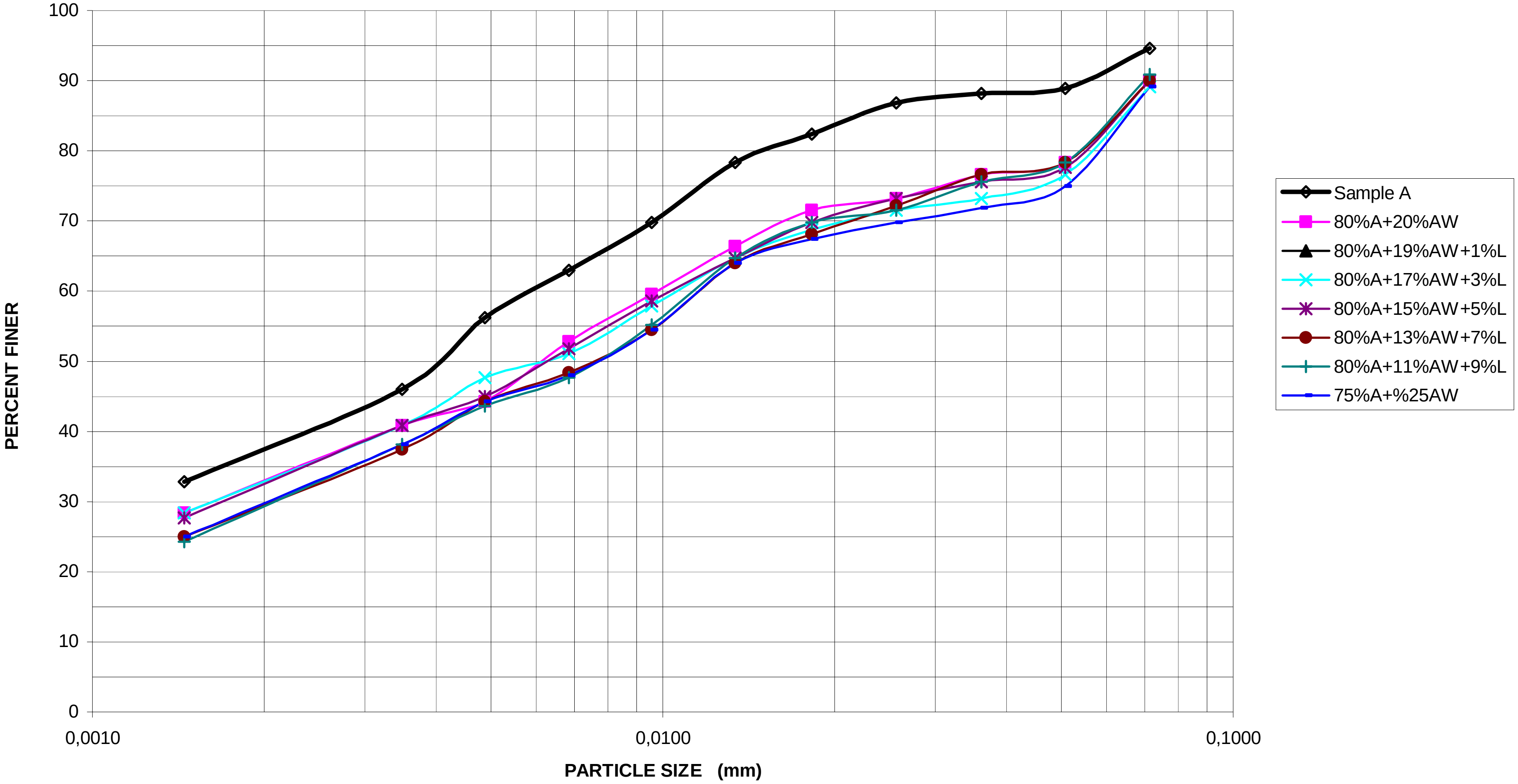

Figure 3.4 Grain size Distribution Curves of Aggregate Waste and Lime Added Samples

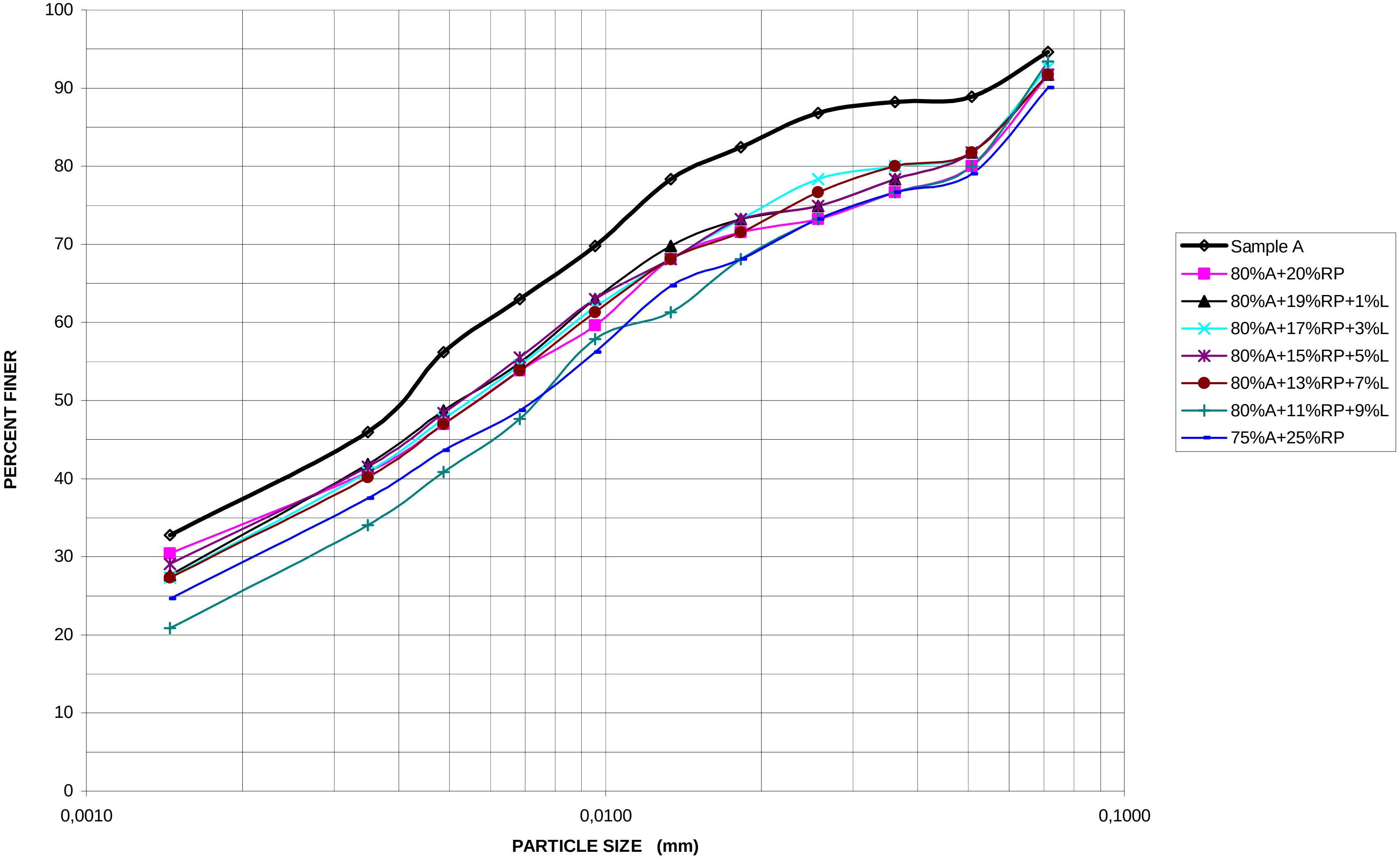

Figure 3.5 Grain size Distribution Curves of Rock Powder and Lime Added Samples.

3.5. Testing Procedure

In this study the effectiveness of the stabilizers on the swelling potential was tested by using free swell method. The samples were prepared as specified in Section 3.3. The soil sample was placed into the consolidation ring (with bulk density of 1.80 Mg/m^3) satisfying a dry density of 1.64 Mg/m^3. The soil samples were placed directly into the consolidation ring, no guide rings were used. In the preliminary studies, the samples were compacted in the guide rings satisfying the specified dry density and later transferred into the consolidation rings. But it was observed that during this transference, the specimens were terribly disturbed. Sometimes cracks were formed on the surfaces of the specimens or some pieces were dropped out from the top and bottom of the specimen. Consequently, in order to avoid further spoilage of specimen it was decided that the samples had to be compacted directly into the consolidation rings.

3.5.1. Free Swell Method

The preparation of the samples was described in Section 3.3. The free swell test is performed by taking a specimen in a consolidation ring that is at least 6.35 cm diameter. The sample is confined in the consolidation ring, which is placed in the oedometer under a small surcharge. Water is then added to the sample and allowed to swell freely. As the sample swells the deflections of the dial gauge is recorded. At some point the sample has no further tendency to swell and maximum deflection recorded is used for the calculation of the free swell. The percent of free swell can be expressed as;

$$\text{Free Swell} = \Delta H/H \times 100$$

Where Δ H= Change in initial height (H) of the specimen

H= Initial height of the specimen

The procedure of the tests was as follows; at first the specimens was compacted in the consolidation ring and then the ring was placed into the oedometer after placing dry filter papers on top and bottom of the sample. (Figure 3.6) The consolidation ring was assembled in the oedometer and air-dry porous stone was placed on top of the sample (there had already been one at the bottom of the oedometer.) After the oedometer was mounted on the loading device, the deflection dial was adjusted to zero reading. The sample was inundated by providing water through standpipes and pouring water directly from the top of the oedometer. Swelling of the sample started at the moment the sample was started to be inundated. The tests were finished when there was no change on the dial gauge.

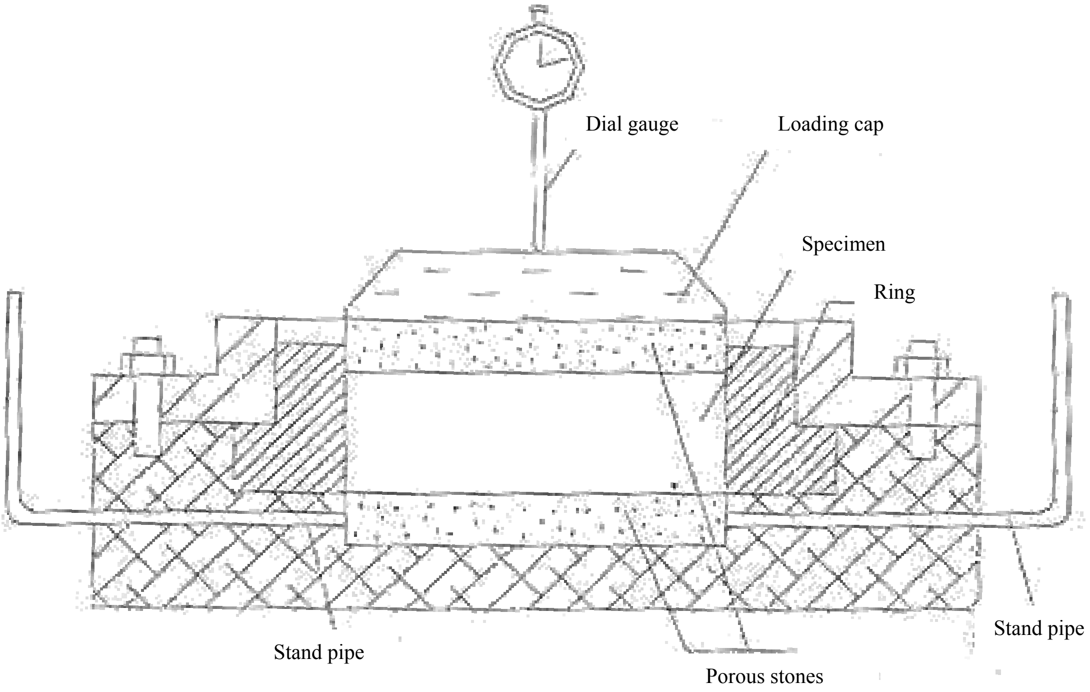

Figure 3.6 Free Swell Test Apparatus.

Free swell tests were performed as described above on samples with an initial water content of 10% and having a dry density value of 1.64 g/cm^3 under a small surcharge of about 1.94 kPa. All mixtures which were sieved through No.30 sieve were prepared and compacted in humidity room in order to maintain its initial water content constant.

3.6. Experimental Program

By means of the preliminary tests, the tests were decided to be performed on twenty samples which were shown in Table 3.4.

The experimental study had four phases;

1. Hydrometer tests, specific gravity tests, Atterberg limit tests, were applied to Sample A and specimens listed in Table 3.4.

2. Swelling characteristics of the samples were determined without curing.

3. Swelling characteristics of the samples were determined after 7 days curing.

4. Swelling characteristics of the samples were determined after 28 days curing.

3.7. Test Results

The results of tests showing liquid limit, plastic limit, shrinkage limit and plasticity index of the samples are shown on Figures 3.7, 3.8, 3.9, 3.10 respectively for lime, aggregate waste and rock powder samples.

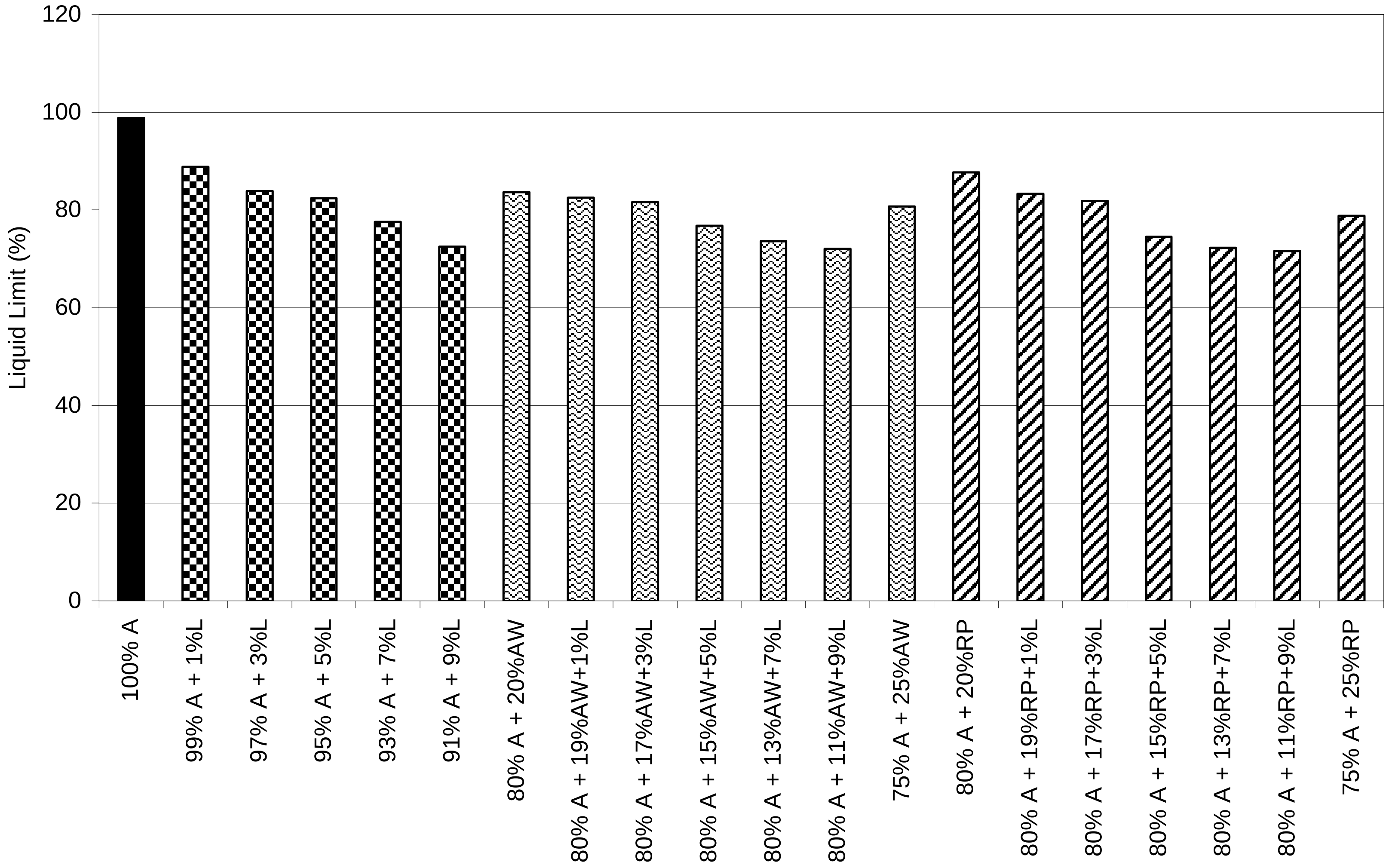

Figure 3.7 Effects of Lime, Aggregate Waste and Rock Powder on the Liquid Limit Value of Sample A

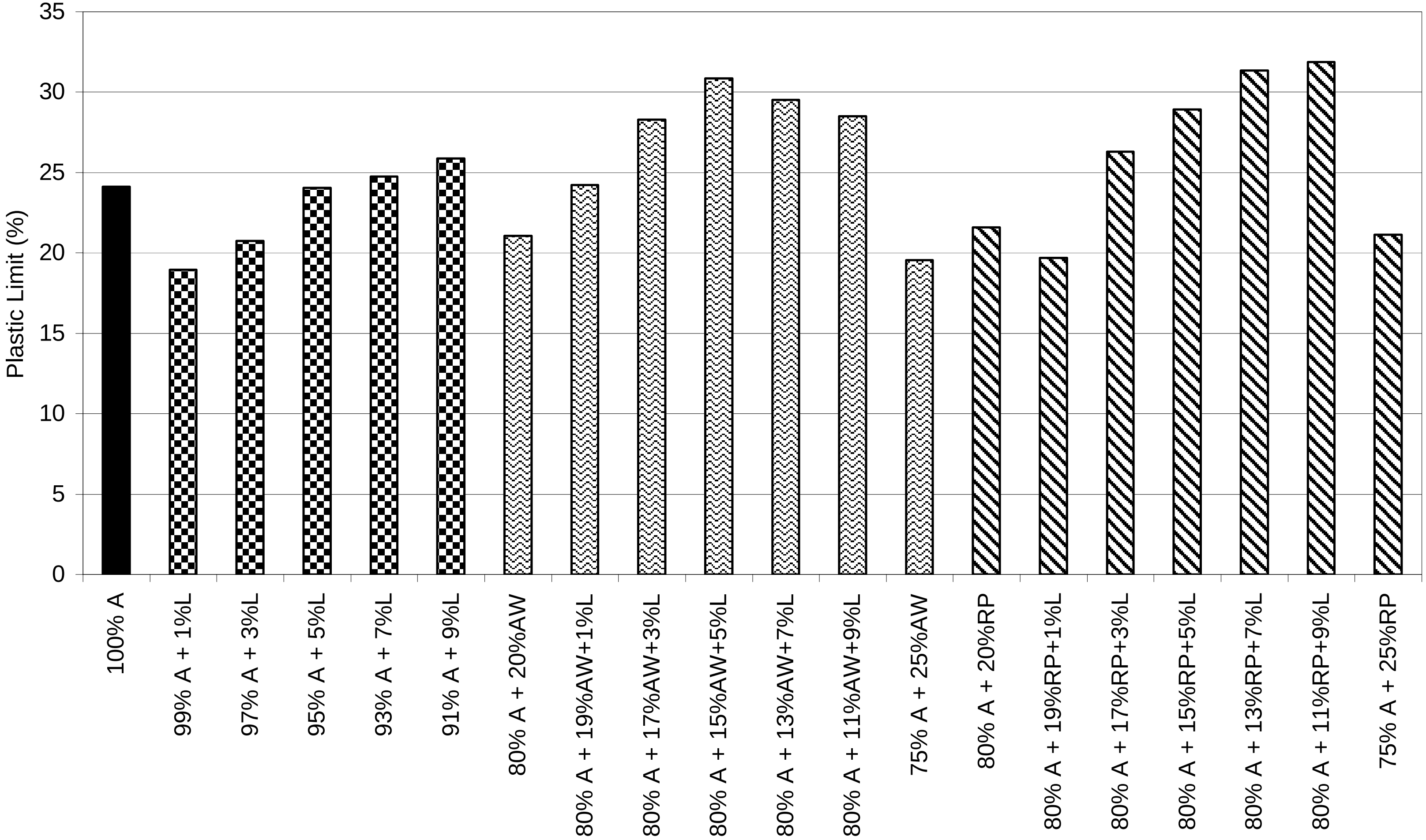

Figure 3.8 Effects of Lime, Aggregate Waste and Rock Powder on the Plastic Limit Value of Sample A.

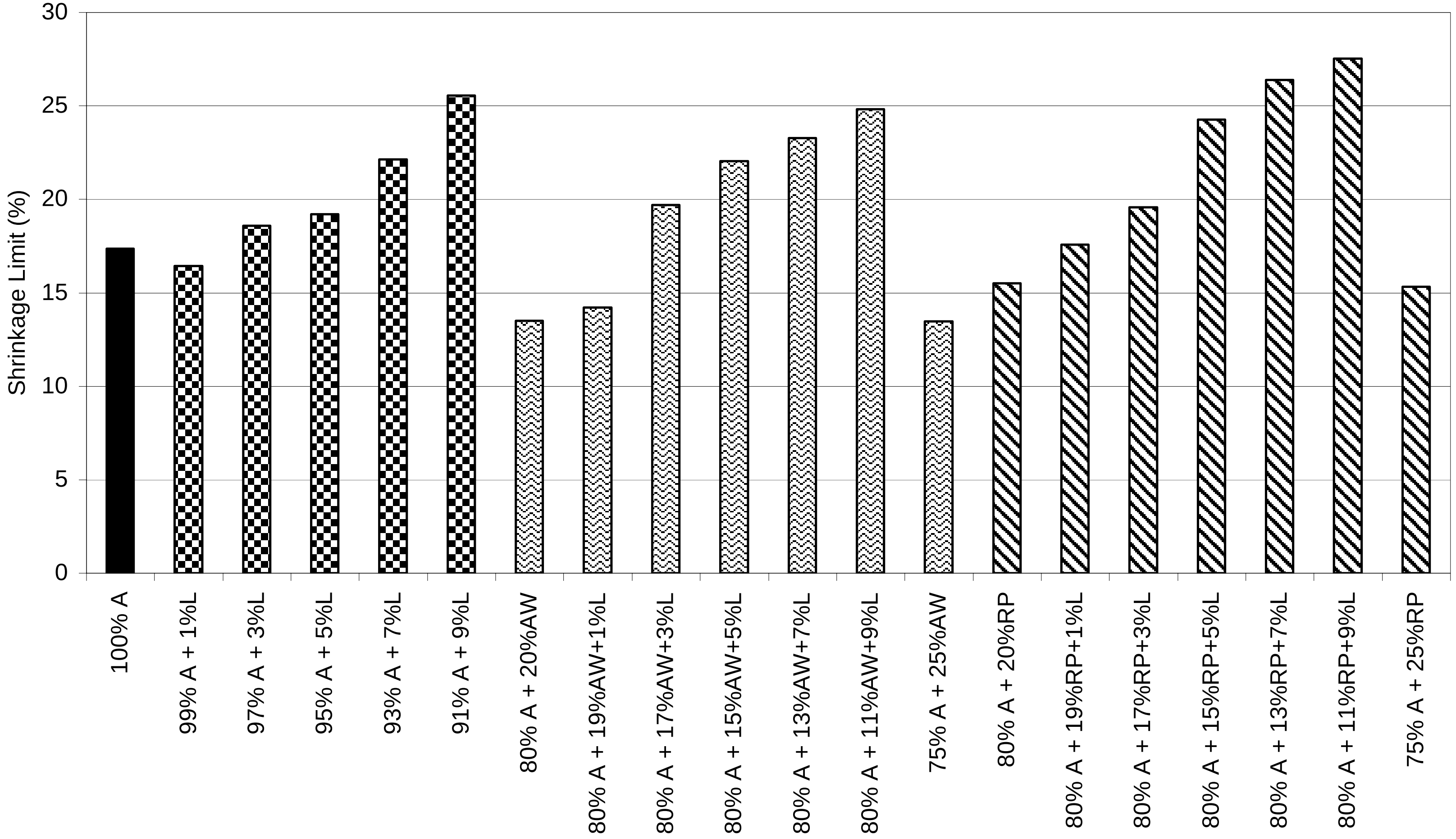

Figure 3.9 Effects of Lime, Aggregate Waste and Rock Powder on the Shrinkage Limit Value of Sample A.

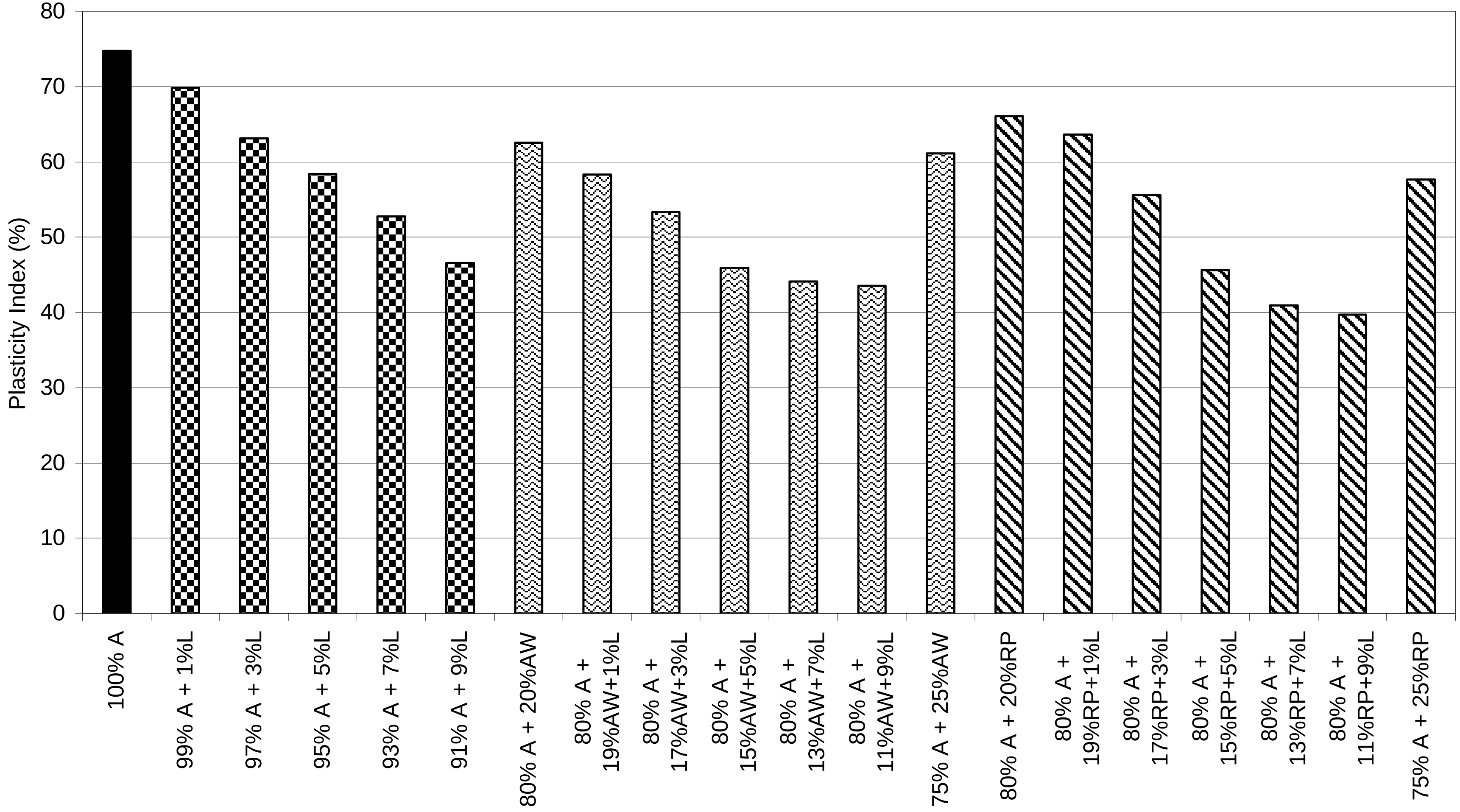

Figure 3.10 Effects of Lime, Aggregate Waste and Rock Powder on the Plasticity Index Value of Sample A.

Figure 3.11 shows the swell percent values of Sample A during free swell test for 0 days curing. Sample A has a very typical swell percent versus time graph. As it is explained before, the dial readings were recorded and ΔH of each reading was calculated by subtracting the initial reading value from the value read at that time and ΔH/H versus time was plotted

The effects of stabilizer percentage on swell percentages are shown on Table 3.7 and Figure 3.12 for 0 days curing, on Figure 3.13 for 7 days curing and on Figure 3.14 for 28 day curing. Figure 3.15 is plotted to see the effect of curing on swell percentages.

The effects of stabilizer percentage on the time necessary to reach fifty percent swell (t_{50}) are shown on Figure 3.16 for 0 days curing, on Figure 3.17 for 7 days curing and on Figure 3.18 for 28 day curing. Figure 3.19 is plotted to see the effect of curing on rate of swell.

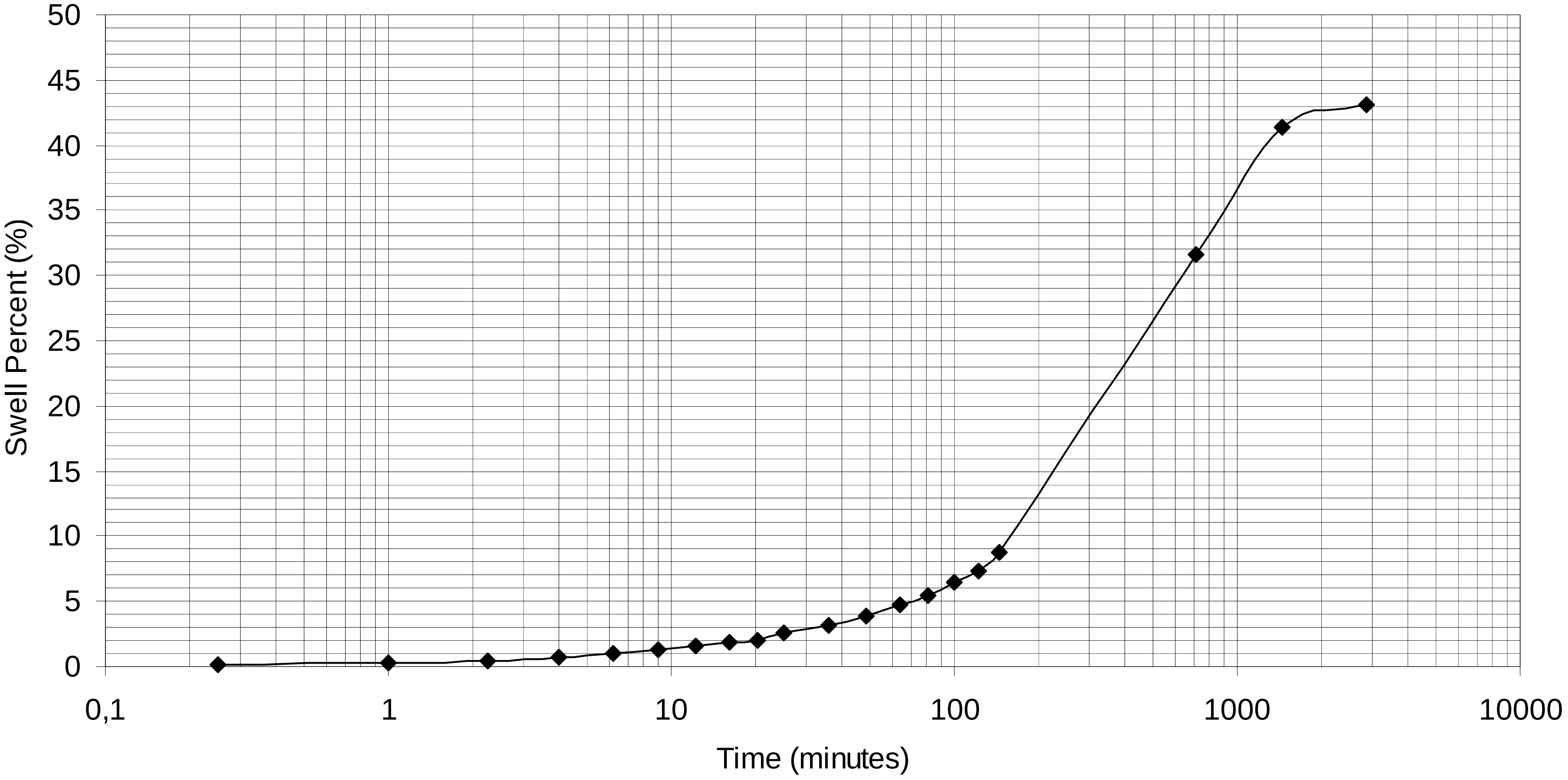

Figure 3.11 Swell Percentage versus Time Relationship for Sample A.

Table 3.7 Swell Percentages of the Samples.

Sample	Swell Percent (%) without curing	Swell Percent (%) 7 days curing	Swell Percent (%) 28 days curing
100% A	43.05	40.42	40.16
99% A + 1%L	39.63	38.26	37.84
97% A + 3%L	21.74	20.84	20.32
95% A + 5%L	20.11	19.89	19.78
93% A + 7%L	19.55	19.45	19.01
91% A + 9%L	18.99	18.75	18.52
80% A + 20%AW	28.79	27.42	27.21
80% A + 19%AW+1%L	27.95	27.37	27.16
80% A + 17%AW+3%L	12.26	12.16	11.84
80% A + 15%AW+5%L	12.21	12.11	11.74
80% A + 13%AW+7%L	11.84	11.74	11.68
80% A + 11%AW+9%L	11.63	11.05	11.00
75% A + 25%AW	25.56	24.78	24.01
80% A + 20%RP	31.11	31.00	30.89
80% A + 19%RP+1%L	24.58	24.00	23.47
80% A + 17%RP+3%L	12.89	12.79	12.68
80% A + 15%RP+5%L	11.63	10.26	10.05
80% A + 13%RP+7%L	10.89	10.16	9.84
80% A + 11%RP+9%L	10.68	9.95	9.11
75% A + 25%RP	22.02	21.28	20.89

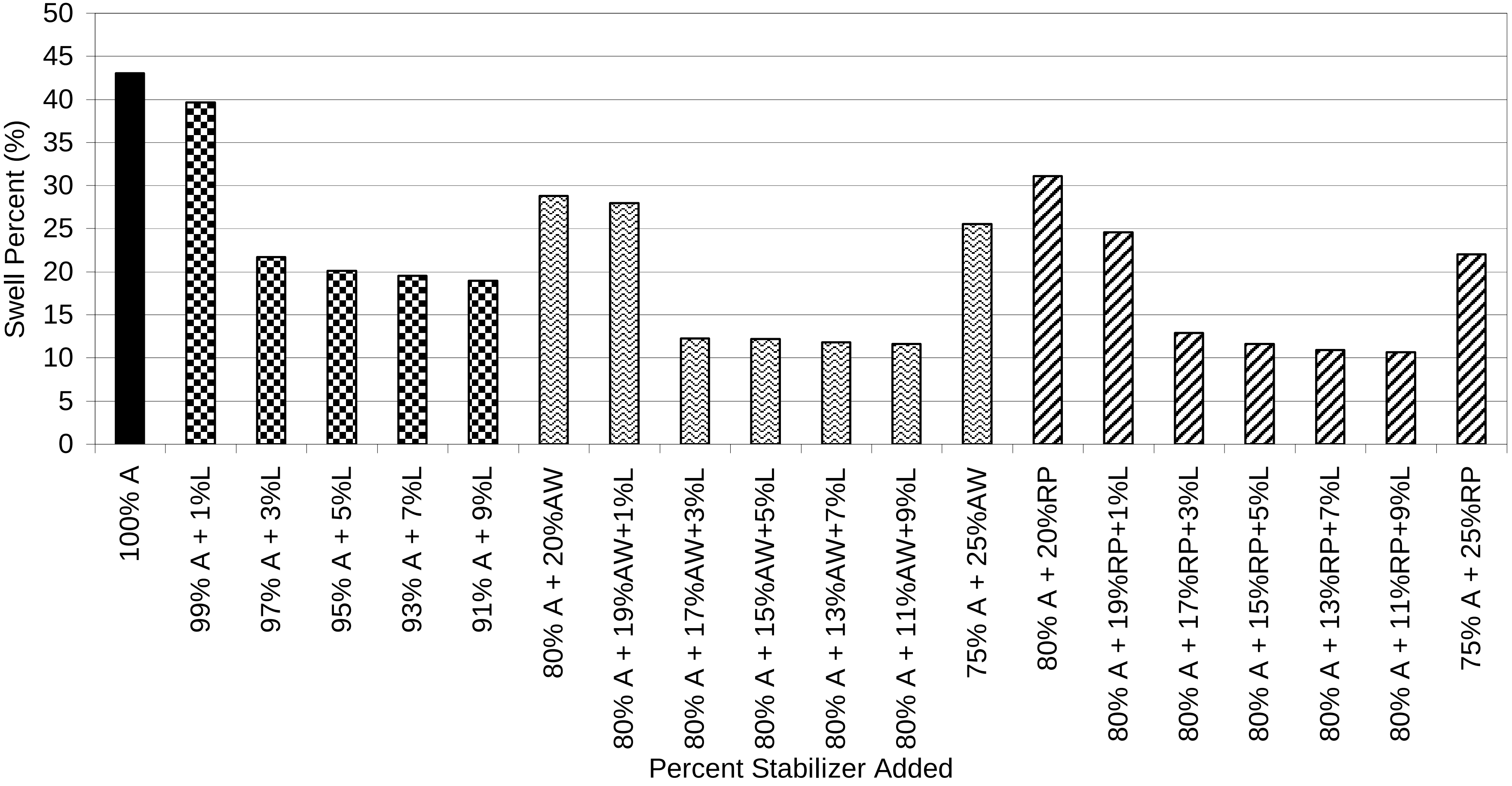

Figure 3.12 Variations of Swell Percentages, according to the Types of Materials Added to Sample A for 0 Days Curing.

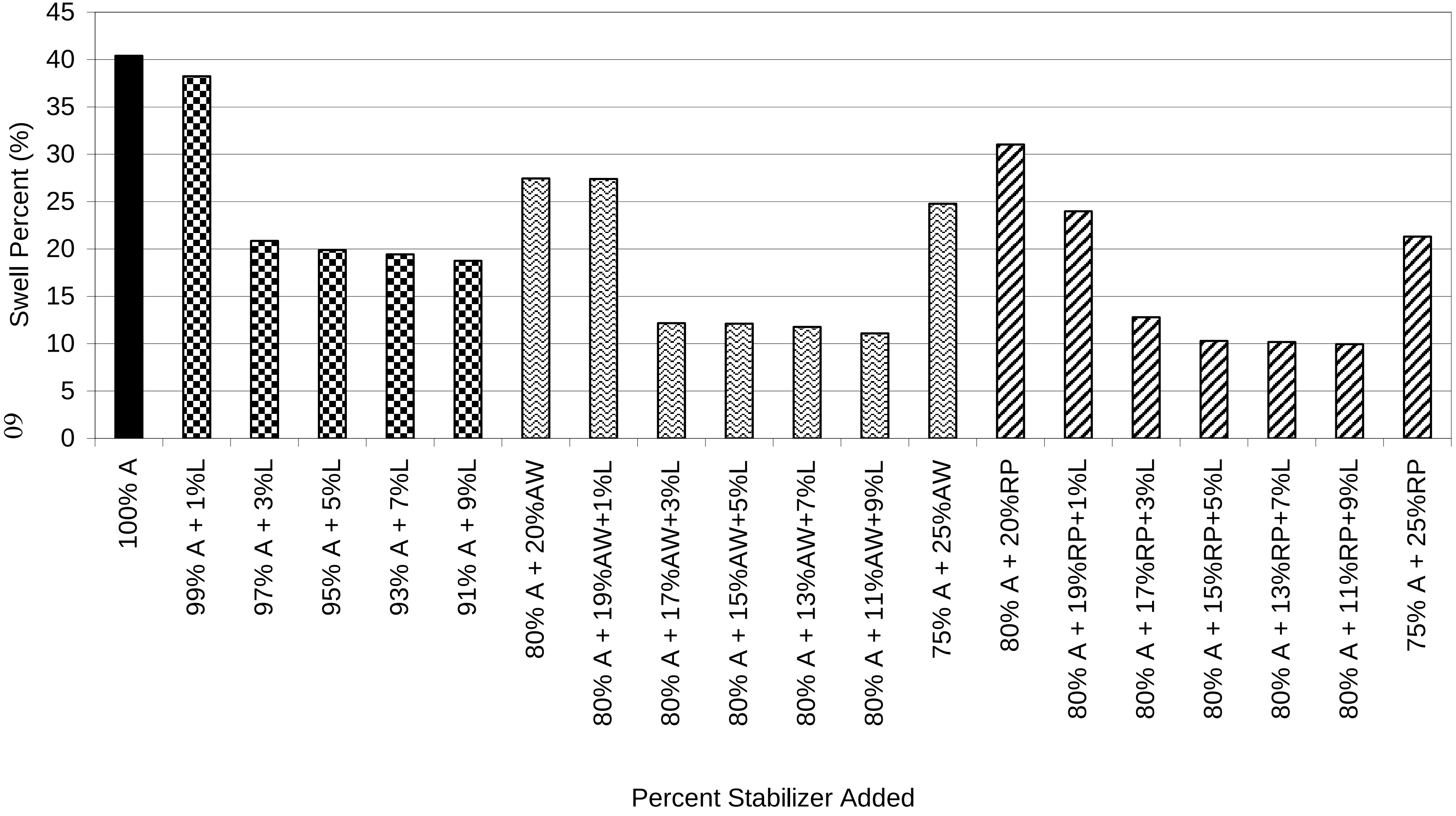

Figure 3.13 Variations of Swell Percentages, according to the Types of Materials Added to Sample A for 7 Days Curing.

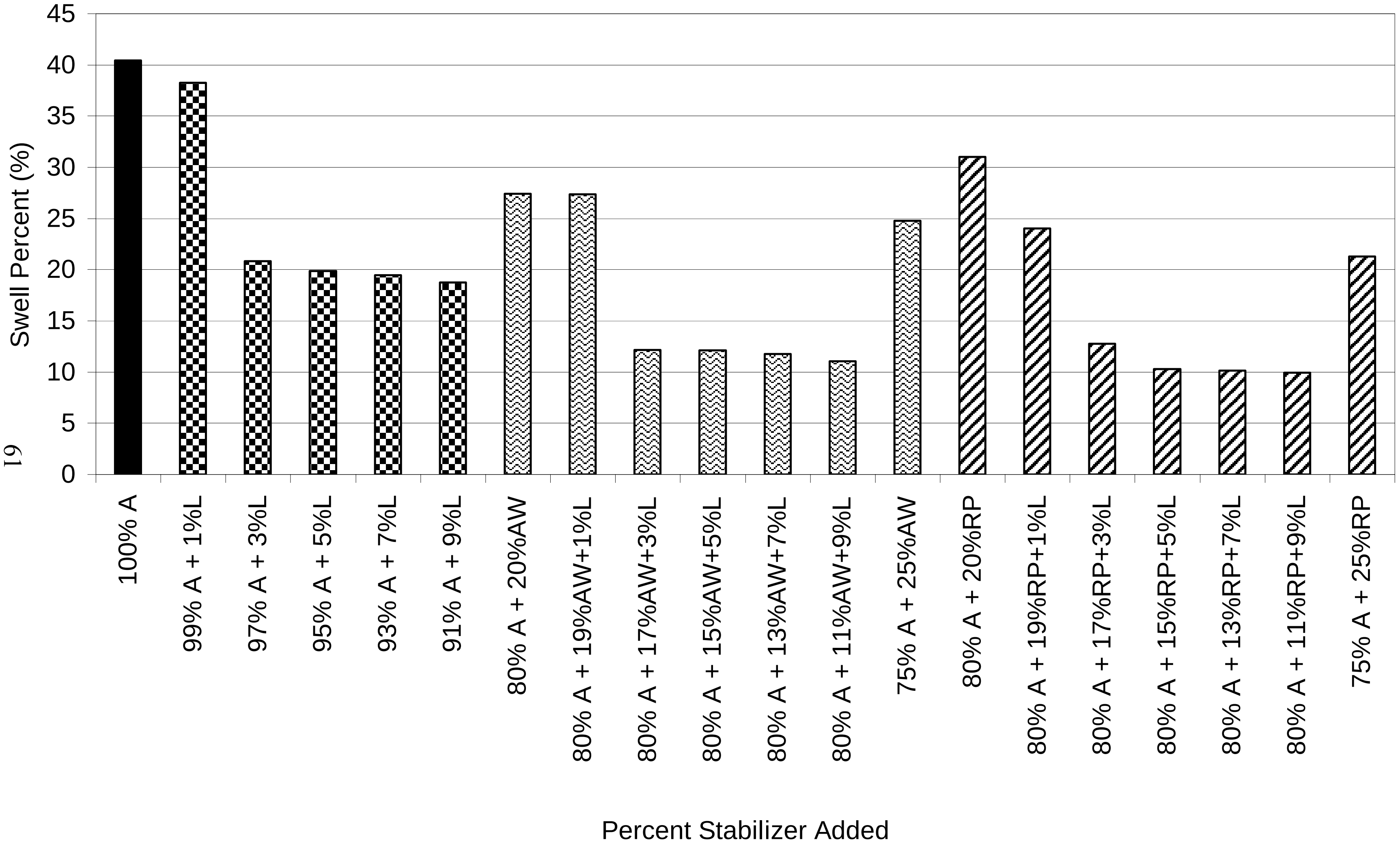

Figure 3.14 Variations of Swell Percentages, according to the Types of Materials Added to Sample A for 28 Days Curing.

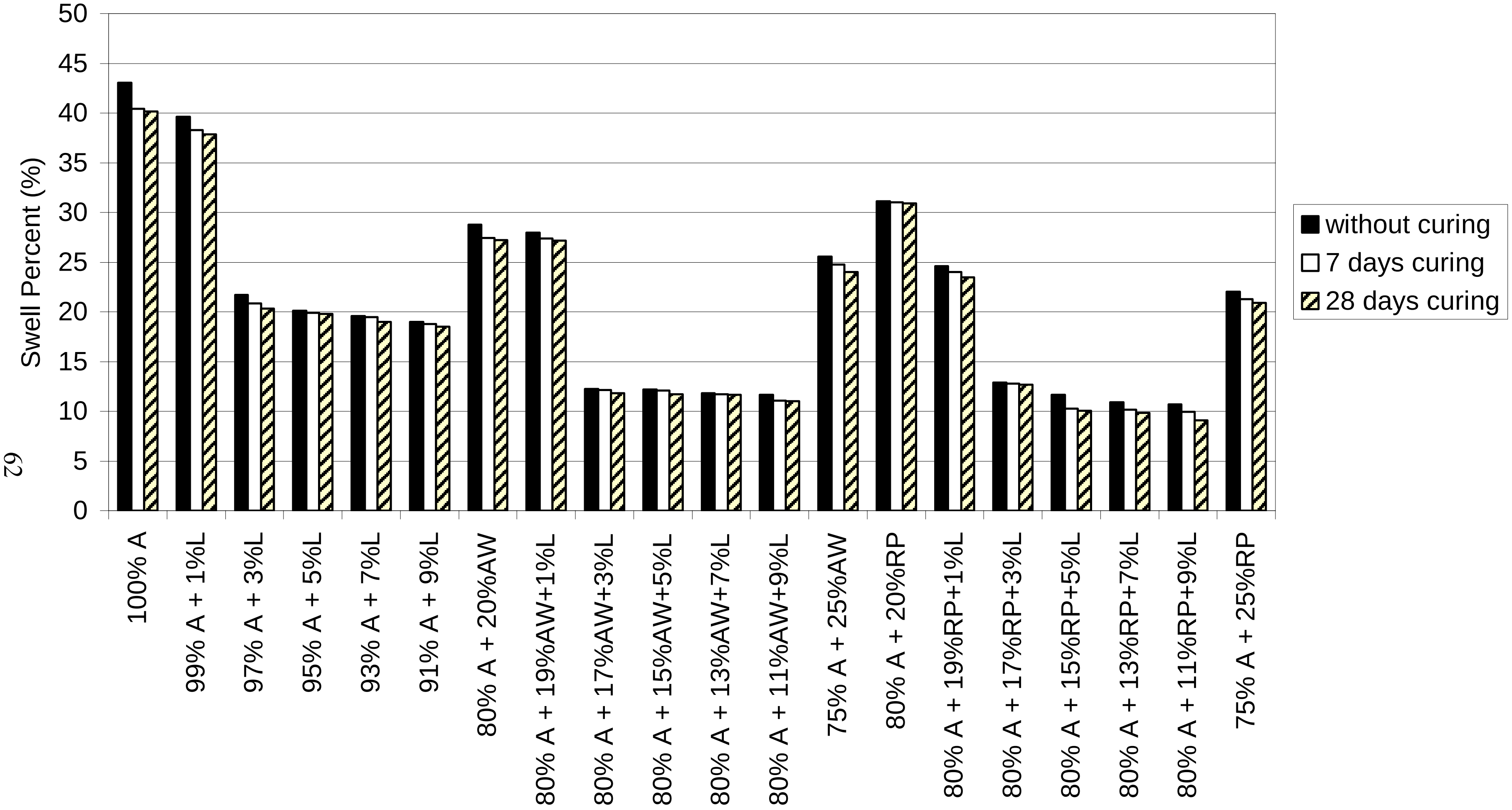

Figure 3.15 The Effect of Curing on Swell Percentages.

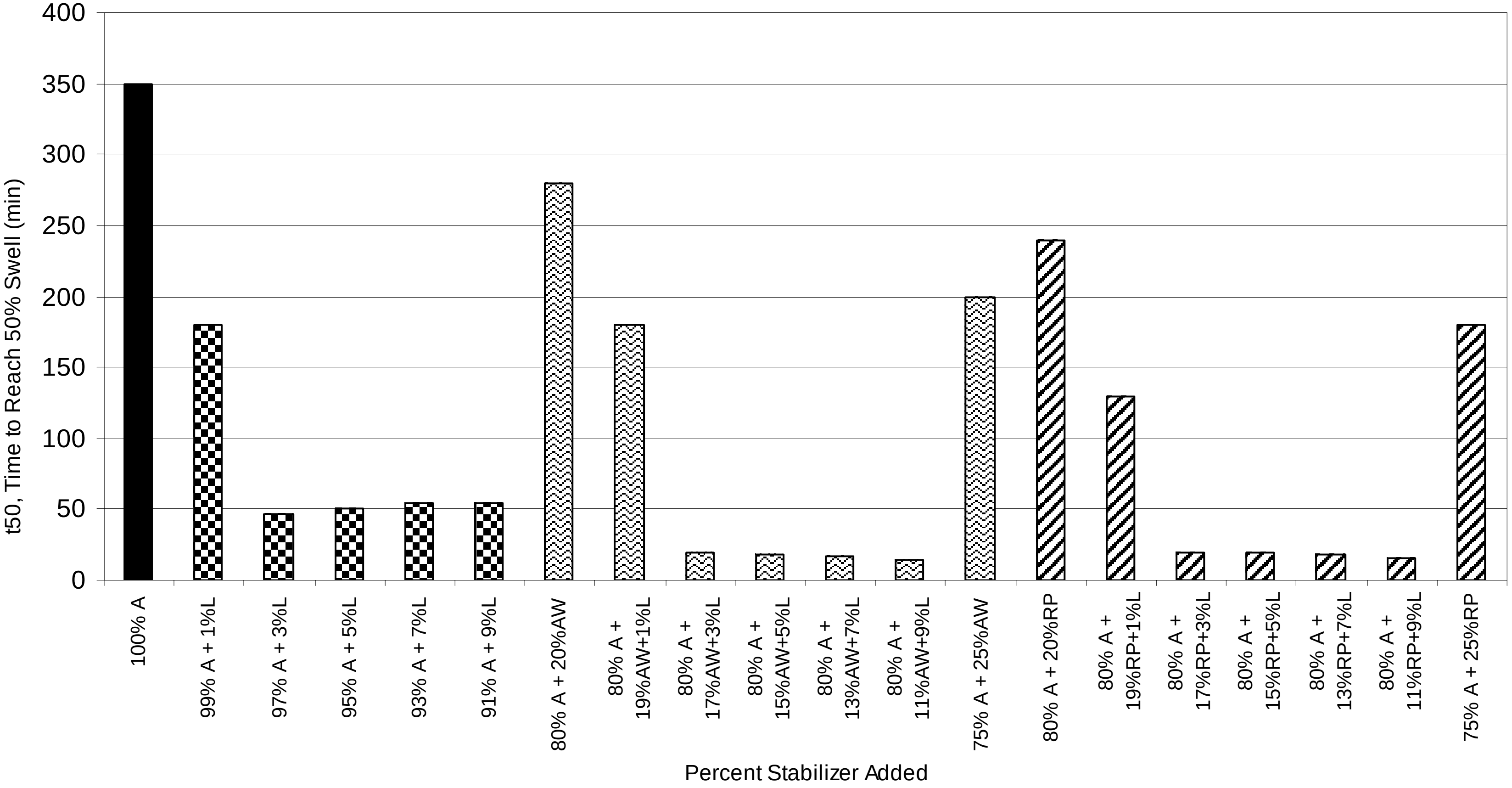

Figure 3.16 Variation of t_{50} with Type and Amount of Stabilizer Added.

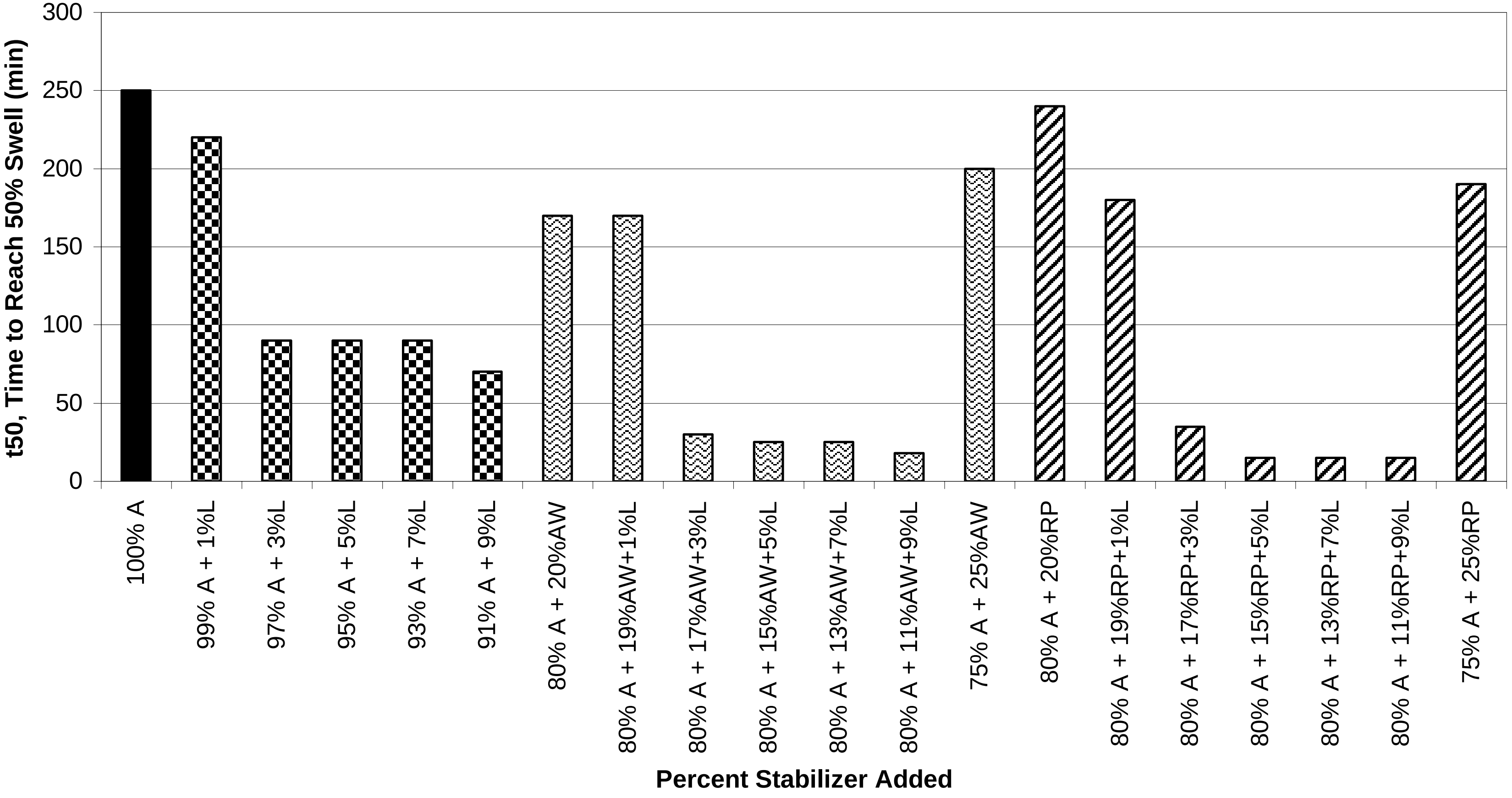

Figure 3.17 Variation of t_{50} with Type and Amount of Stabilizer Added after 7 Days Curing.

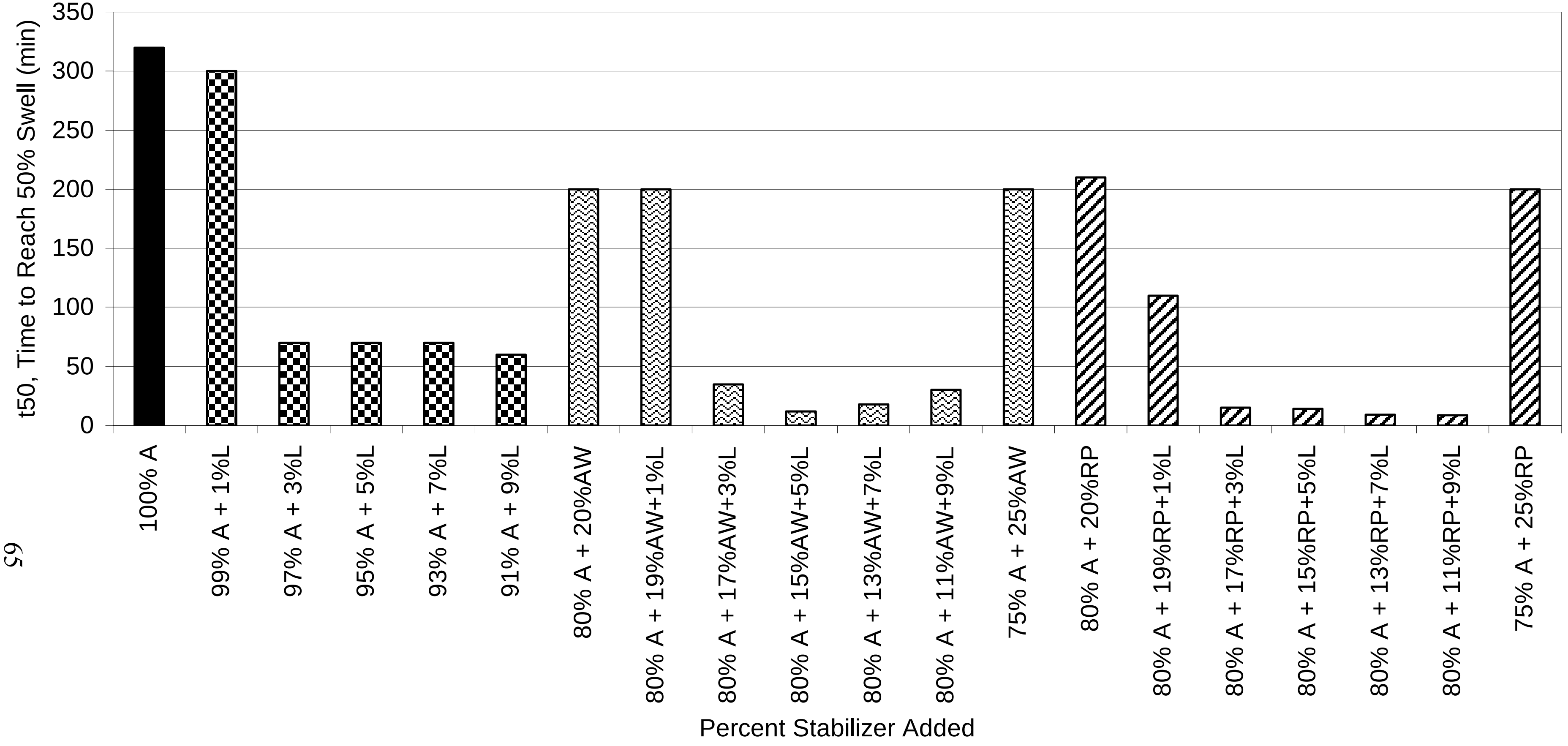

Figure 3.18 Variation of t_{50} with Type and Amount of Stabilizer Added After 28 Days Curing.

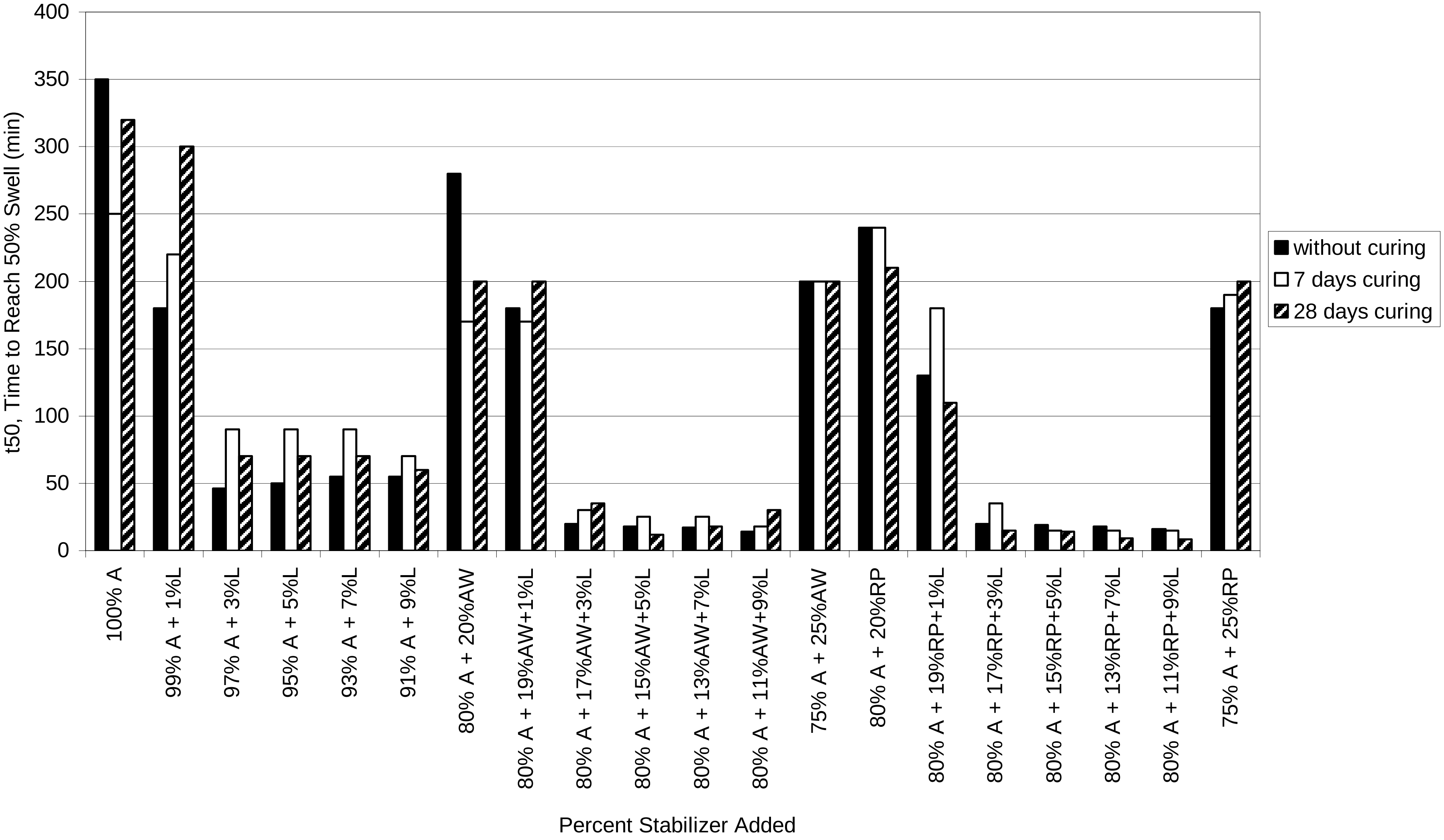

Figure 3.19 Effect of curing time on the rate of swell.

CHAPTER 4

DISCUSSION OF TEST RESULTS

4.1. Effects of Lime, Aggregate Waste, Rock Powder Addition on Grain Size Distribution of the Expansive Soil

Addition of all materials shifted the grain size distribution curve of Sample A (Figures 3.3, 3.4, 3.5).

For the samples including only lime and Sample A shifting to the coarser side was started after 5% lime was added to Sample A and especially when 7% lime was added shifting was noticeable. Addition of 1% and 3% lime to the soil did not make a significant change (Figure 3.3).

The grain size distributions curve of Sample A shifted significantly to the coarser side when aggregate waste or rock powder was added. (Figures 3.4 and 3.5) This shifting may occur either from pozzolonic reactions or from the addition of silt sized particles to Sample A. To distinguish the reason of this shifting for the samples containing aggregate waste, Figure C1 (Appendix C) was plotted. On this figure, particle size distribution curves of Sample A, 100% AW, 25%AW, and Calculated 25%AW (25%AW and 75% Sample A were considered on the mass basis) are shown. Calculated 25%AW curve is on the left side of 25%AW curve, but these curves are close to each other. This shows that the shifting of the particle size distribution curve of Sample A due to the addition of aggregate waste, is mainly depend on the addition of silt sized particles, but on the other hand this does not mean that there is no effect of pozzolanic reactions, there are also some pozzolanic reactions which can cause the flocculation of clay particles. The same procedure was applied to the samples containing rock powder materials (Appendix C, Figure C2) and nearly the same results were taken. Consequently, as the amount of aggregate waste or rock powder materials increases, the soil becomes more granular and this is mainly because of the addition of silt sized particles to the soil.

4.2. Effects of Lime, Aggregate Waste, Rock Powder Addition on Liquid Limit of the Expansive Soil (Table 4.1)

There had been significant decrease on liquid limit values of the samples with the increasing amount of stabilizer content. Liquid limit values decreased as the amount of added lime increases. Addition of 1% lime to the soil gave a reduction of 10.19%. The maximum reduction was observed when 9% lime was added to the sample which is 26.7%.

When stabilizers and lime were added to the samples together, the reductions were greater. Addition of 11% aggregate waste and 9% lime diminished the liquid value of Sample A by 27.15% and the addition of 11% rock powder and 9% lime diminished the liquid value by 27.59%. When only aggregate waste or rock powder was added to the samples the liquid limit of Sample A was decreased from 98.84%

to 80.89% for 25% aggregate waste, to 78.77% for 25% rock powder. This shows 18.36% reduction for 25%AW and 20.31% reduction for 25%RP.

4.3. Effects of Lime, Aggregate Waste, Rock Powder Addition on Plastic Limit of the Expansive Soil (Table 3.5 and 4.1)

Plastic limits for lime added samples vary in a narrow range. Addition of 1 to 5 % lime decreased, 7% and 9% lime increased the plastic limit of Sample A.

Addition of 20 % and 25% aggregate waste decreased the plastic limit of Sample A. Addition of 17% AW and 3%L increased the plastic limit of Sample A from 24.12% up to 28.30%. Nearly the same results were shown for rock powder added samples. The only difference is that when 19%AW and 1%L was added plastic limit of Sample A did not changed but when 19%RP and 1%L was added plastic limit of Sample A decreased.

4.4. Effects of Lime, Aggregate Waste, Rock Powder Addition on Shrinkage Limit of the Expansive Soil (Table 3.5)

Shrinkage limit values of lime, aggregate waste and rock powder added samples were first showed a decrease and than an increase on the value of shrinkage limit of Sample A.

Table 4.1 Percent Changes in Liquid Limit, Plastic Limit, Shrinkage Limit, Plasticity Index Values and Activity

Sample	% Change in LL	% Change in PL	% Change in PI	% Change in SL	% Change in Activity
100% A	0.00	0.00	0.00	0.00	0.00
99% A + 1%L	-10.19	-21.48	-6.54	-5.41	-3.92
97% A + 3%L	-15.13	-14.01	-15.48	6.97	-11.68
95% A + 5%L	-16.58	-0.29	-21.84	10.65	-11.57
93% A + 7%L	-21.57	2.61	-29.38	27.52	-10.20
91% A + 9%L	-26.70	7.21	-37.65	47.15	-18.79
80% A + 20%AW	-15.37	-12.60	-16.26	-22.28	-1.90
80% A + 19%AW+1%L	-16.53	0.41	-22.00	-18.19	-11.17
80% A + 17%AW+3%L	-17.44	17.31	-28.66	13.36	-13.97
80% A + 15%AW+5%L	-22.33	28.01	-38.57	26.83	-29.73
80% A + 13%AW+7%L	-25.50	22.43	-40.97	33.94	-32.32
80% A + 11%AW+9%L	-27.15	18.16	-41.78	42.89	-30.42
75% A + 25%AW	-18.36	-18.91	-18.19	-22.39	4.81
80% A + 20%RP	-11.28	-10.45	-11.55	-10.71	3.62
80% A + 19%RP+1%L	-15.67	-18.33	-14.82	1.15	-5.61
80% A + 17%RP+3%L	-17.19	9.04	-25.66	12.78	-16.26
80% A + 15%RP+5%L	-24.59	19.94	-38.96	39.61	-32.36
80% A + 13%RP+7%L	-26.85	29.93	-45.18	51.93	-37.57
80% A + 11%RP+9%L	-27.59	32.17	-46.88	58.43	-35.94
75% A + 25%RP	-20.31	-12.35	-22.87	-11.74	-4.18

* ‘+’ shows the increase in values and ‘-’ shows the decrease in values.

For 1% lime added samples there was a reduction of 5.41% on shrinkage limit of Sample A. But on the other hand, 3% to 9% lime added samples increased the value up from 17.37% to18.58% for 3%L and 22.56% for 9%L.

4.5. **Effects of Lime, Aggregate Waste, Rock Powder Addition on Plasticity Index of the Expansive Soil** (Table 4.1)

Addition of the stabilizers decreased the plasticity index of Sample A significantly.

Maximum amount of lime (9%) reduced the plasticity index of Sample A by 37.65%.

Addition of 11%AW and 9%L and addition of 11%RP and 9%L decreased the plastic limit value from 74.72% to 43.51% and 39.69%. This shows that rock powder is more effective additive to decrease the plasticity index values.

4.6. Effects of Lime, Aggregate Waste, Rock Powder Addition on Specific Gravity of the Expansive Soil

The addition of lime increased the specific gravity but the addition of aggregate waste and rock powder with lime decreased the specific gravity value of Sample A. (Table 3.1)

4.7. Effects of Lime, Aggregate Waste, Rock Powder Addition on Activity of the Expansive Soil

Activity value of Sample A is generally decreased as percent stabilizer increased. (Table 3.1)

4.8. Effects of Lime, Aggregate Waste, Rock Powder Addition on Swelling Potential of the Expansive Soil (According to Seed et al. 1962)

Sample A was categorized as a soil which has a very high swelling potential by means of the 'Swell Potential Classification' chart of Seed et al. 1962. (Figure 3.2) Addition of all of the stabilizers caused considerable reduction in the swelling potential (Table 3.6). It is also consistent that swelling potential decreases with the amount of stabilizer.

4.9. Effects of Lime, Aggregate Waste, Rock Powder Addition on Swell Percentage of the Expansive Soil

All the stabilizers reduced the swelling percentage of Sample A.

Lime added samples reduced the swelling percentage to 18.99% from 43.05%. But when the two stabilizers were used the reduction was more.(Table 3.7) Adding 11%AW and 9%L reduced the swell to 11.63% and adding 11%RP and 9%L reduced the swelling potential to10.68%.

To understand the effect of stabilizers on the swelling potential we can look at the results of 25 % AW and 25% RP. Swell percentage of Sample A was reduced when the stabilizer content increased. 25% AW showed a swelling percentage of 25.56% and 25% RP showed a swelling percentage of 22.02%. This shows that as a stabilizing material, rock powder is more effective than aggregate waste. These reductions are mainly due to the chemical composition of the materials. The chemicals with in the materials go into reactions with the chemicals in Kaolinite and Bentonite minerals. Especially CaO % of each additive is important in these reactions. After these reactions soil hardens and gains strength and the swelling potential decreases.

Consequently, as a second material, aggregate waste and rock powder showed a significant change in swelling percentage of Sample A when used with lime. On the other hand when only aggregate waste and rock powder were added to the soil without lime, there was a reduction in the swelling potential, either.

4.10. Effects of Lime, Aggregate Waste, Rock Powder Addition on Rate of Swell of the Expansive Soil

Adding lime, aggregate waste and rock powder definitely affects the time to reach fifty percent swell and accordingly the rate of swell.

Lime is very effective in reducing the time necessary to reach fifty percent swell. Especially when 3% lime is added to the soil it is conceivable to visualize the increase in rate of swell. (Figure 3.16) Also the other stabilizers increased rate of swell. But the most effective stabilizer was lime to reduce the time necessary to reach fifty percent swell, in comparison to aggregate waste and rock powder.

4.11. Effect of Curing on Swell Percentage

Generally all samples showed a small reduction in swelling potential after curing. $\Delta H/H$ value of each sample is slightly higher for 0 days curing and slightly lower for 28 days curing according to Figure 3.15. It can be said that the curing does not have a significant effect on swelling potential. (Table 4.2)

For lime addition of 3% or higher, percent change in swell percentage gets an almost constant value, for without curing and for 7 days and 28 days curing conditions (Table 4.2).

4.12. Effect of Curing on Rate of Swell

Remembering the rate of swell is represented by t_{50}, it is noticeable that the rate of swell changes with increasing amount of stabilizers and curing time. (Figure 3.19 and Table 4.2)

For lime addition of 3% or higher, percent change on rate of swell gets an almost constant value, for without curing and for 7 days and 28 days curing conditions (Table 4.2)

Table 4.2 Percent Changes in Swell Percent and Rate of Swell (with no curing, 7 days curing and 28 days curing)

Sample	**% Change in $\Delta H/H$ with no curing**	**% Change in $\Delta H/H$ 7 days curing**	**% Change in $\Delta H/H$ 28 days curing**	**% Change in t_{50} with no curing**	**% Change in t_{50} 7 days curing**	**% Change in t_{50} 28 days curing**
100% A	0.00	0.00	0.00	0.00	0.00	0.00
99% A + 1%L	-7.95	-11.12	-12.10	-48.57	-37.14	-14.29
97% A + 3%L	-49.51	-51.59	-52.81	-86.86	-74.29	-80.00
95% A + 5%L	-53.29	-53.80	-54.06	-85.71	-74.29	-80.00
93% A + 7%L	-54.59	-54.82	-55.84	-84.29	-74.29	-80.00
91% A + 9%L	-55.89	-56.45	-56.98	-84.29	-80.00	-82.86
80% A + 20%AW	-33.13	-36.31	-36.80	-20.00	-51.43	-42.86
80% A + 19%AW+1%L	-35.09	-36.43	-36.92	-48.57	-51.43	-42.86
80% A + 17%AW+3%L	-71.52	-71.76	-72.49	-94.29	-91.43	-90.00
80% A + 15%AW+5%L	-71.64	-71.88	-72.74	-94.86	-92.86	-96.57
80% A + 13%AW+7%L	-72.49	-72.74	-72.86	-95.14	-92.86	-94.86
80% A + 11%AW+9%L	-72.98	-74.33	-74.45	-96.00	-94.86	-91.43
75% A + 25%AW	-40.63	-42.44	-44.23	-42.86	-43.36	-43.72
80% A + 20%RP	-27.75	-28.00	-28.24	-31.43	-31.43	-40.00
80% A + 19%RP+1%L	-42.91	-44.25	-45.48	-62.86	-48.57	-68.57
80% A + 17%RP+3%L	-70.05	-70.29	-70.54	-94.29	-90.00	-95.71
80% A + 15%RP+5%L	-72.98	-76.16	-76.65	-94.57	-94.71	-96.00
80% A + 13%RP+7%L	-74.69	-76.41	-77.14	-94.86	-95.45	-97.43
80% A + 11%RP+9%L	-75.18	-76.89	-78.85	-95.43	-95.71	-97.57
75% A + 25%RP	-48.85	-50.57	-51.48	-48.57	-45.71	-42.86

***** ‘+’ shows the increase in values and ‘-’ shows the decrease in values.

CHAPTER 5

CONLUSIONS

The effect of lime, aggregate waste and rock powder addition on swelling potential of an expansive soil sample is presented. Lime was introduced as an admixture up to a maximum of 9 percent; aggregate waste and rock powder were introduced up to 25 percent by dry weight of the soil. According to the results of the experiments, the following conclusions can be listed:

1. The grain size distributions of the soils tested are altered by the addition of all the admixtures. The silt size particles increased whereas clay size particles decreased with increasing amount of admixtures. The grain size distributions curve of Sample A shifted significantly to the coarser side when aggregate waste or rock powder was added. It is mainly depends on the addition of silt sized particles.

2. Liquid limit and Plasticity Index values of Sample A decreased with the addition of all stabilizers.

3. Addition of the stabilizers change the swelling potential of Sample A. Sample A is described as a high swelling potentially soil. Especially rock powder added samples changed the potential type into medium, according to the chart of Seed et al. (1962)

4. The addition of lime increased the specific gravity but the addition of aggregate waste and rock powder with lime decreased the specific gravity value of Sample A.

5. Activity value of Sample A decreased as percent stabilizer increased.

6. As a second material, aggregate waste and rock powder showed a significant change in swelling percentage of Sample A when used with lime. When only aggregate waste and rock powder were added to the soil without lime, there was a reduction in the swelling potential, too.

7. Addition of 3% lime to the samples reduces swell percentage significantly after 3 % lime addition there is no significant change in swell percentages.

8. Generally all samples showed a small reduction in swelling potential after curing.

9. On the basis of this research study both aggregate waste and rock powder with lime can be recommended as effective stabilizing agents for improvement of expansive soils for the construction of dam and highway embankments. The use of aggregate waste and rock powder as stabilizing agents can be economically attractive in regions near to the areas where these waste by-products are obtained. Utilization of aggregate waste and rock powder in this manner also has the advantage of reusing and industrial waste by-product without adversely affecting the environment or potential land use.

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APPENDIX A

Swell percentage versus time relationship of the samples for 0 days curing are shown in Appendix A.

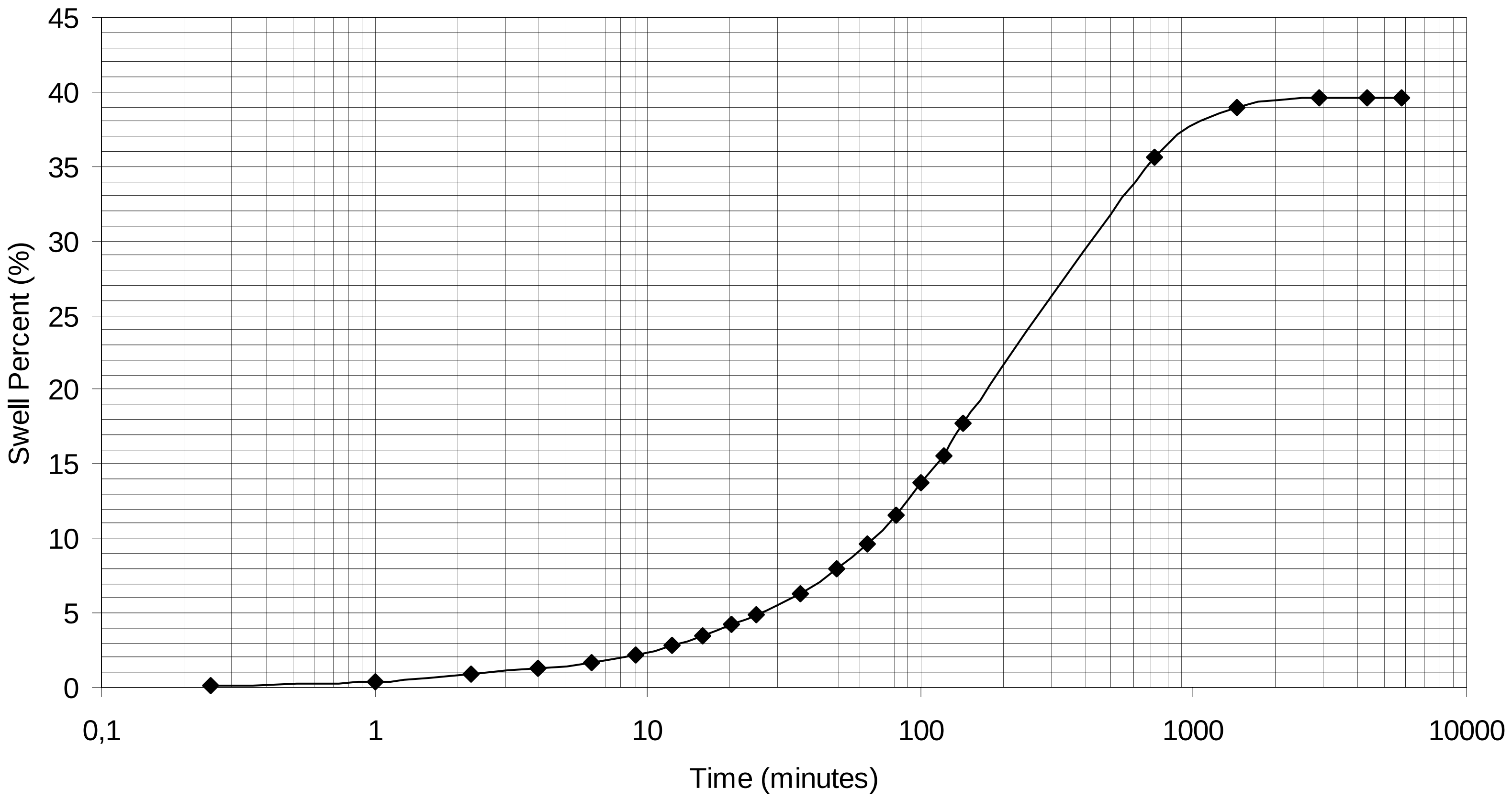

Figure A.1. Swell Percentage versus Time Relationship for Sample 99%A+1%L

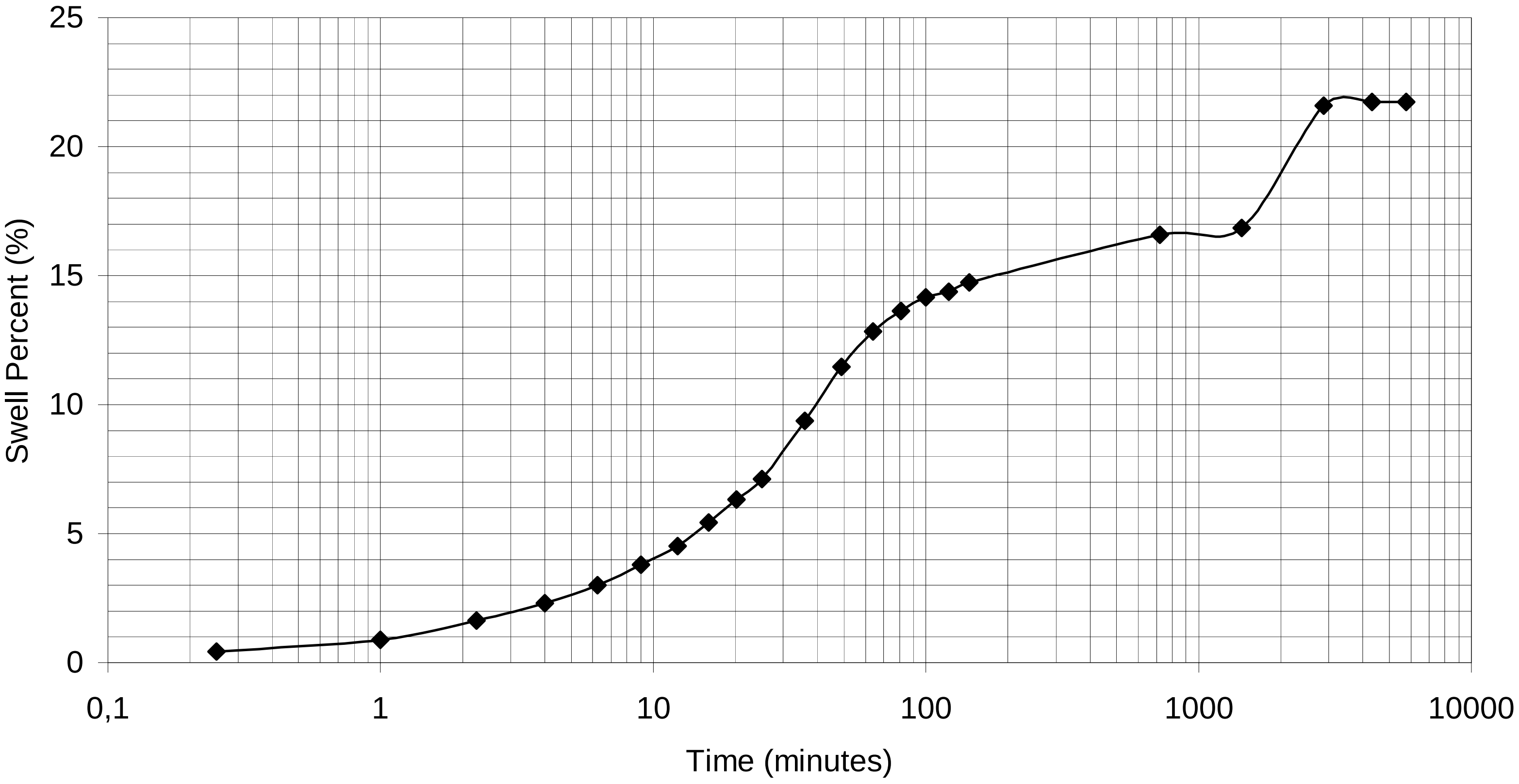

Figure A.2. Swell Percentage versus Time Relationship for Sample 97%A+3%L

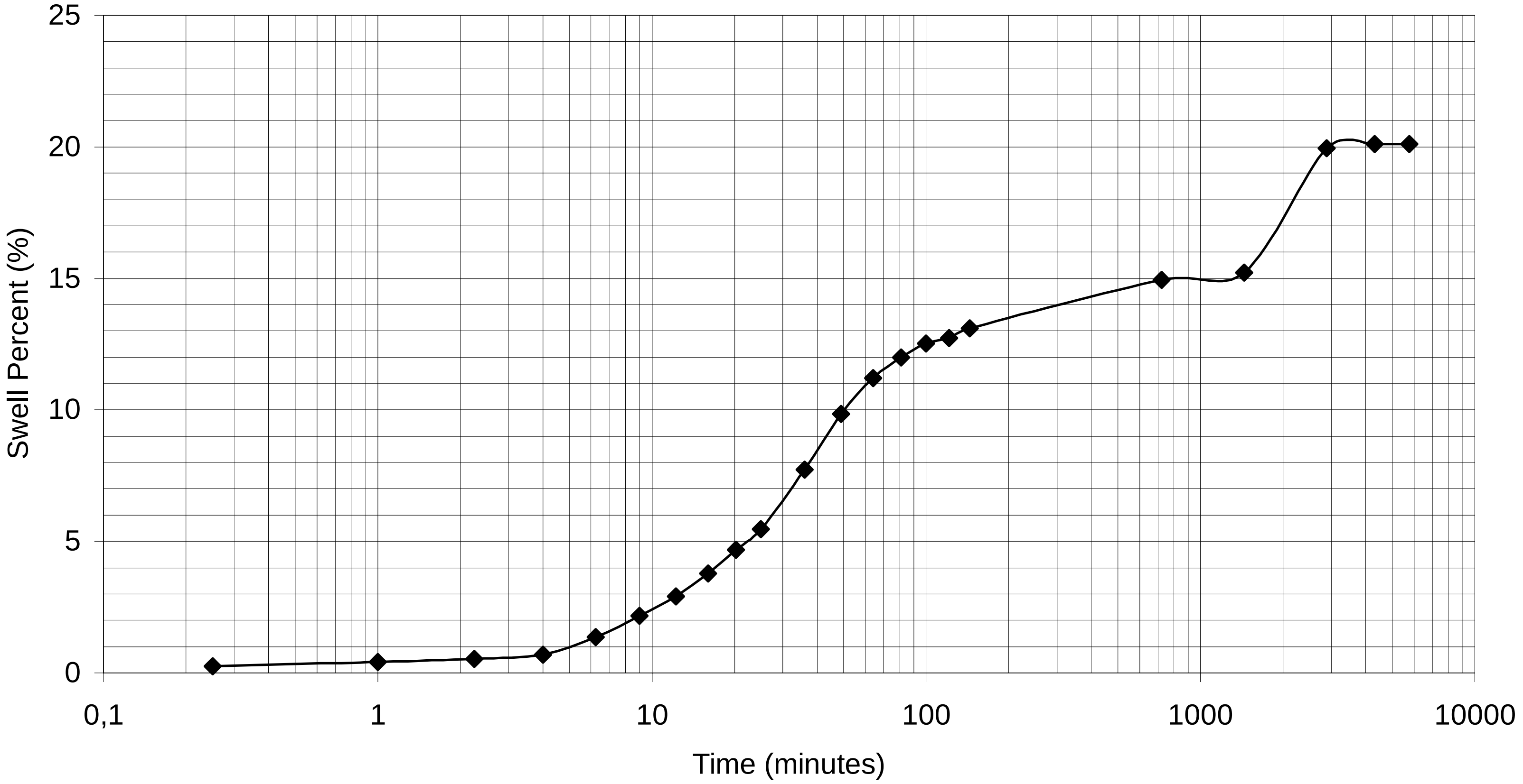

Figure A.3. Swell Percentage versus Time Relationship for Sample 95%A+5%L.

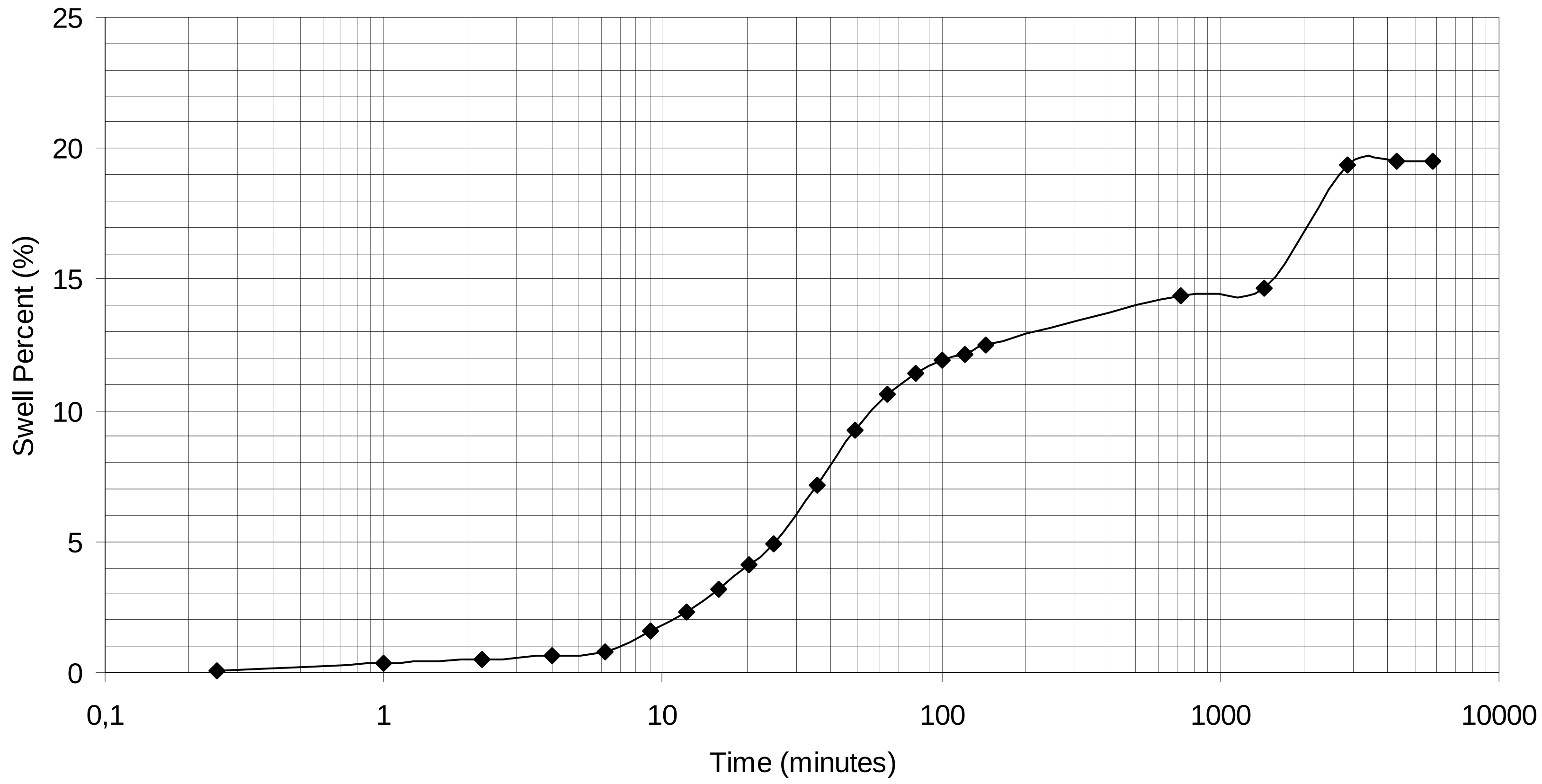

Figure A.4. Swell Percentage versus Time Relationship for Sample 93%A+7%L.

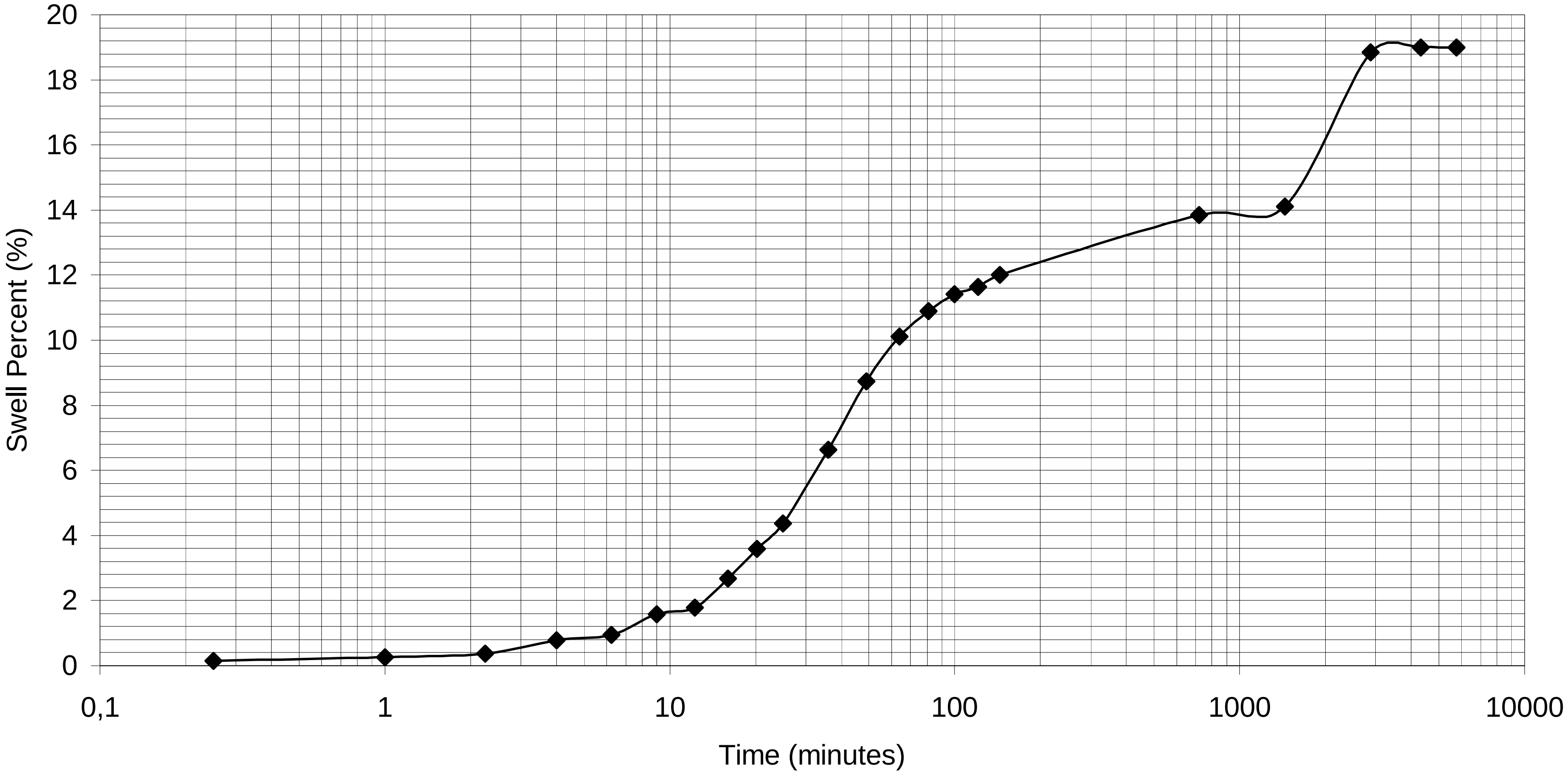

Figure A.5. Swell Percentage versus Time Relationship for Sample 91%A+9%L.

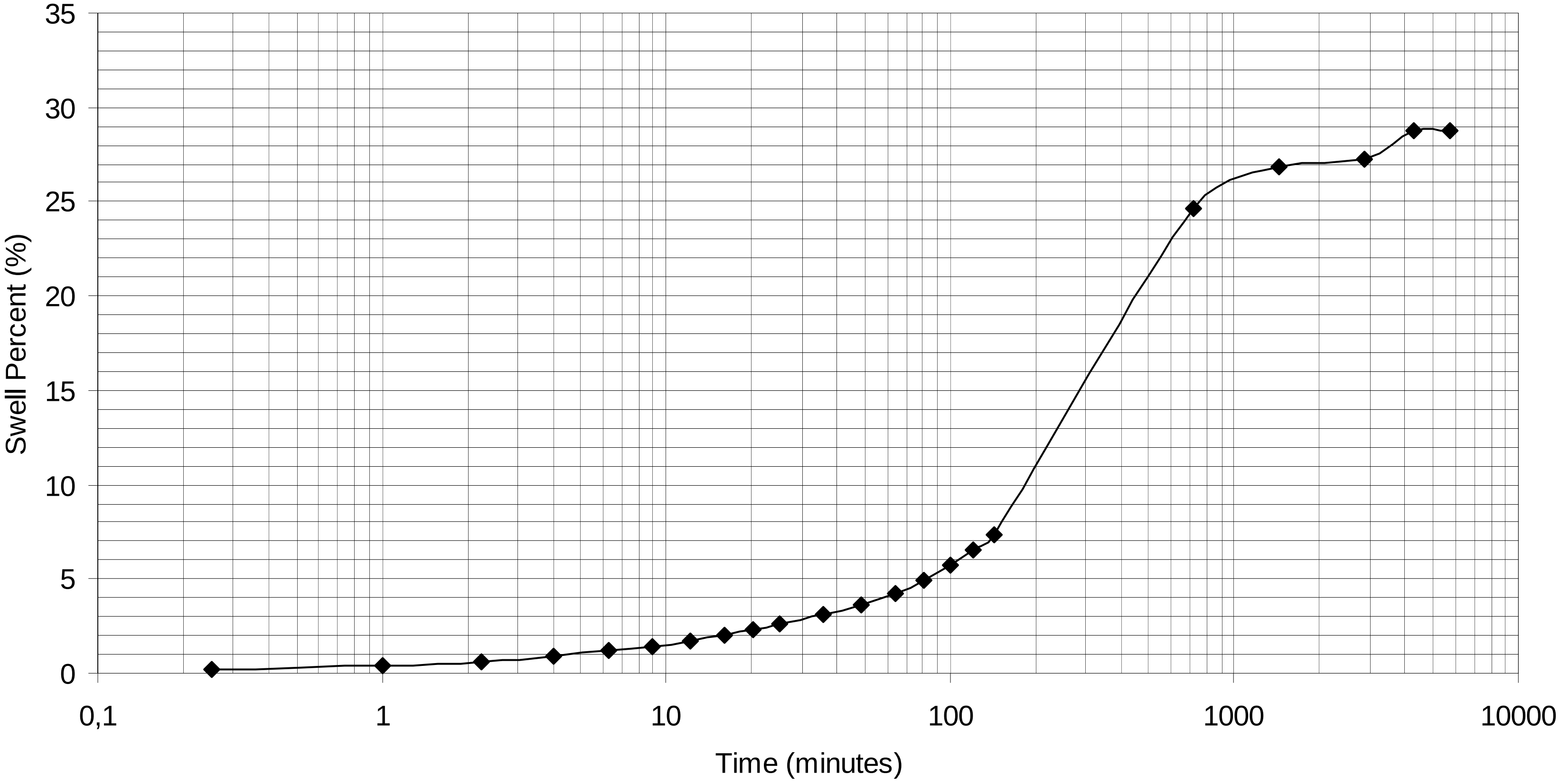

Figure A.6. Swell Percentage versus Time Relationship for Sample 80%A+20%AW.

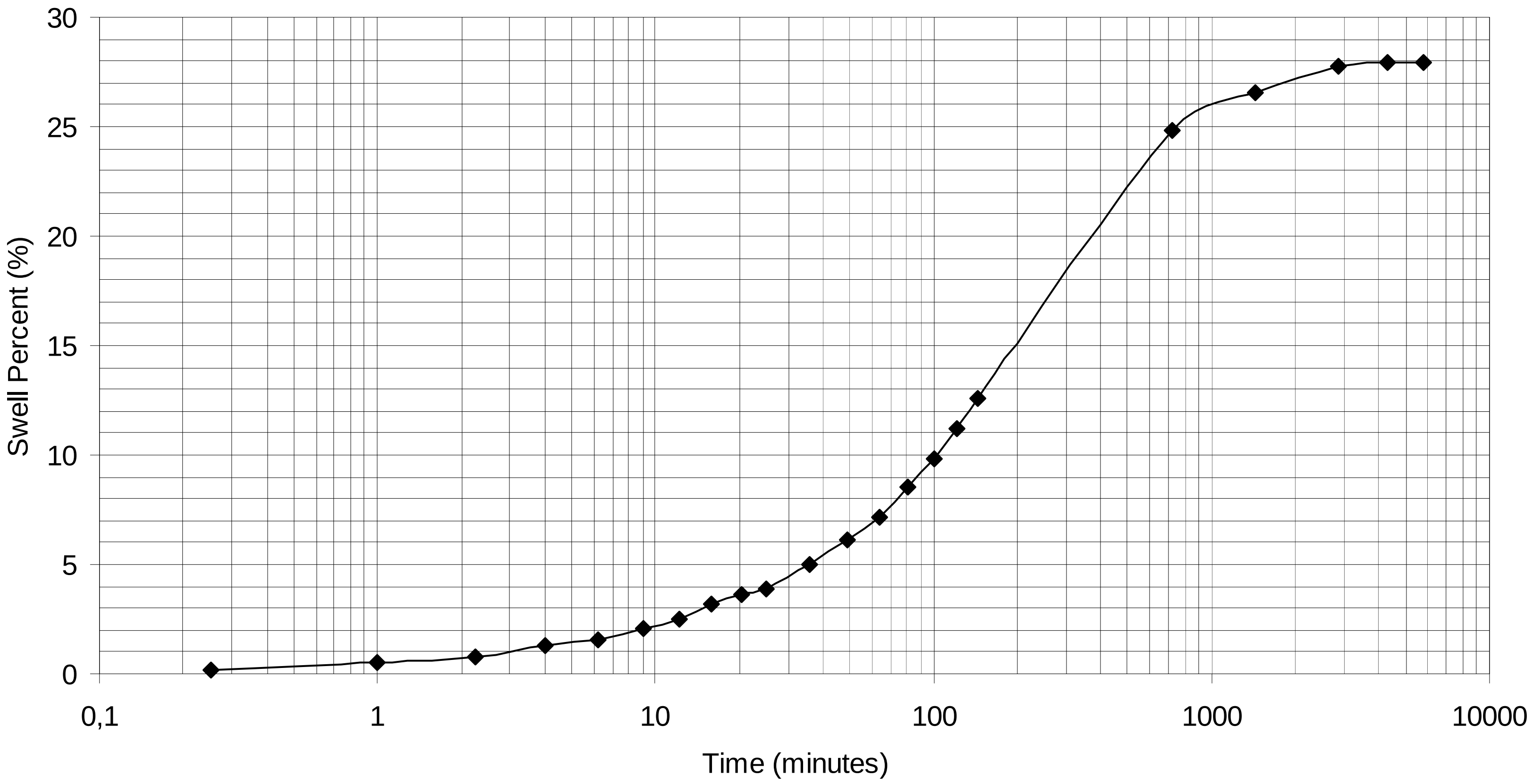

Figure A.7. Swell Percentage versus Time Relationship for Sample 80%A+19%AW+1%L.

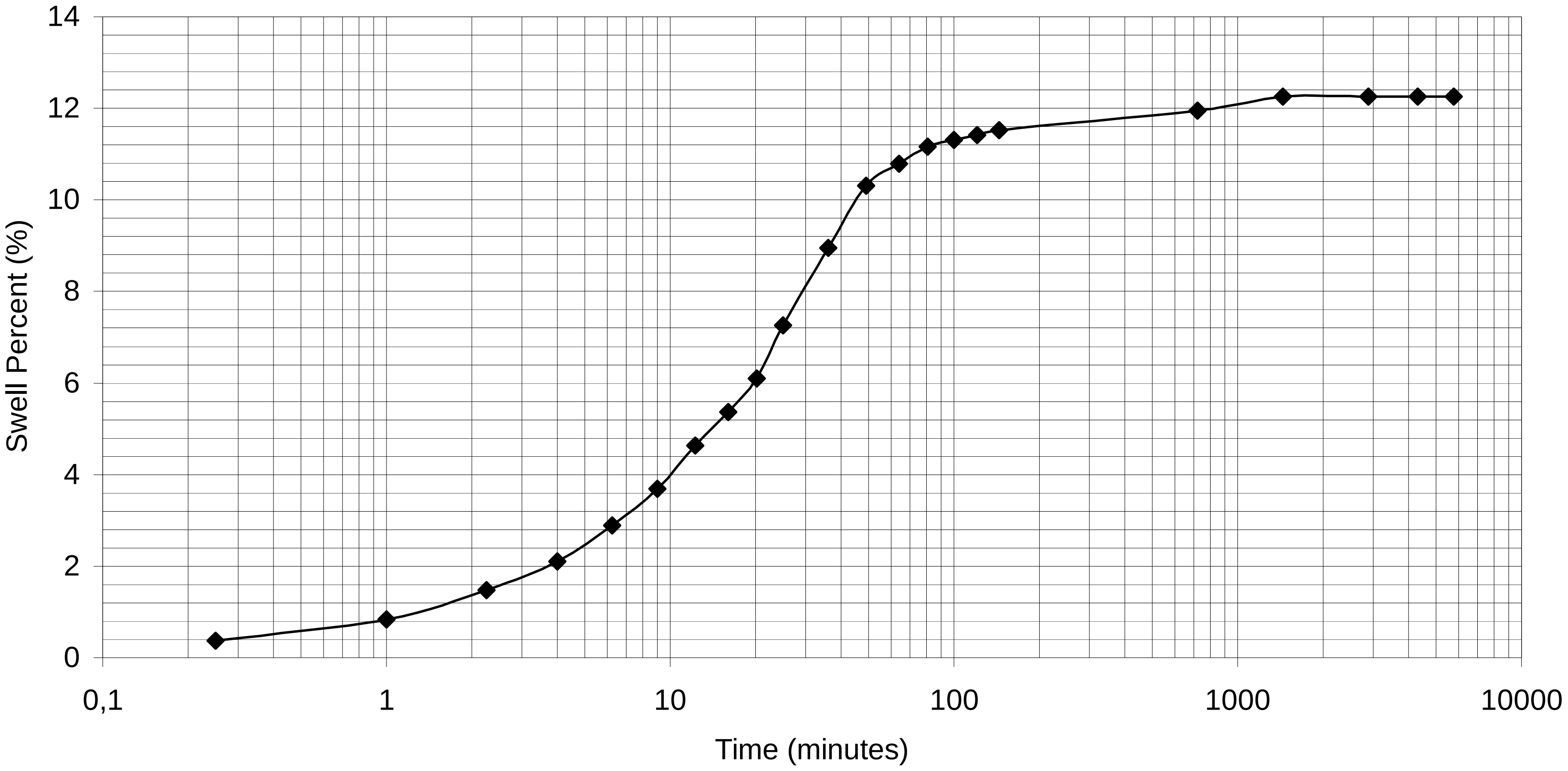

Figure A.8. Swell Percentage versus Time Relationship for Sample 80%A+17%AW+3%L.

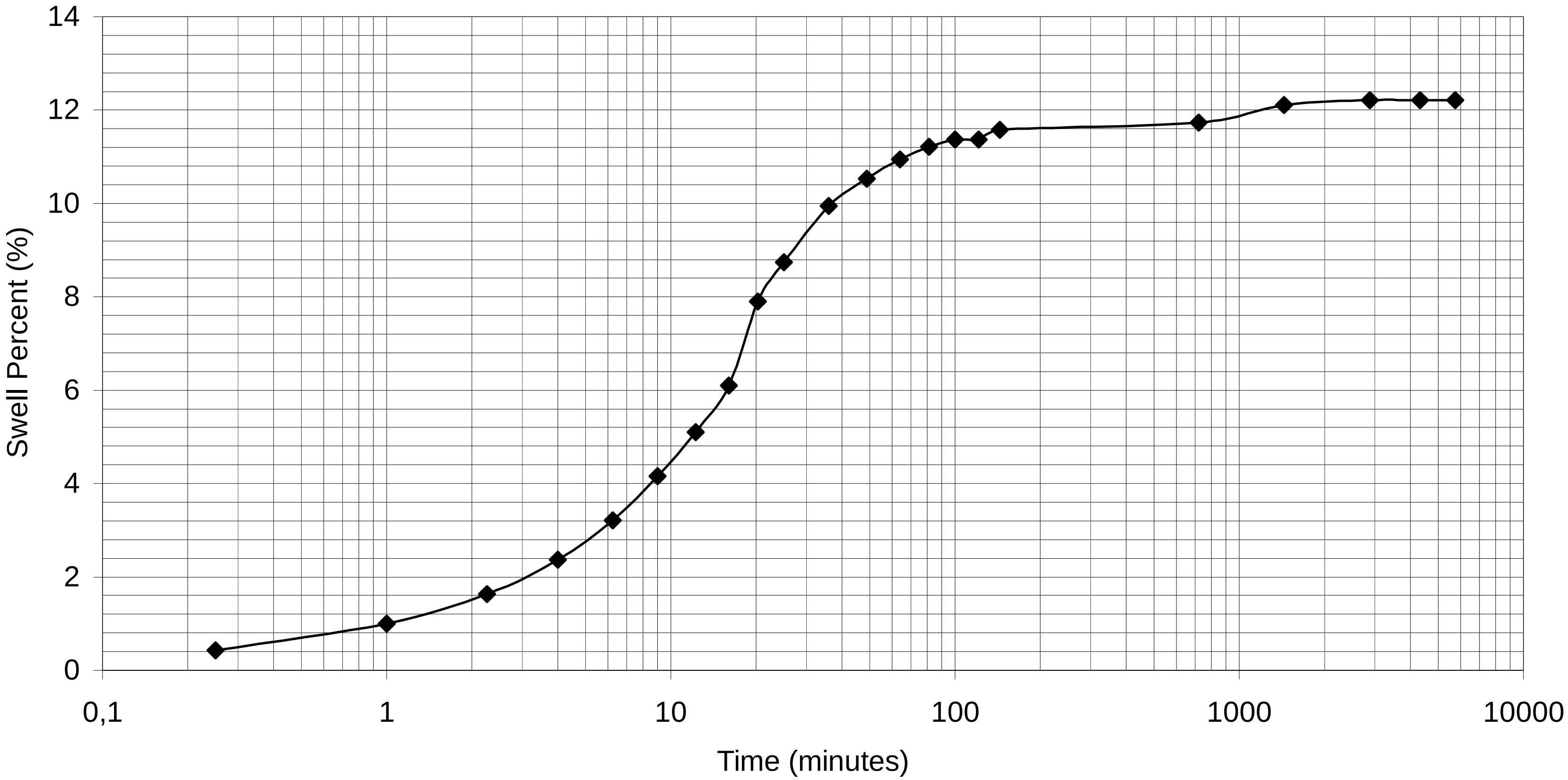

Figure A.9. Swell Percentage versus Time Relationship for Sample 80%A+15%AW+5%L

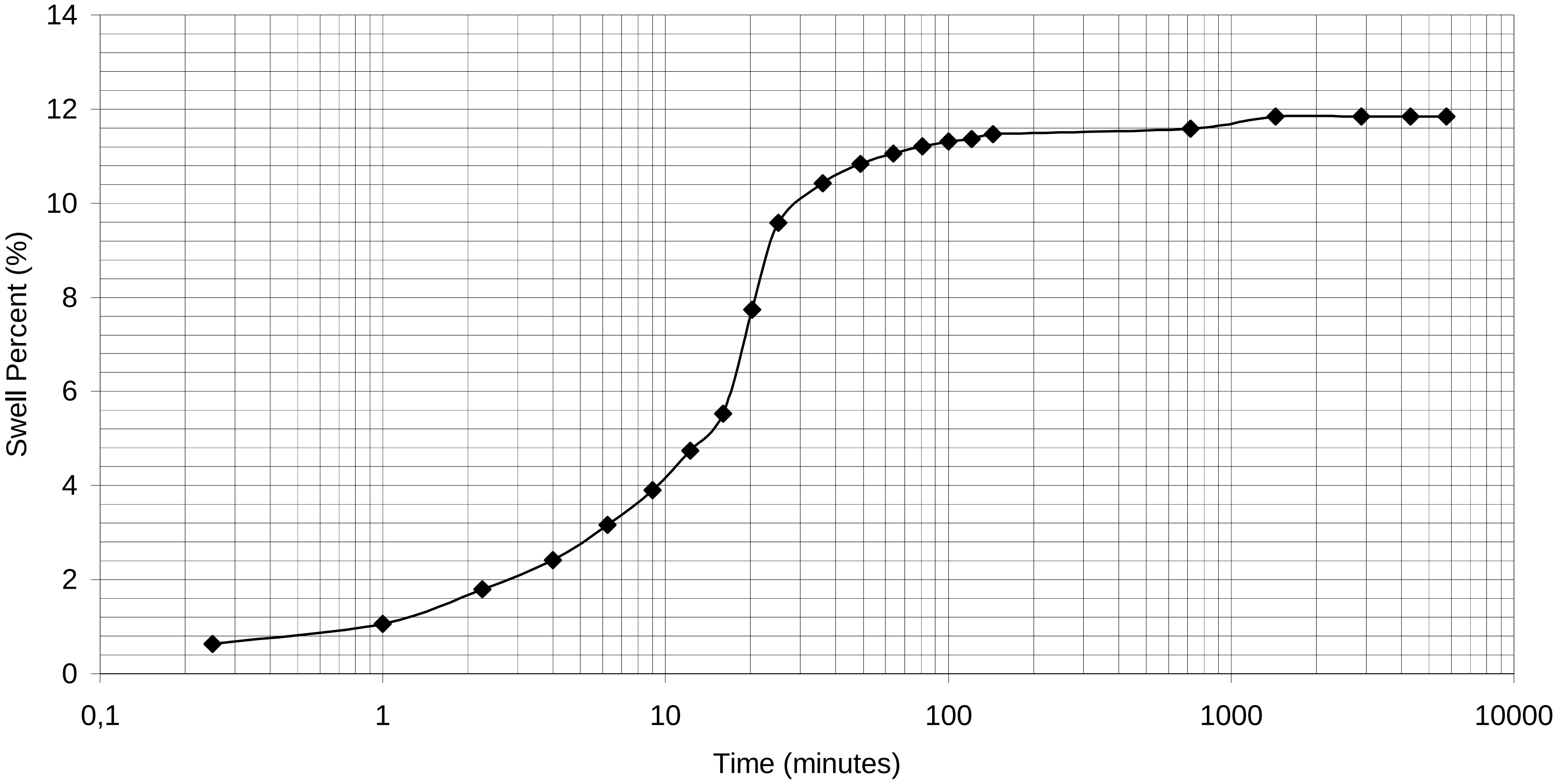

Figure A.10. Swell Percentage versus Time Relationship for Sample 80%A+13%AW+7%L.

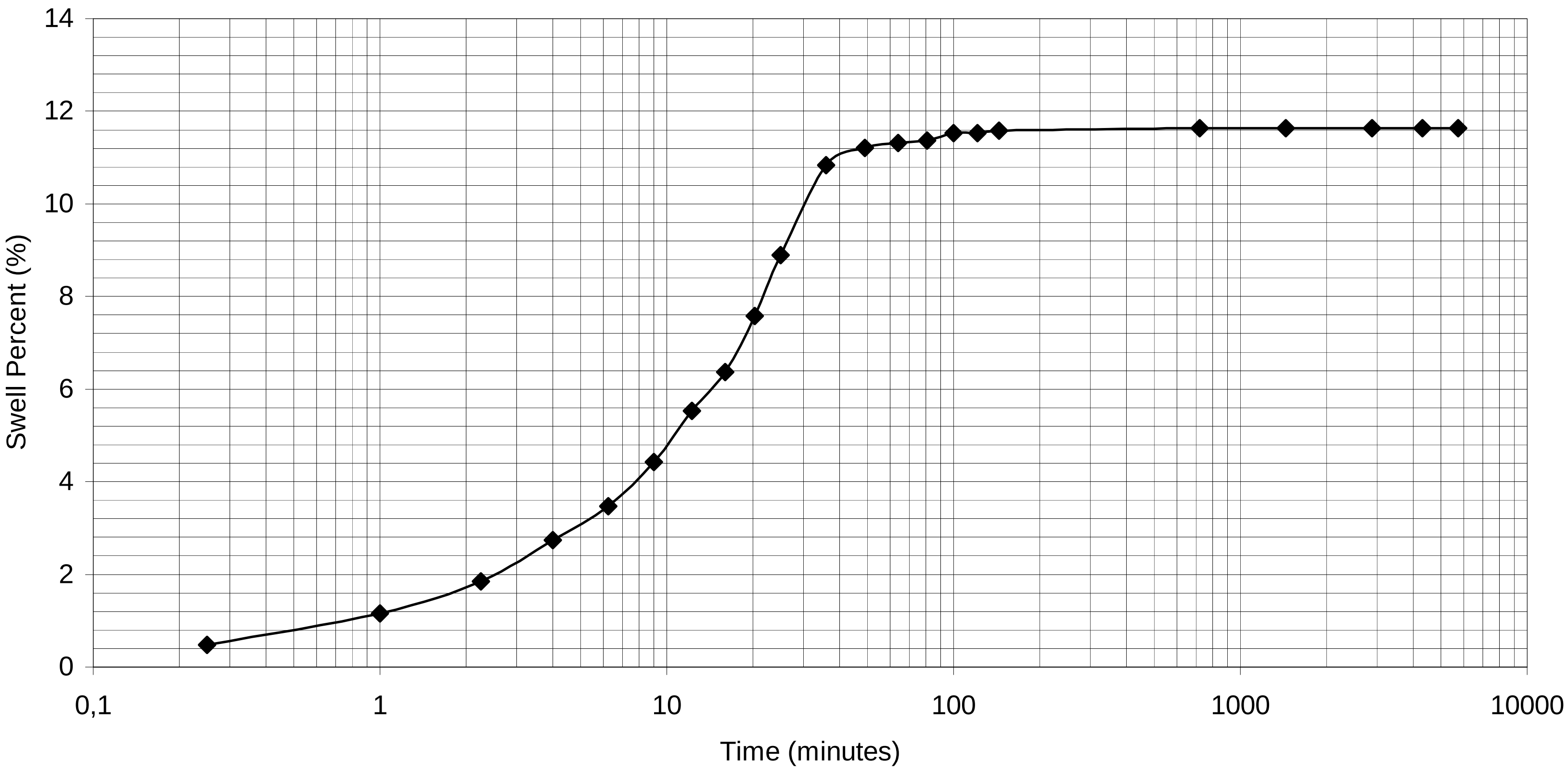

Figure A.11. Swell Percentage versus Time Relationship for Sample 80%A+11%AW+9%L.

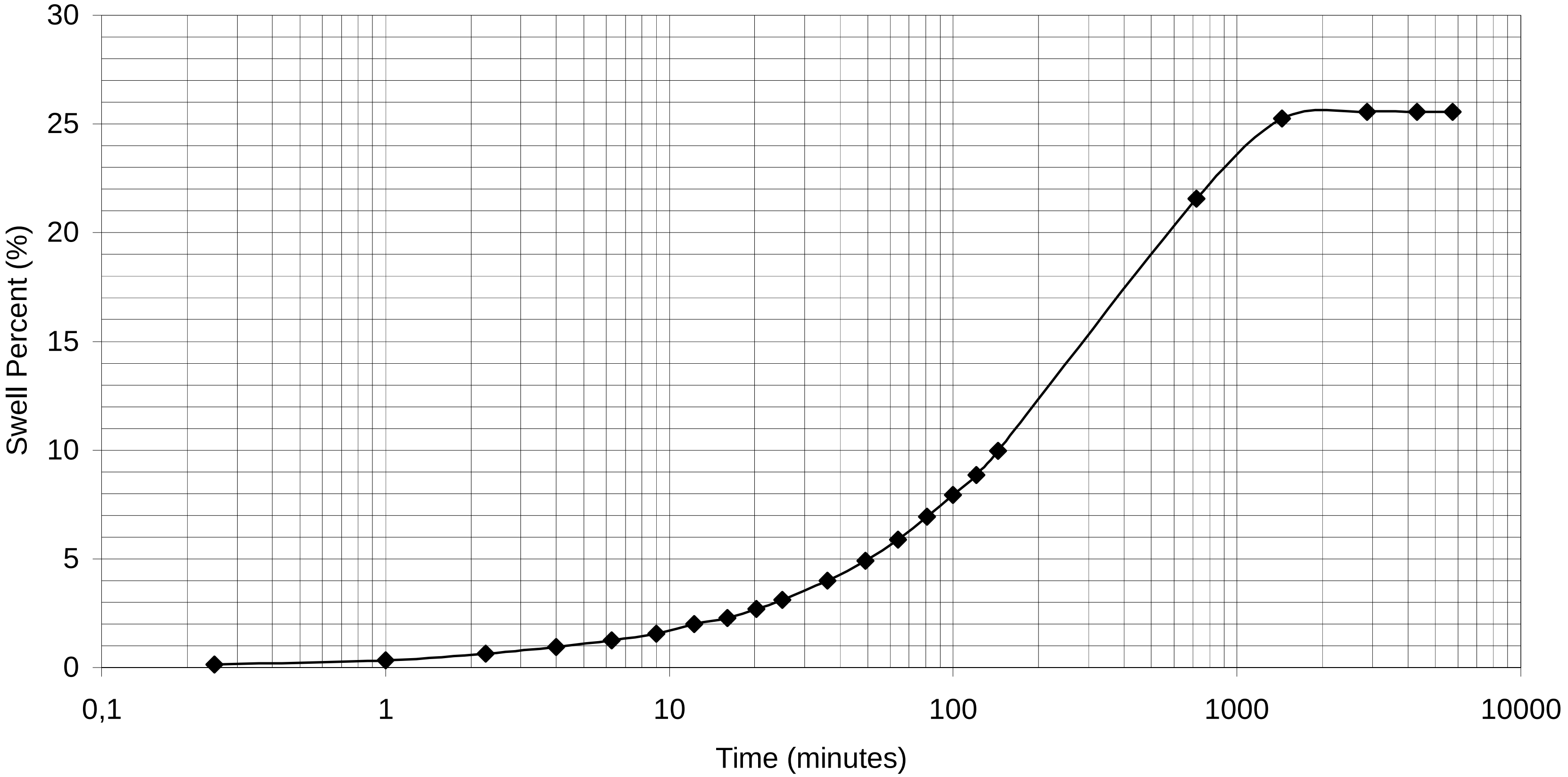

Figure A.12. Swell Percentage versus Time Relationship for Sample 75%A+25%AW.

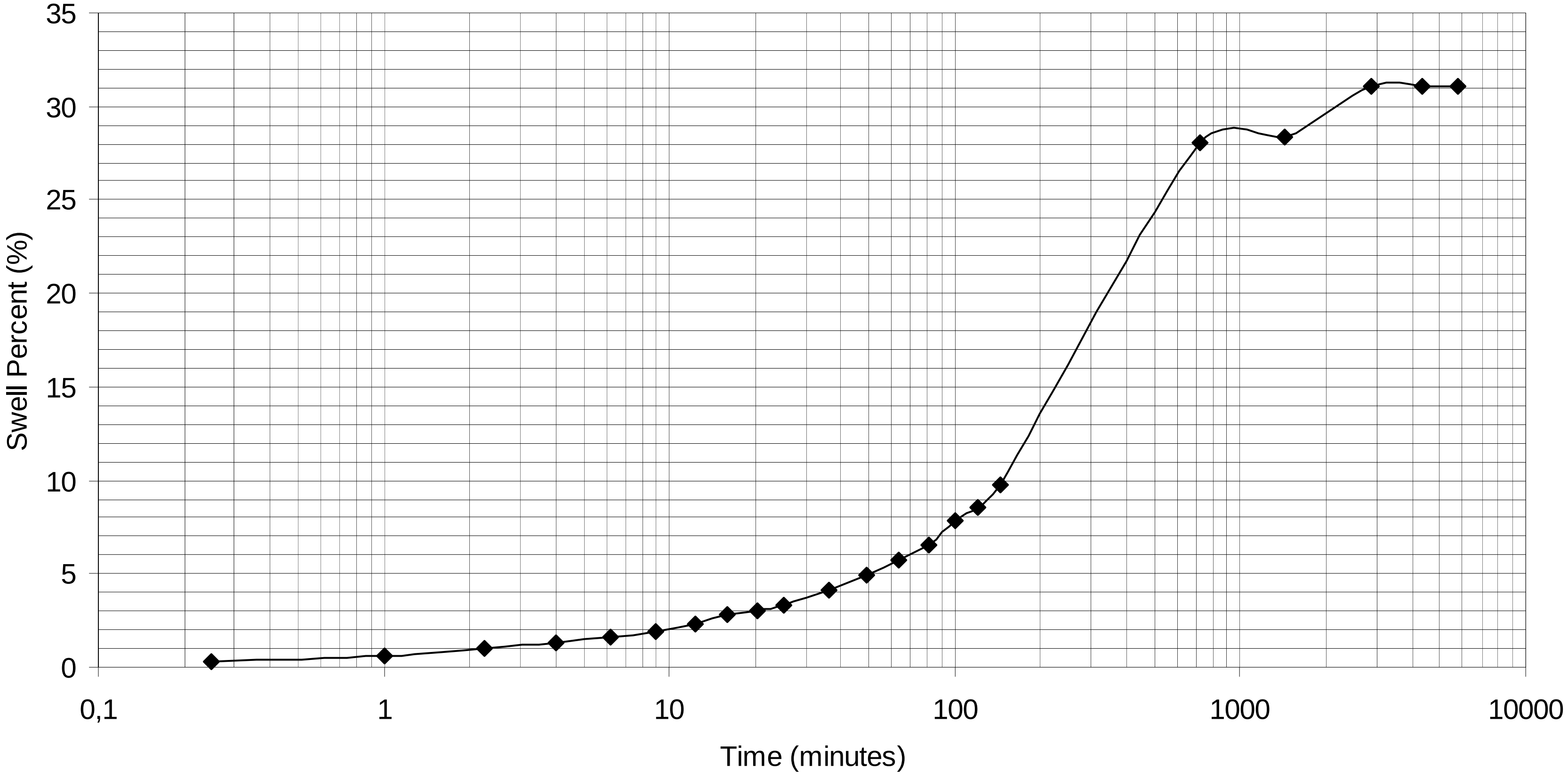

Figure A.13. Swell Percentage versus Time Relationship for Sample 80%A+20%RP.

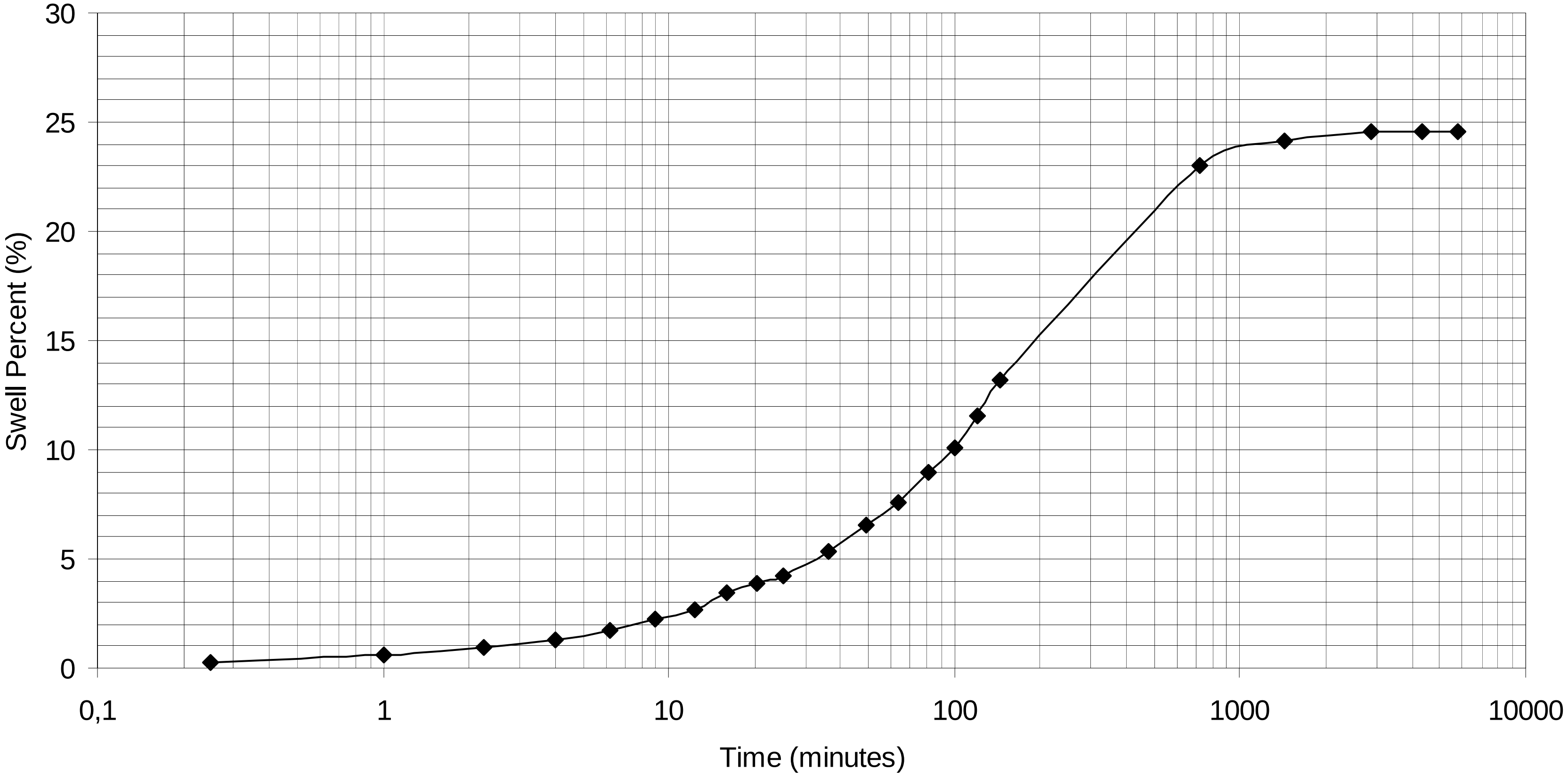

Figure A.14. Swell Percentage versus Time Relationship for Sample 80%A+19%RP+1%L.

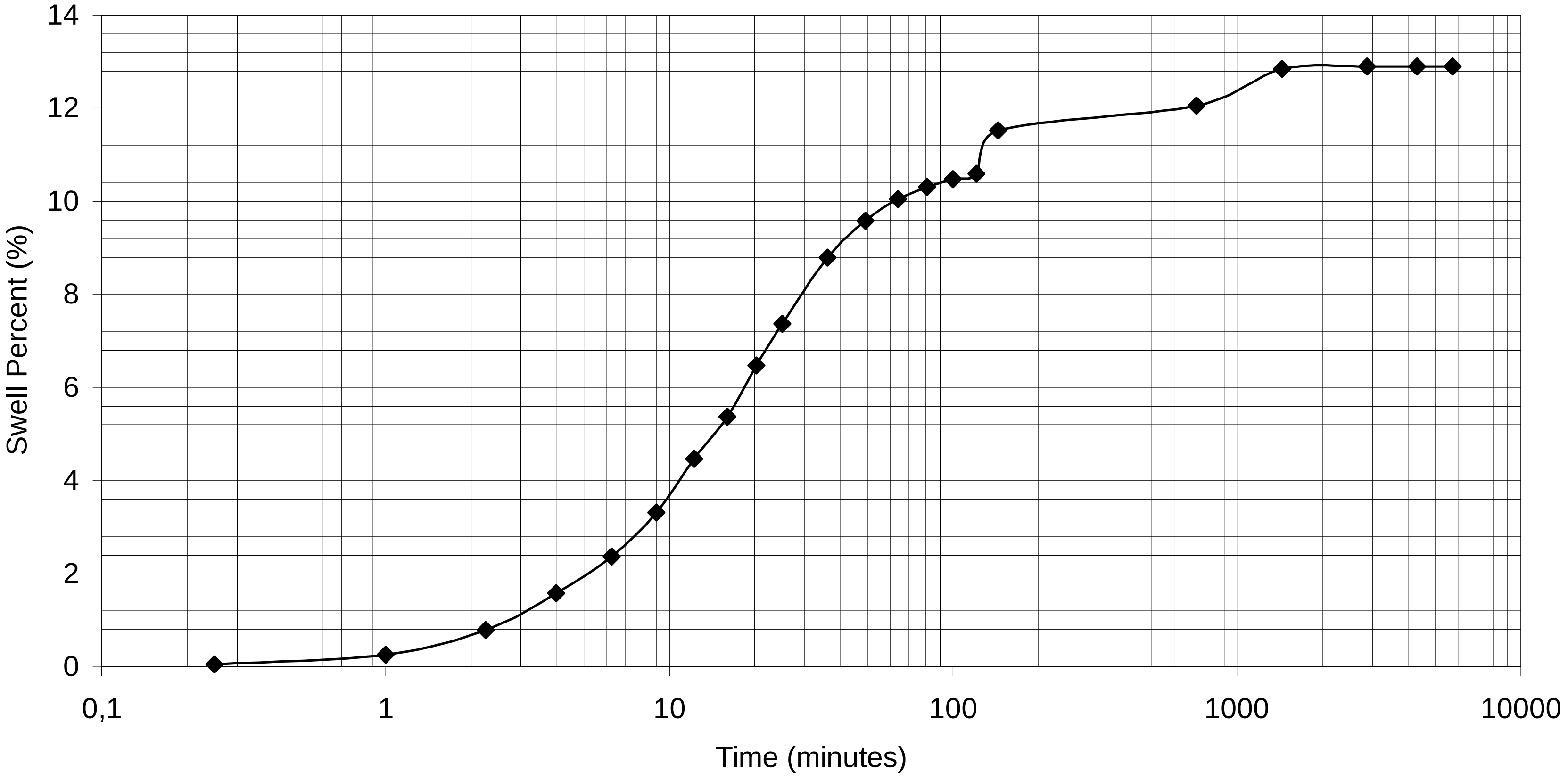

Figure A.15. Swell Percentage versus Time Relationship for Sample 80%A+17%RP+3%L.

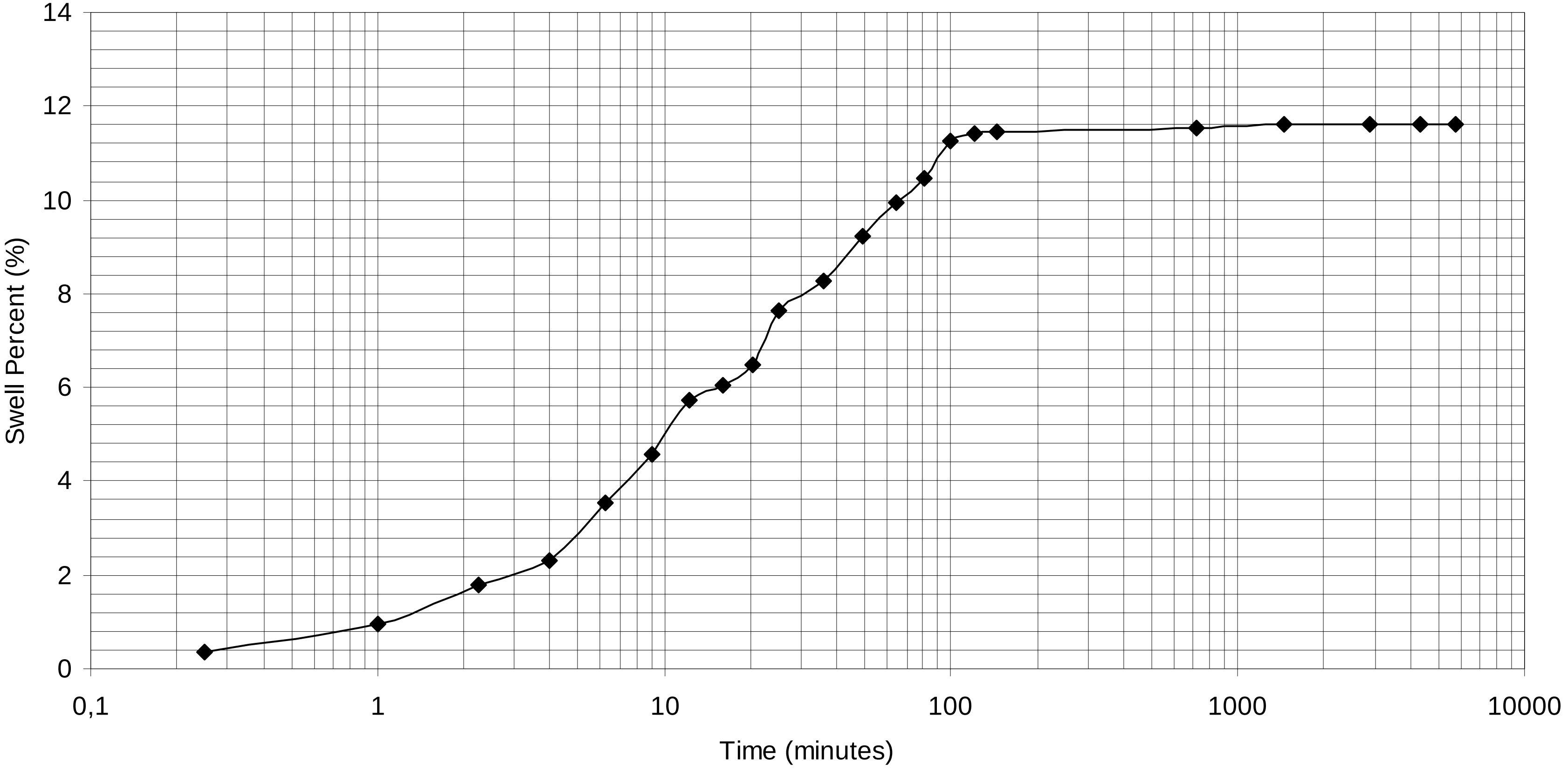

Figure A.16. Swell Percentage versus Time Relationship for Sample 80%A+15%RP+5%L.

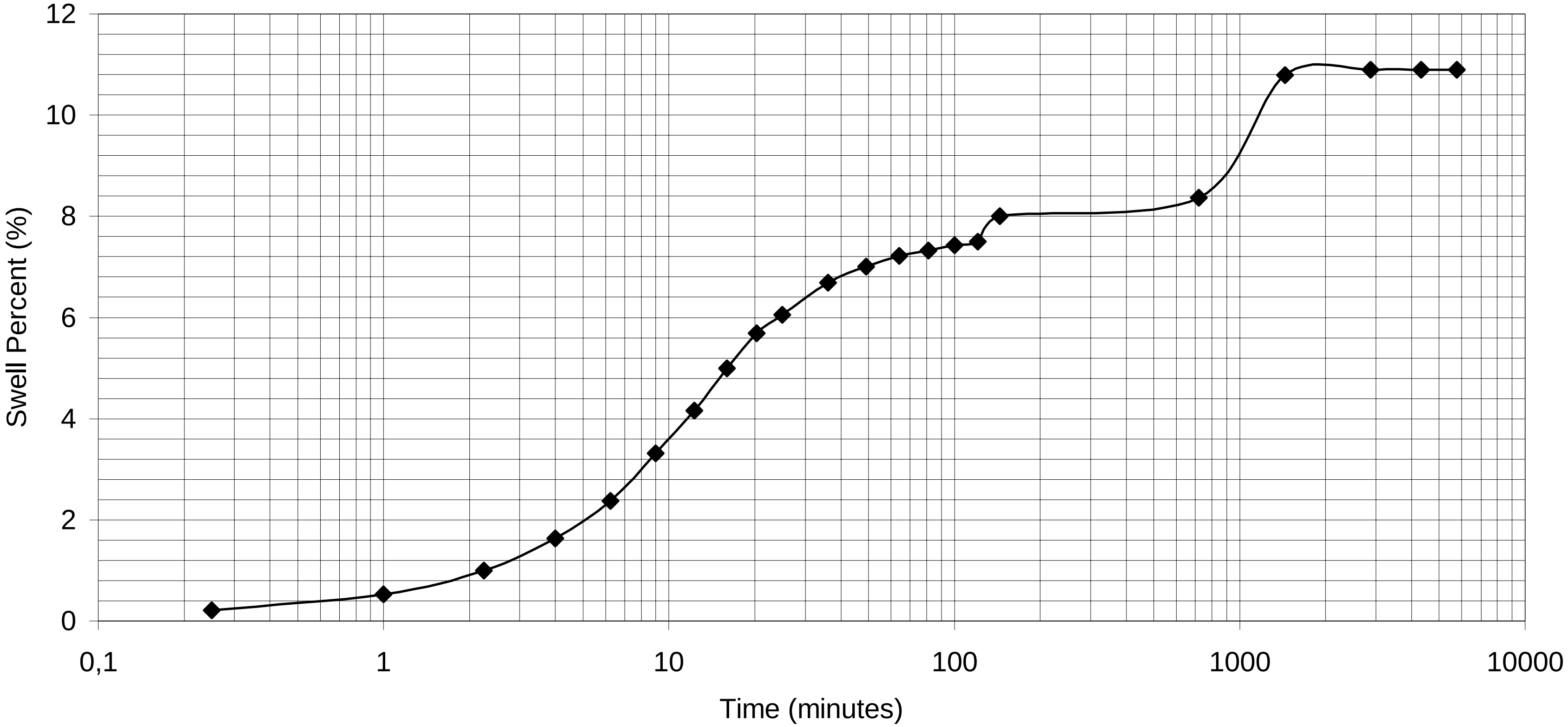

Figure A.17. Swell Percentage versus Time Relationship for Sample 80%A+13%RP+7%L.

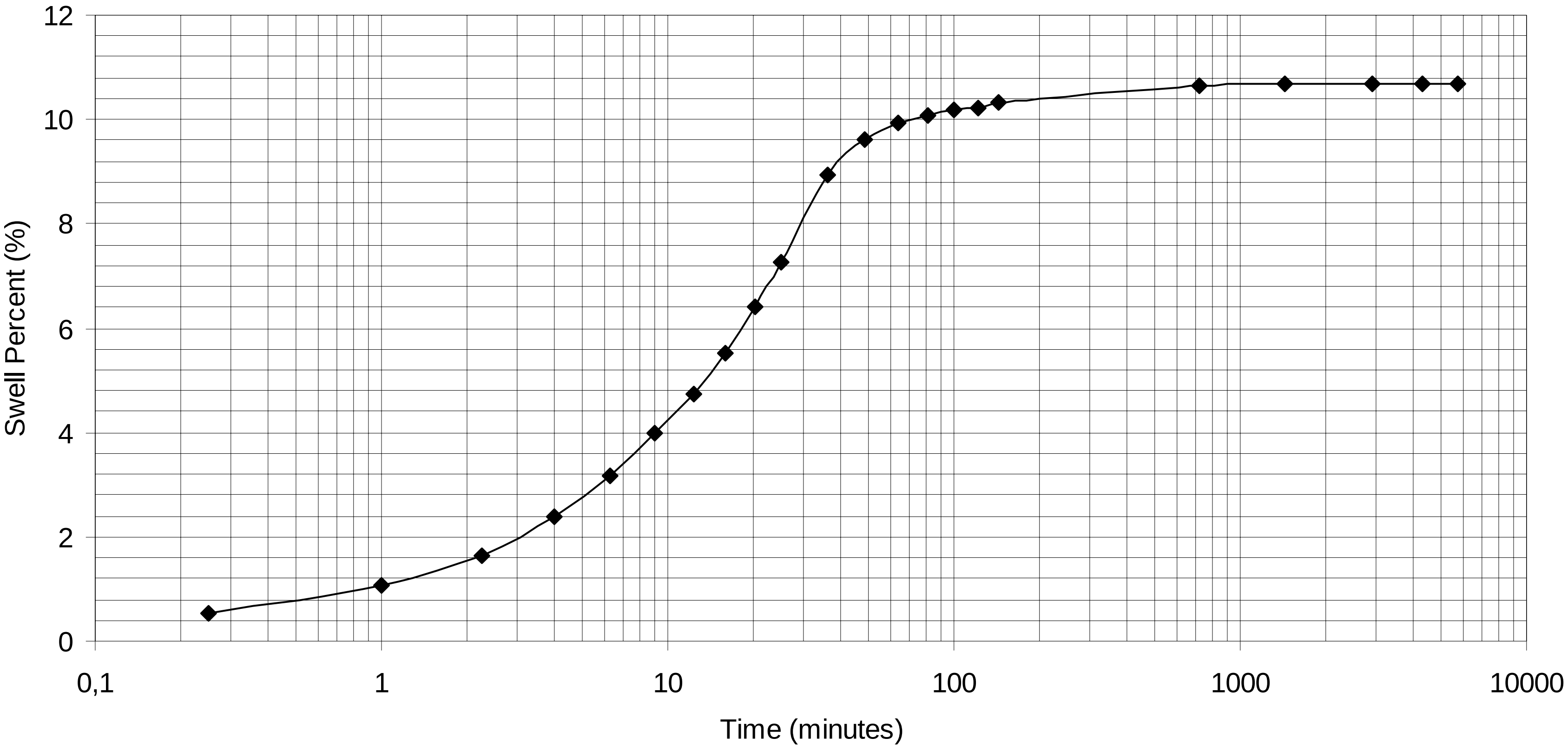

Figure A.18. Swell Percentage versus Time Relationship for Sample 80%A+11%RP+9%L.

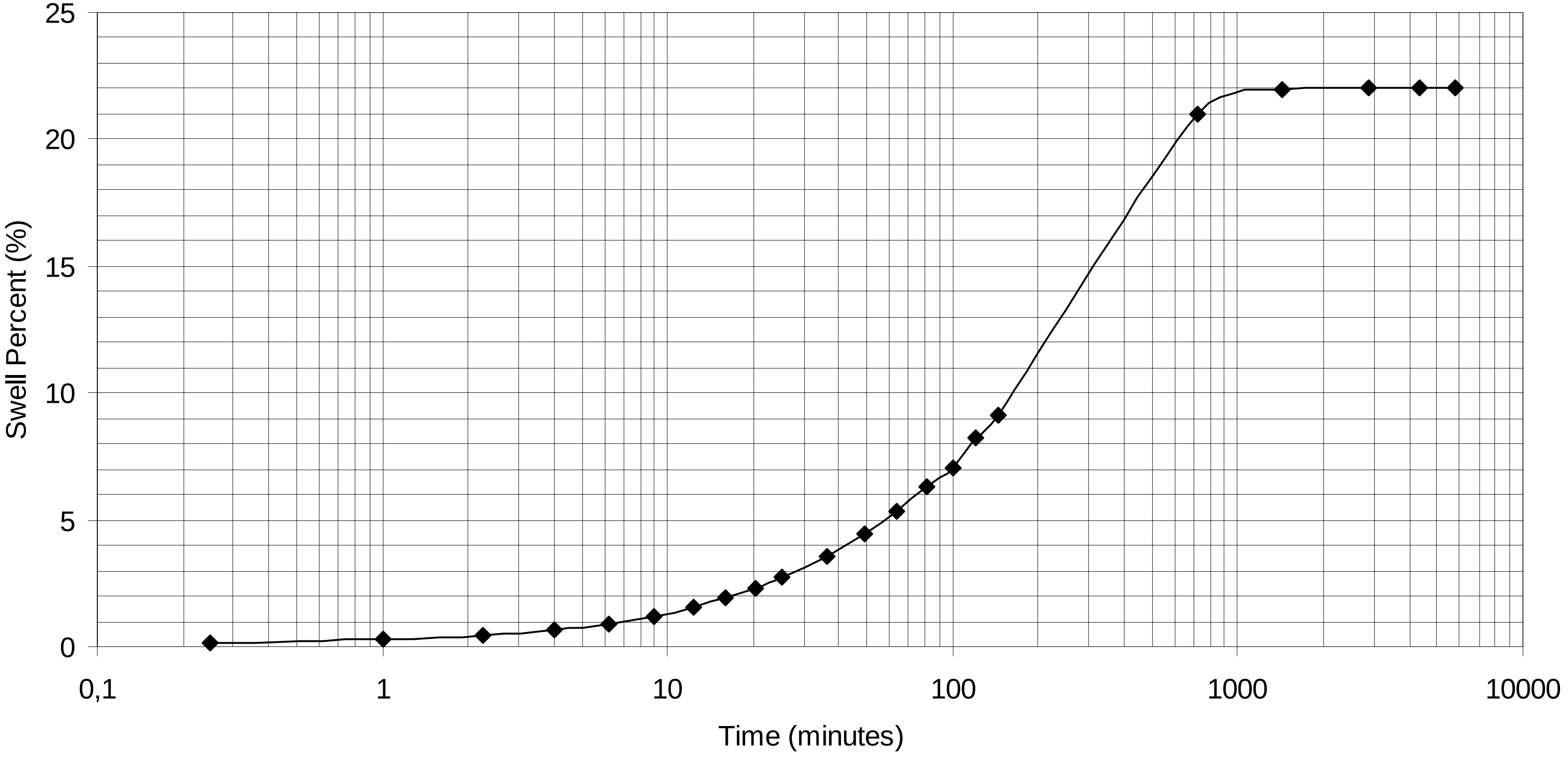

Figure A.19. Swell Percentage versus Time Relationship for Sample 75%A+25%RP.

APPENDIX B

Swell percentage versus time relationship of the samples for 7 days and 28 days curing are shown in Appendix B.

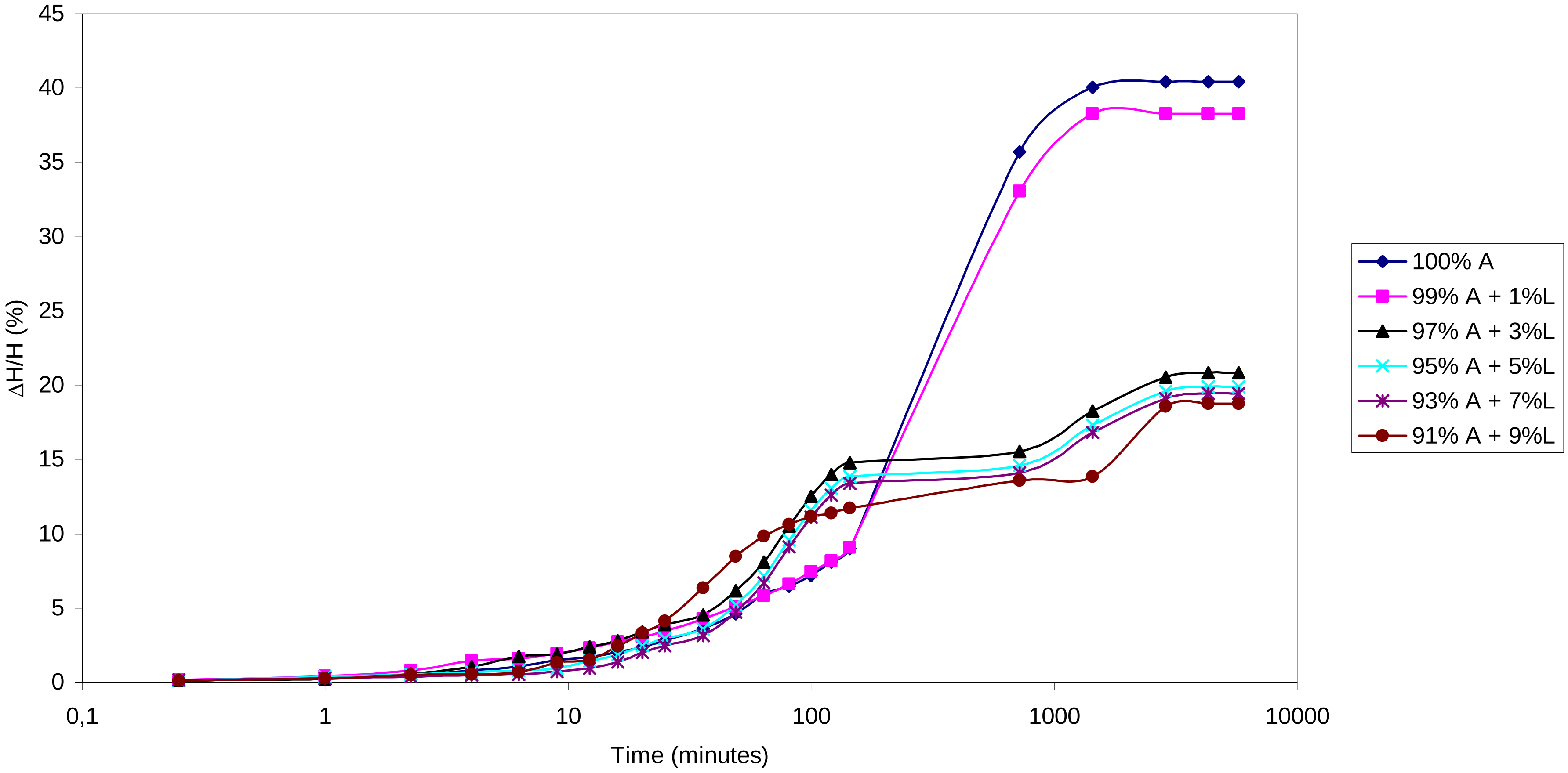

Figure B.1. Swell percentage versus Time relationship for lime added samples after 7 days curing.

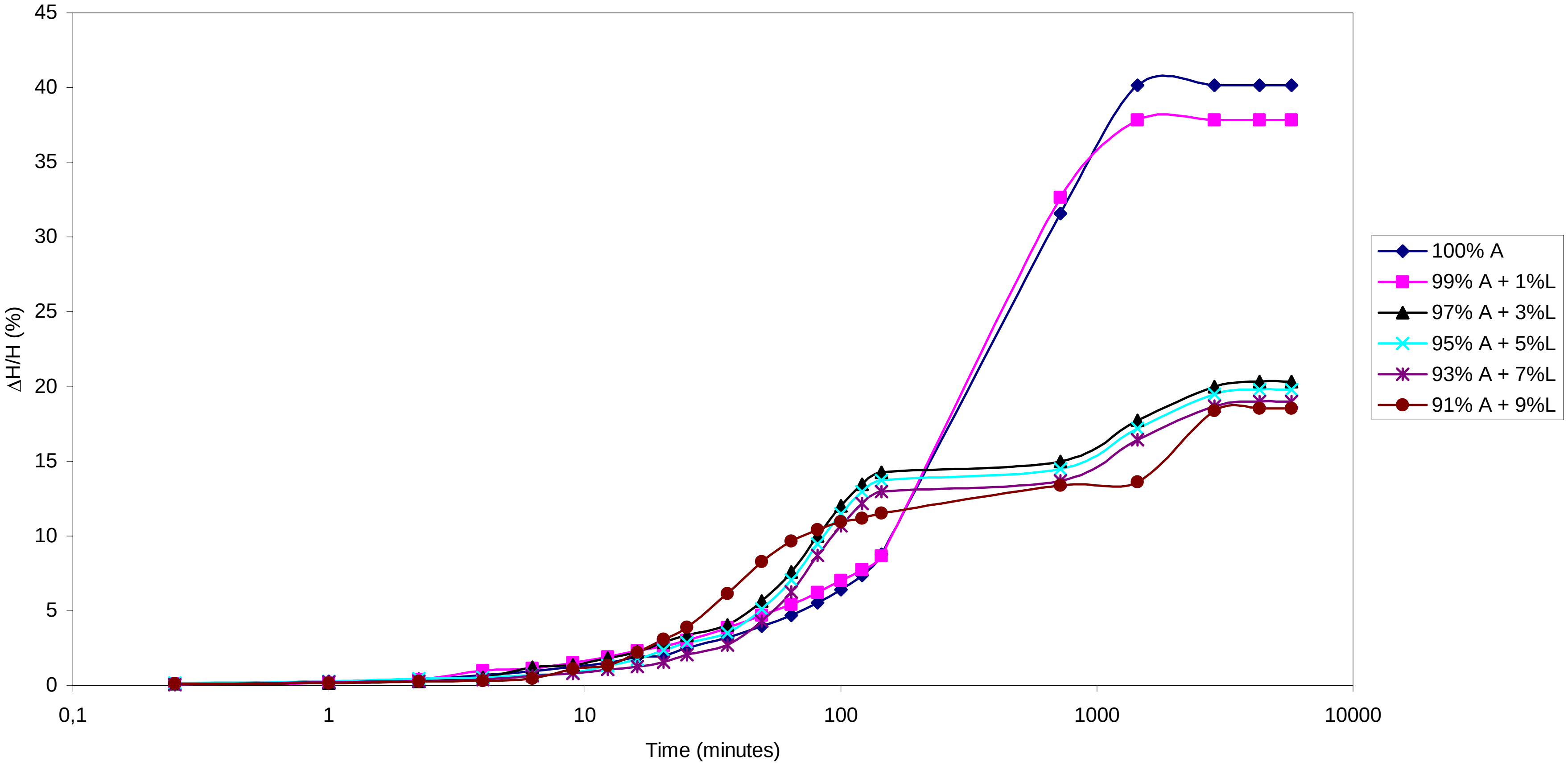

Figure B.2. Swell percentage versus Time relationship for lime added samples after 28 days curing.

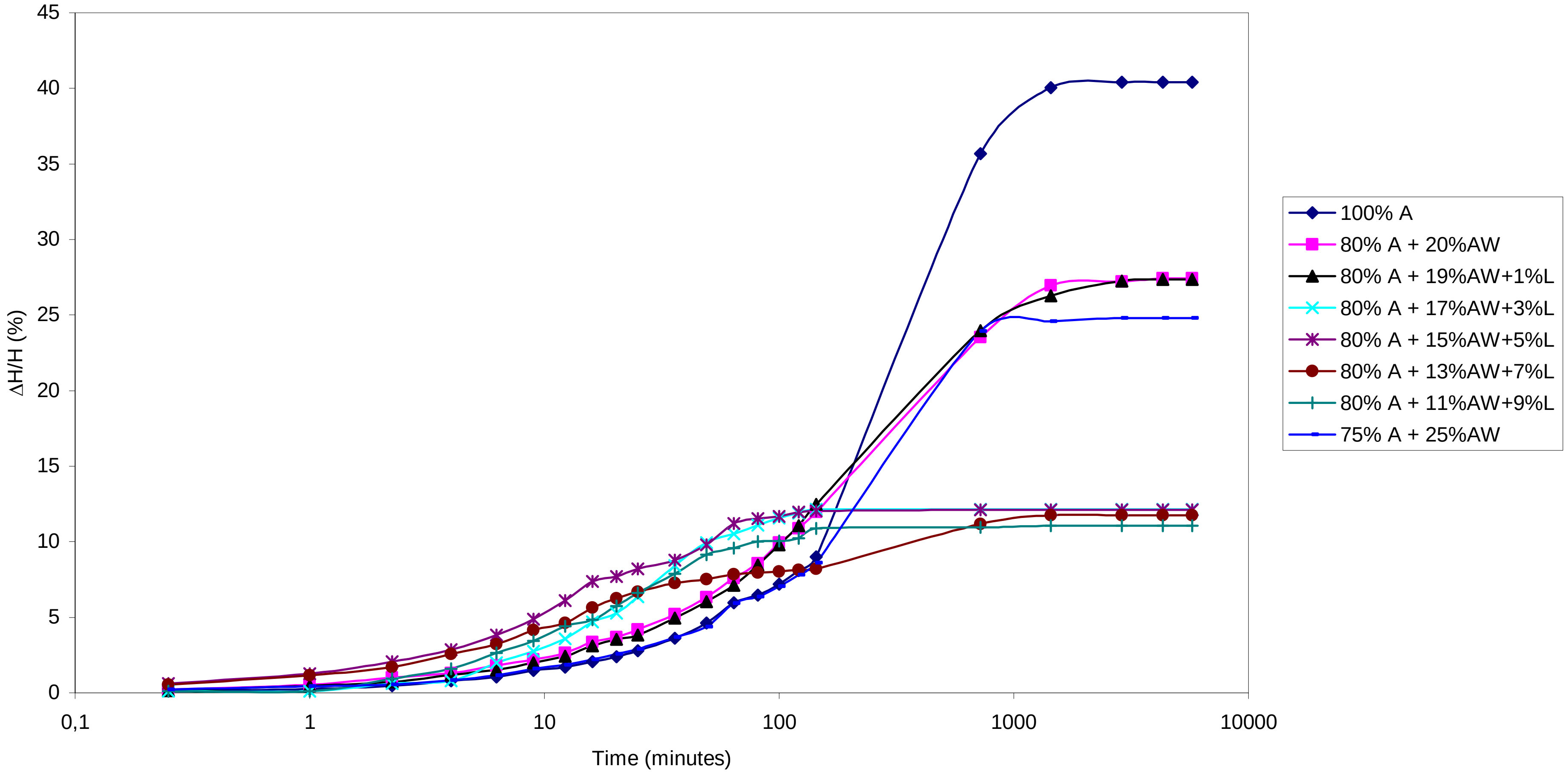

Figure B.3. Swell percentage versus Time relationship for aggregate waste and lime added samples after 7 days curing.

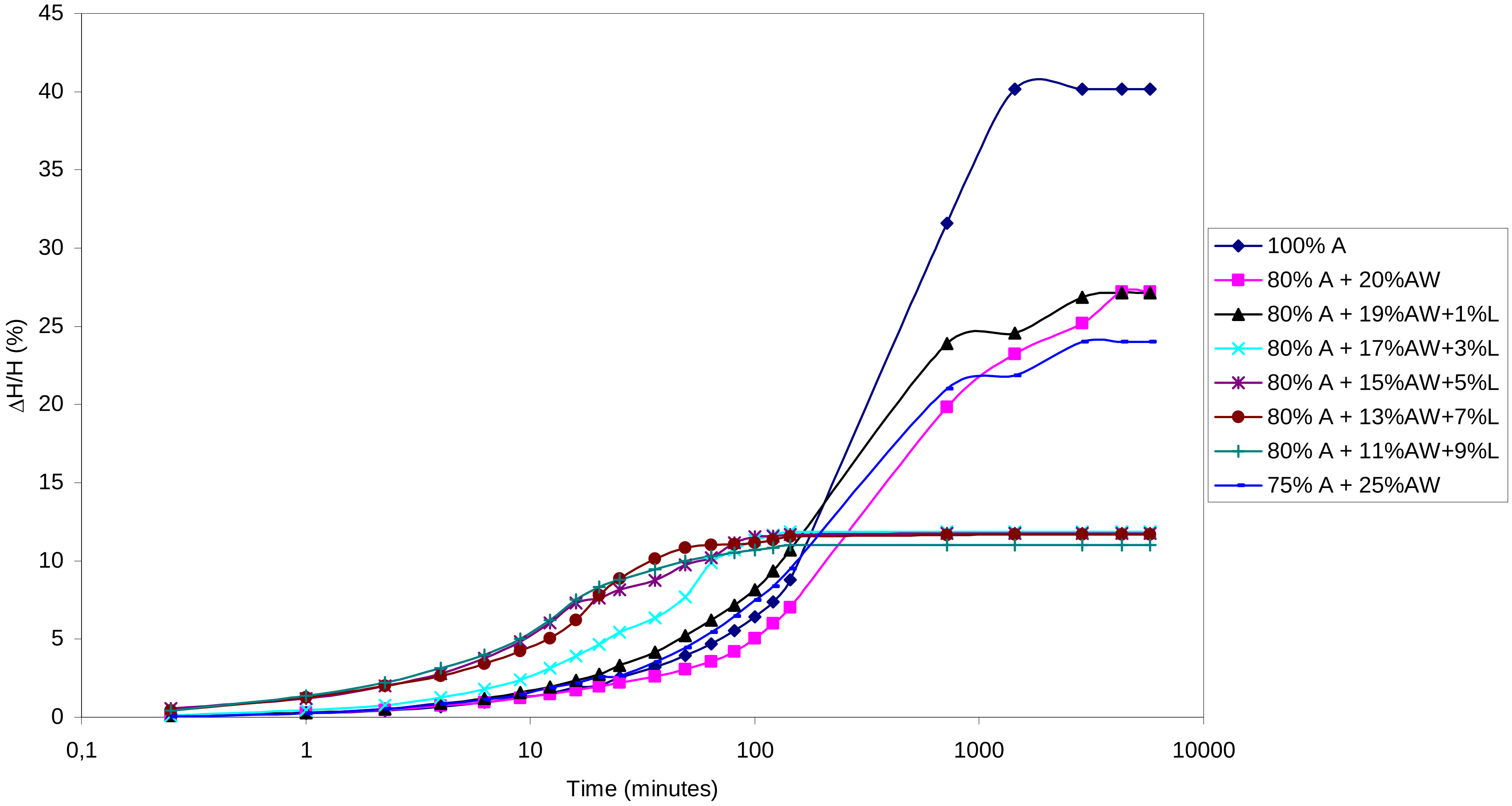

Figure B.4. Swell percentage versus Time relationship for aggregate waste and lime added samples after 28 days curing.

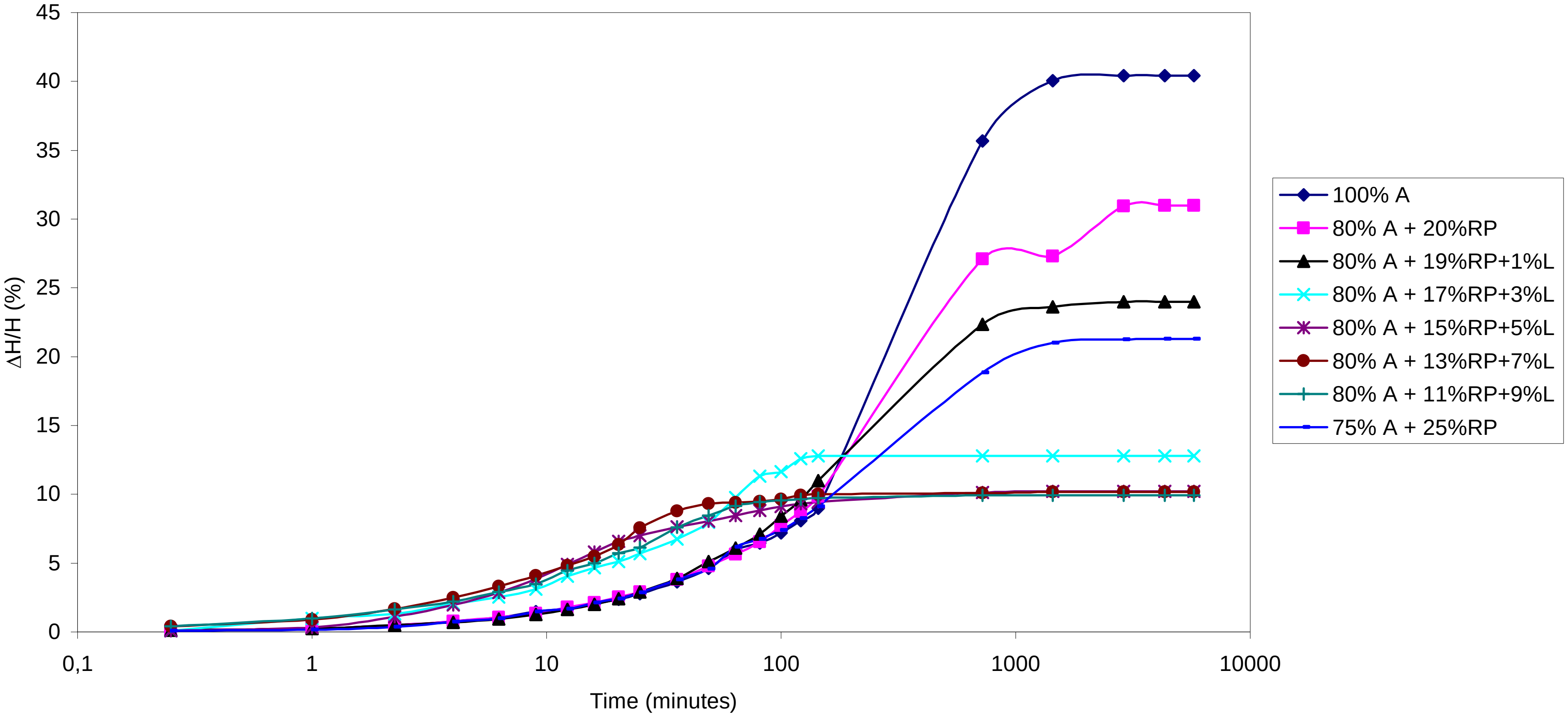

Figure B.5. Swell percentage versus Time relationship for rock powder and lime added samples after 7 days curing.

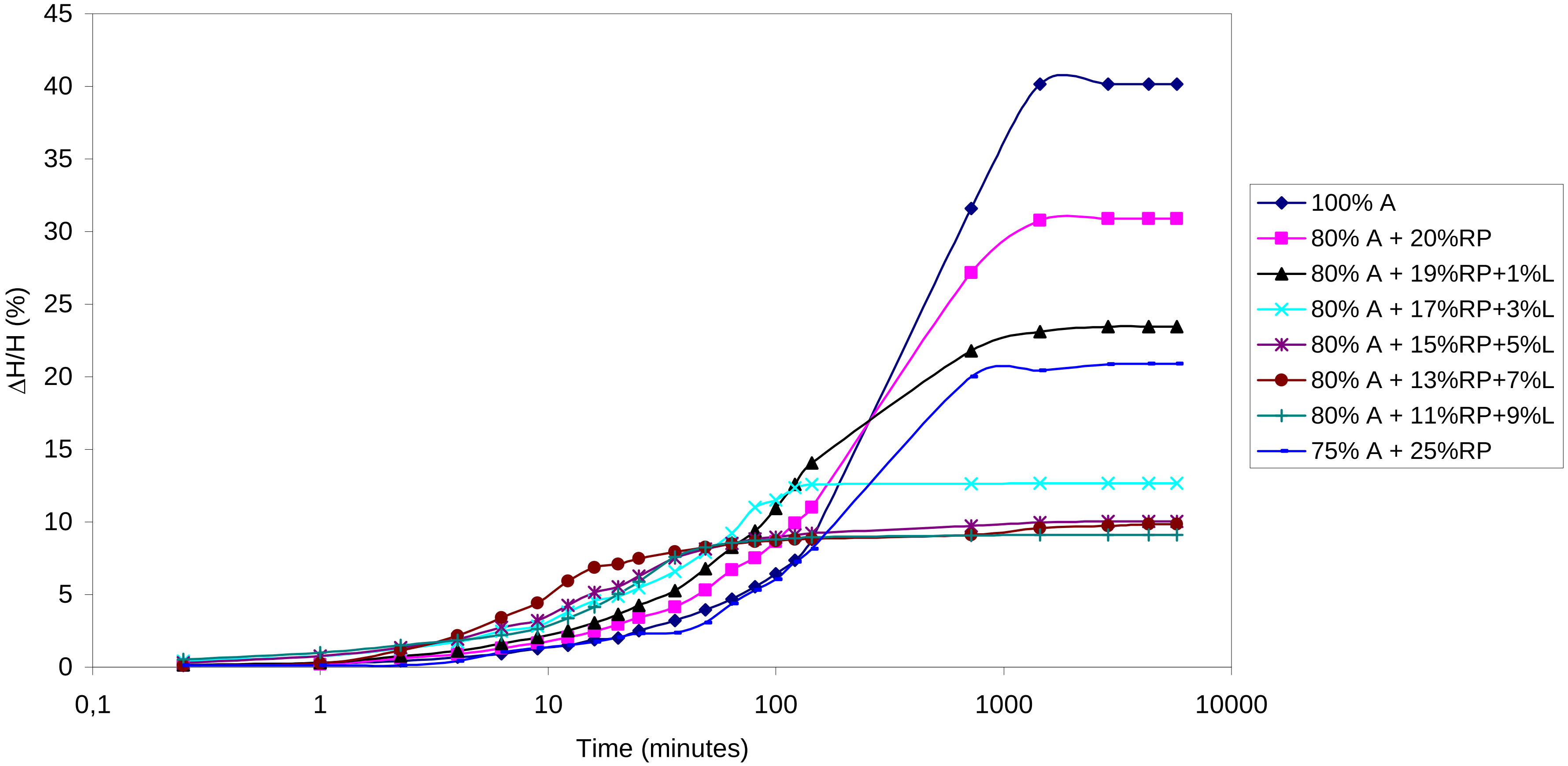

Figure B.6. Swell percentage versus Time relationship for rock powder and lime added samples after 28 days curing.

APPENDIX C

Grain Size Distribution Curves of Sample A, 75 % Sample A + 25%AW, Calculated 75 % Sample A + 25% AW, 100%AW, 75 % Sample A + 25%RP, Calculated 75 % Sample A + 25% RP and 100%RP are shown in Appendix C.

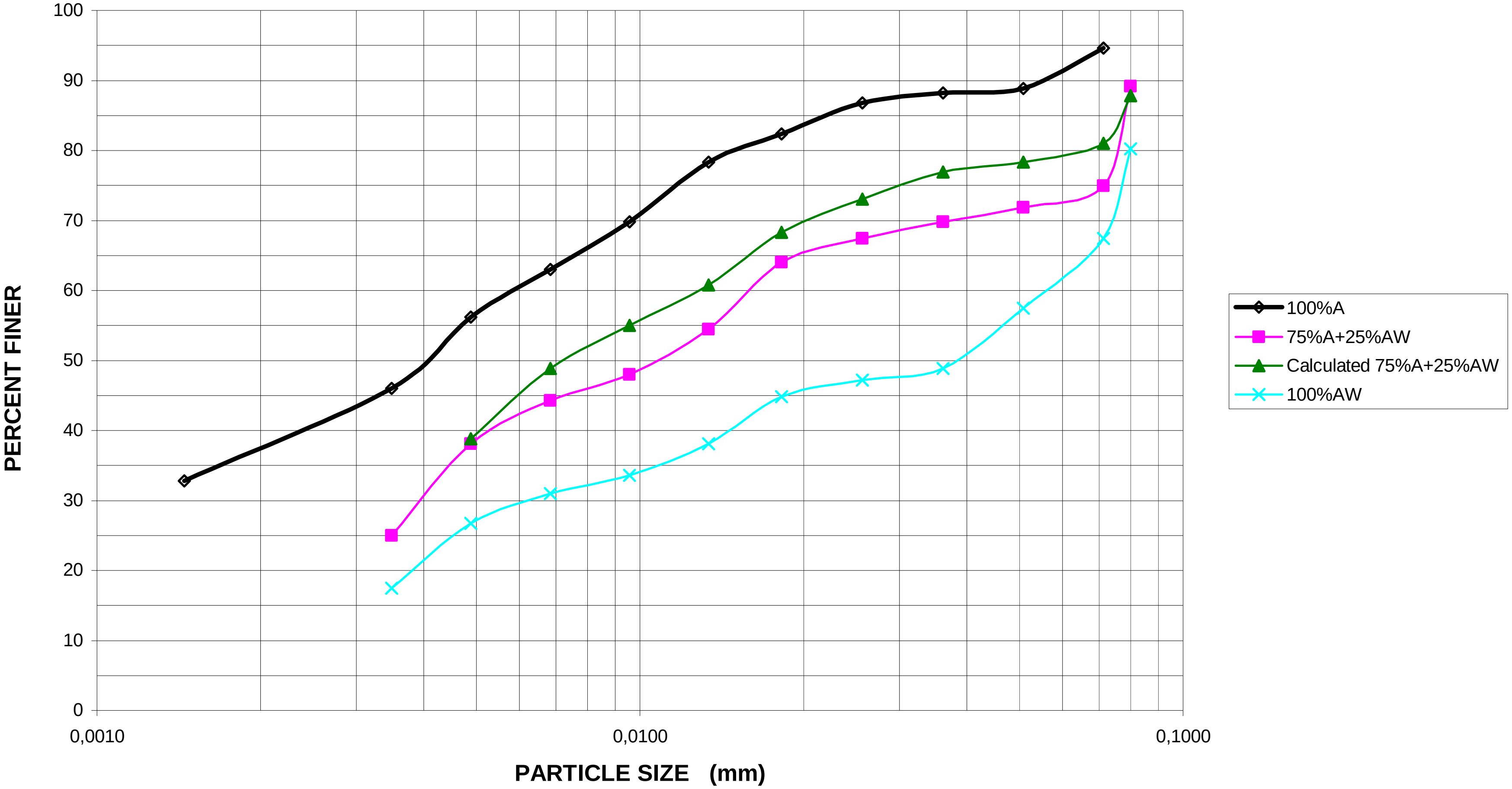

Figure C.1 Grain size Distribution Curves of Sample 100%A, 75%A+25%AW, Calculated 75%A+25%AW and 100%AW.

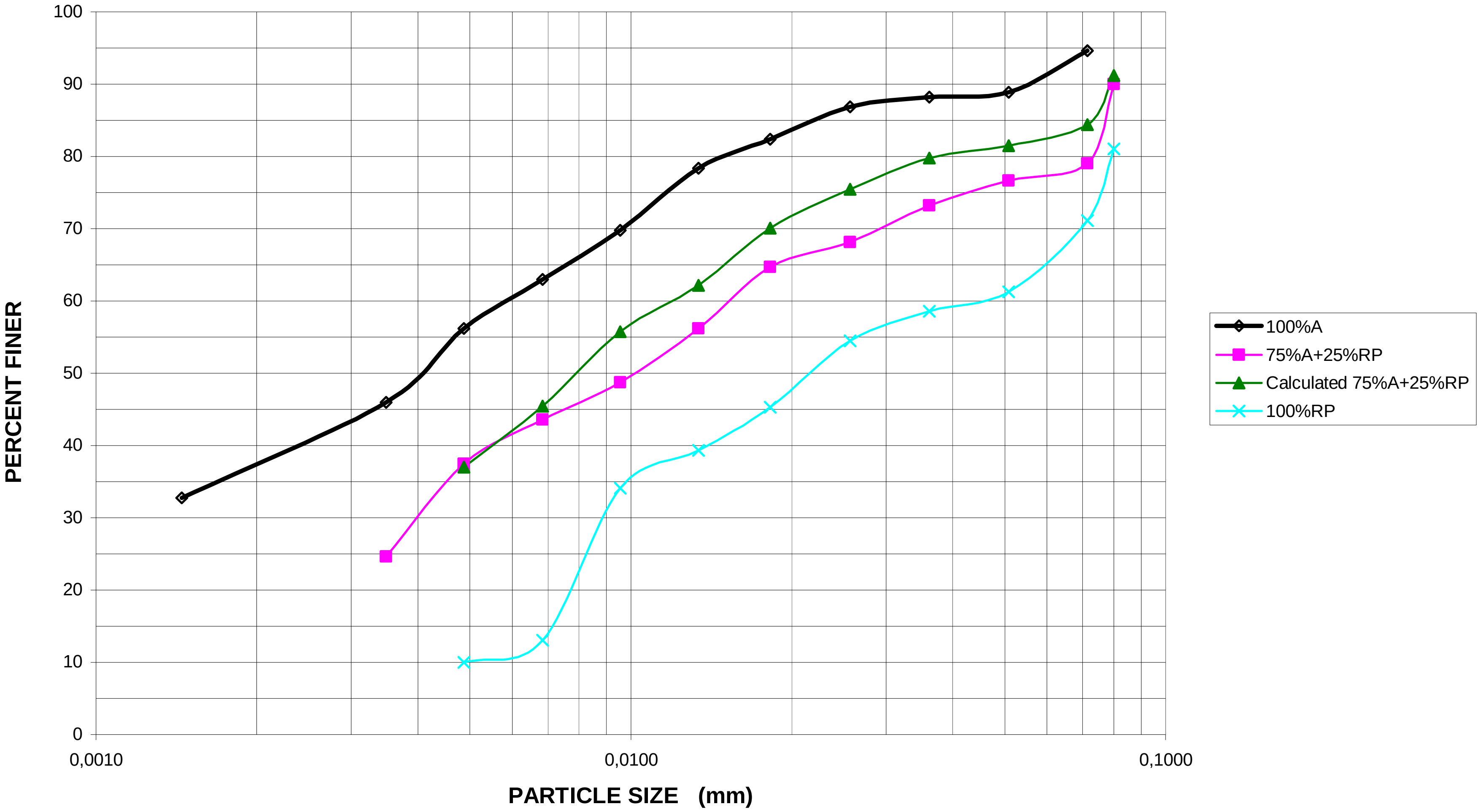

Figure C.2 Grain size Distribution Curves of Sample 100%A, 75%A+25%RP, Calculated 75%A+25%RP and 100%RP.