

STABILIZATION OF EXPANSIVE SOILS USING
WASTE MARBLE DUST

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Approval of the thesis:

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WASTE MARBLE DUST**

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ABSTRACT

STABILIZATION OF EXPANSIVE SOILS USING WASTE MARBLE DUST

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Expansive soils occurring in arid and semi-arid climate regions of the world cause serious problems on civil engineering structures. Such soils swell when given an access to water and shrink when they dry out. Several attempts are being made to control the swell-shrink behavior of these soils. Soil stabilization using chemical admixtures is the oldest and most widespread method of ground improvement. In this study, waste limestone dust and waste dolomitic marble dust, by-products of marble industry, were used for stabilization of expansive soils. The expansive soil is prepared in laboratory as a mixture of kaolinite and bentonite. Waste limestone dust and waste dolomitic marble dust were added to

the expansive soil with predetermined percentage of stabilizer varying from 0 to 30 percent. Grain size distribution, consistency limits, chemical and mineralogical composition, swelling percentage, and rate of swell were determined for the samples. Swelling percentage decreased and rate of swell increased with increasing stabilizer percentage. Also, samples were cured for 7 days and 28 days before applying swell tests. Curing of samples affects swell percentages and rate of swell in positive way.

Key words: Expansive Soil, Soil Stabilization, Swelling Potential, Waste Limestone Dust, Waste Dolomitic Marble Dust

ÖZ

ŞİŞEN ZEMİNLERİN ATIK MERMER TOZU KULLANILARAK STABİLİZASYONU

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Yüksek Lisans, İnşaat Mühendisliği Bölümü

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Şişen zeminler dünyanın kurak ve yarı kurak iklime sahip bölgelerinde yer almakta ve inşaat mühendisliği yapılarında ciddi sorunlara neden olmaktadır. Bu tür zeminler suya maruz kaldığında şişer ve kuruduklarında büzülürler. Bu zeminlerin şişme-büzülme davranışını kontrol edebilmek için bir çok araştırmalar yapılmaktadır. Kimyasal katkı kullanarak zemin stabilizasyonu yapmak, zemin stabilizasyonunun en eski ve en yaygın yöntemidir. Bu çalışmada şişen zeminlerin stabilizasyonu için mermer endüstrisinin atıkları olan kireçtaşı tozu atığı ve dolomitik mermer tozu atığı kullanılmıştır. Şişen zemin laboratuvarında kaolin ve bentonit kullanılarak hazırlanmıştır. Kireçtaşı tozu atığı ve dolomitik mermer tozu atığı şişen zemine önceden belirlenmiş

%0 - %30 arasında deęişen oranda eklenmiştir. Numuneler üzerinde dane dağılımı, kıvam limitleri, kimyasal ve mineralojik bileşim, şişme yüzdesi ve şişme hızı deneyleri yapılmıştır. Katkı yüzdesinin artmasıyla şişme yüzdesi düşmüş ve şişme hızı artmıştır. Ayrıca, k r n şişme üzerindeki etkisini g rebilmek i in şişme deneylerinden  nce numunelere 7 g nl k ve 28 g nl k k r uygulanmıştır. K r n numunelerin şişme yüzdesinin d şmesine ve şişme hızının artmasına olumlu y nde katkısı olmuştur.

Anahtar Kelimeler: Şişen Zemin, Zemin Stabilizasyonu, Şişme Potansiyeli Kire taşı Tozu Atığı, Dolomitik Mermer Tozu Atığı

To My Family

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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

DMD: Waste Dolomitic Marble Dust

G_s: Specific gravity

H: Initial height of the sample

LD: Waste Limestone Dust

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

t₅₀: Time to reach fifty percent swell

CHAPTER 1

INTRODUCTION

Expansive soil deposits occur in the arid and semi arid regions of the world and are problematic to engineering structures because of their tendency to heave during wet season and shrink during dry season (Mishra et al. 2008).

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 (Al-Rawas, 2002). Expansive soils cause more damage to structures, particularly light buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973).

During the last few decades damage due to swelling action has been observed clearly in the semi arid regions in the form of cracking and breakup of pavements, roadways, building foundations, slab-on-grade members, and channel and reservoir linings, irrigation systems, water lines, and sewer lines (Çokça, 2001).

On the other hand, in the world, marble (natural stone) production amount was 21.7 million tons in the year of 1986; however in 1998 this amount increased to 51 million tons (DPT, 2001). Increasing demand for marble product raises the generation of waste marble material. The proportion of marble discharged as waste during block production at the quarries is equal to 40 - 60 % of the overall production volume (Çelik, 1996).

Turkey, due to its location in the Alpine-Himalayan belt, has numerous marble deposits. More than 250 marble types with different colors and patterns have been produced from these deposits and one hundred of these are well known around the world (Çelik and Sabah, 2007). Only, in Afyon, disposed waste marble material is about 125.000 ton/year (Çelik, 1996).

Large pieces of marble waste can be used as embankment or pavement material, and waste marble dust can be used as additives in some industries (paper, cement, ceramic etc.). But, only small portion of the waste marble products is utilized economically, most of them are stored on lands. Increasing of usage fields of waste marble products will eliminate the potentially harmful effects of them on environment and minimize the cost due to storage.

In this study, the suitability of waste marble dust (waste limestone dust and waste dolomitic marble dust) as stabilizers for swelling potential of an expansive soil was studied.

CHAPTER 2

REVIEW ON EXPANSIVE SOILS

Expansive soils, which usually contain the clay mineral montmorillonite, include sedimentary and residual soils, claystones, and shales. In arid and semiarid climates, they exist in a moisture-deficient, unsaturated condition. The expansive nature of soil is most obvious near ground surface where the profile is subject to seasonal, environmental changes (Terzaghi, Peck and Mesri, 1996; Fredlund and Rahardjo, 1993).

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 Clay Mineralogy

The term clay can refer both to a size and to a class of minerals. As a size term, it refers to all constituents of a soil smaller than a particular size, usually 0.002 mm in engineering classifications. As a mineral term, it refers to

specific clay minerals that are distinguished by (1) small particle size, (2) a net electrical charge, (3) plasticity when mixed with water and (4) high weathering resistance (Mitchell and Soga, 2005).

The basic idealized crystalline structural unit of a clay mineral is composed of a silica tetrahedron block and an aluminum octahedron block. Aluminum octahedron block may have Aluminum (Al^{3+}) or magnesium (Mg^{2+}). If only aluminum is present, it is called gibbsite [$\text{Al}_2(\text{OH})_6$]; if only magnesium is present, it is called brucite [$\text{Mg}_3(\text{OH})_6$]. Various clay minerals are formed as these sheets stack on top of each other with different ions bonding them together (Oweis and Khera, 1998). A silica tetrahedron and a silica sheet, also an octahedron and an octahedron sheet are presented in Figure 2.1 and Figure 2.2, respectively. Also, these figures consist of schematic representations of silica and octahedron sheets.

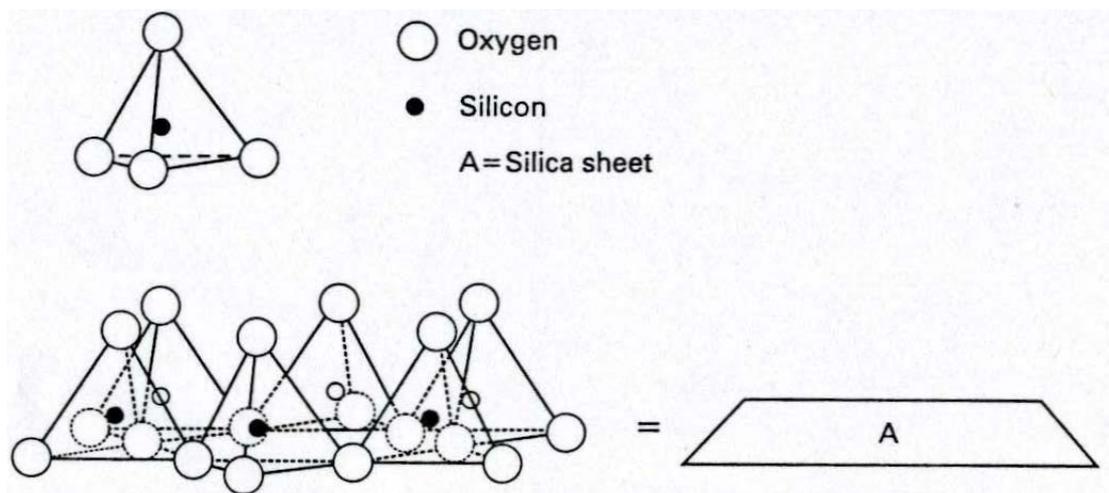


Figure 2.1 A Silica Tetrahedron and a Silica Sheet (after Oweis and Khera, 1998)

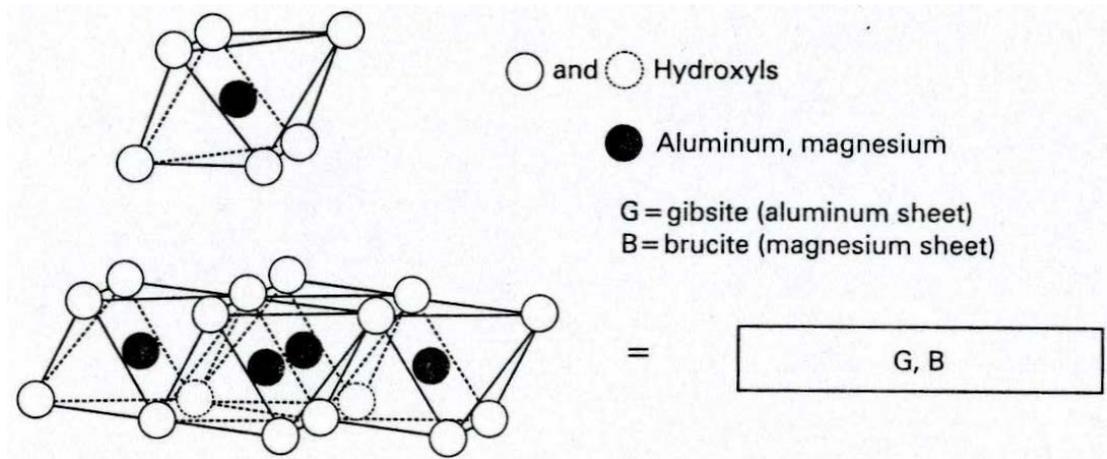


Figure 2.2 An Octahedron and an Octahedron Sheet (after Oweis and Khera, 1998)

Three important structural groups of clay minerals are described for engineering purposes as follows:

Kaolinite group - generally nonexpansive

Mica-like group - includes illites and vermiculites, which can be expansive but generally do not pose significant problems.

Smectite group - includes montmorillonites, which are highly expansive and are the most troublesome clay minerals (Nelson and Miller, 1992).

2.1.1 Kaolinite group

Kaolinite

Kaolinite crystals consist of tetrahedron and octahedron sheets. The bonding between successive layers is by van der Waals forces and hydrogen bonds. The bonding is sufficiently strong that there is no interlayer swelling in the presence of water (Mitchell and Soga, 2005).

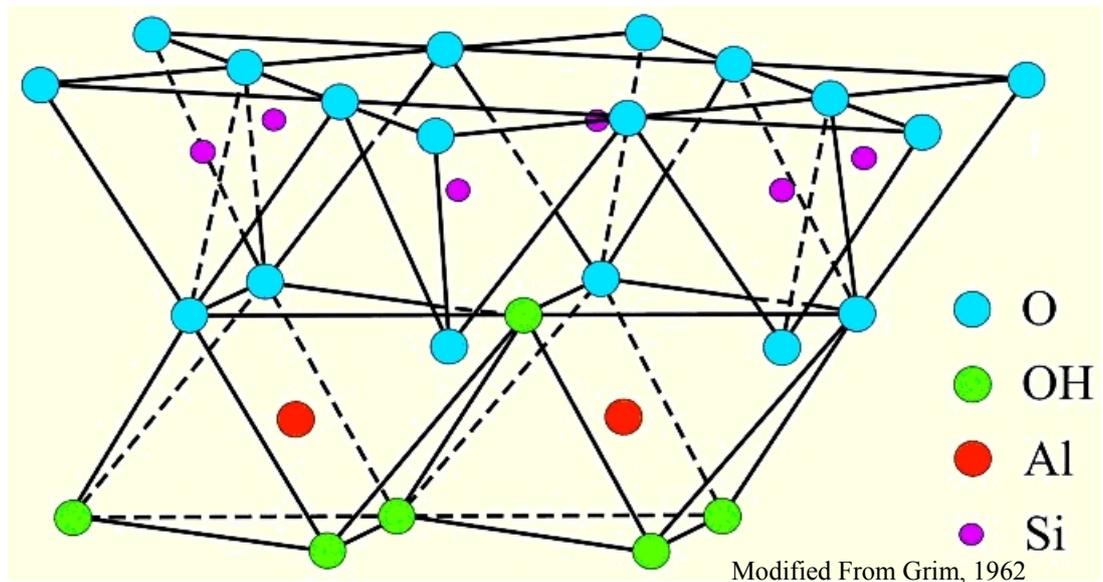


Figure 2.3 Diagrammatic Sketch of the Kaolinite (after USGS, 2001)

2.1.2 Mica-like group

Illite

Illite has a basic structure consisting of a sheet of alumina octahedrons between and combined with two sheets of silica tetrahedrons. In the octahedral sheet there is partial substitution of aluminum by magnesium and iron, and in

the tetrahedral sheet there is partial substitution of silicon by aluminum. The combined sheets are linked together by fairly weak bonding due to (non - exchangeable) potassium ions held between them (Craig, 1997).

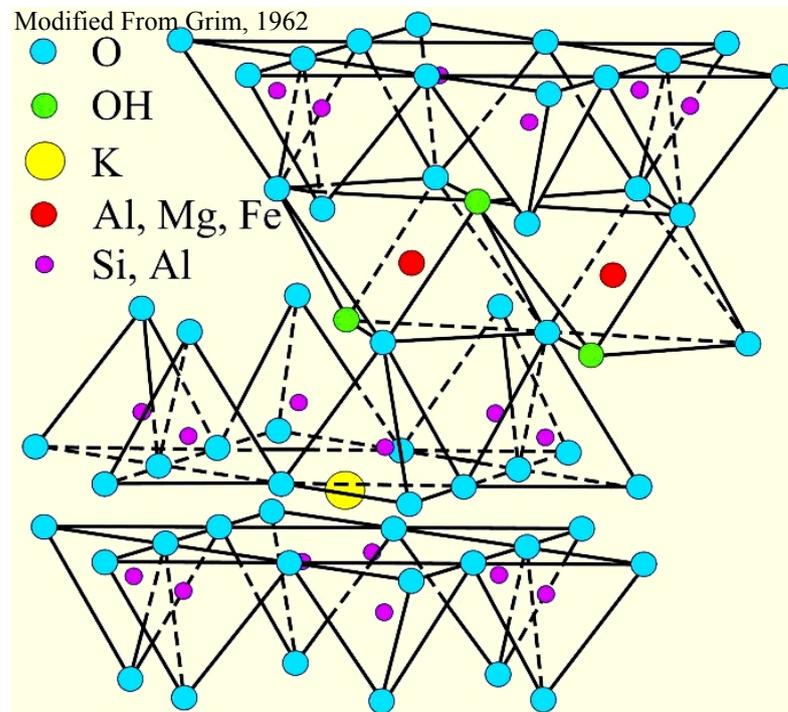


Figure 2.4 Diagrammatic Sketch of the Illite (after USGS, 2001)

2.1.3 Smectite group

Montmorillonite

Montmorillonite is formed from weathering of volcanic ash under poor drainage conditions or in marine waters. The basic building sheets for smectite are the same as for illite except there is no potassium ion present. The space between the combined sheets is occupied by water molecules and exchangeable cations. There is a very weak bond between the combined sheets due to these ions. Considerable swelling of montmorillonite can occur due to additional

water being absorbed between the combined sheets (Craig, 1997; Oweis and Khera, 1998).

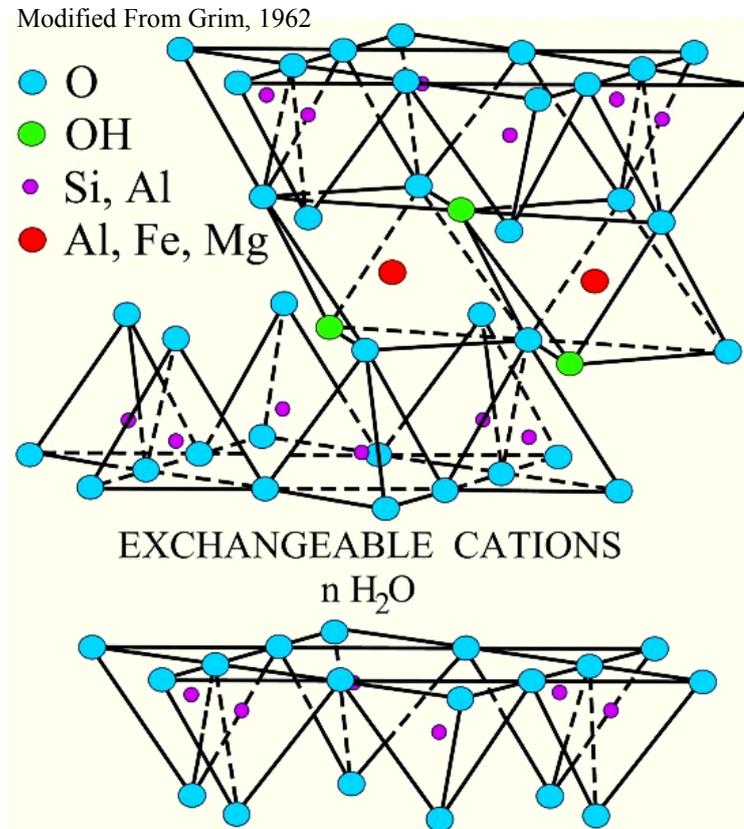


Figure 2.5 Diagrammatic Sketch of the Montmorillonite (after USGS, 2001)

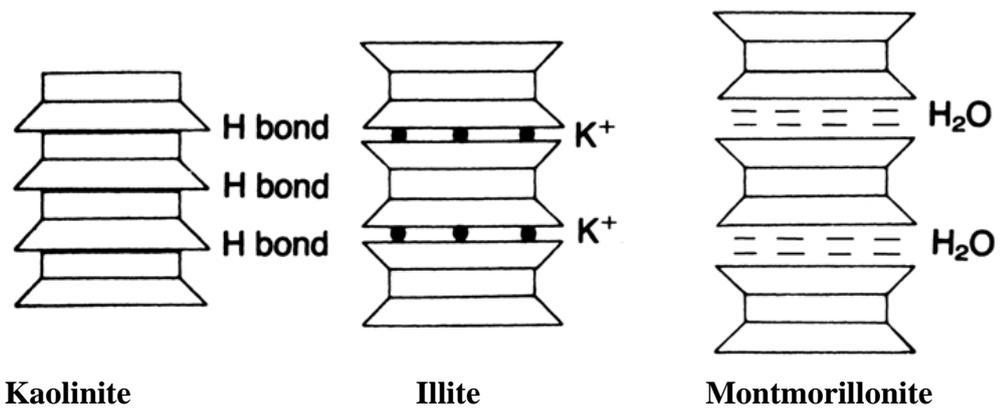


Figure 2.6 Schematic Representations of Clay Minerals (after Craig, 1997)

2.2 Mechanism of Swelling

Swelling of clay minerals is directly related with diffused double layer and cation exchange capacity of them.

2.2.1 Double Layer of Clay Minerals

The negatively charged clay particle surface and the concentration of positive ions in solution adjacent to the particle form what is referred to as a diffuse double layer or DDL (Bohn et al. 1985). Overlapping DDLs between clay particles generate interparticle repulsive forces or microscale “swelling pressures”. Interaction of the DDL and, hence, swelling potential, increases as the thickness of the DDL increases (Mitchell, 1976). The thickness of DDL is associated with valence of cations, concentration of cations, temperature and pH.

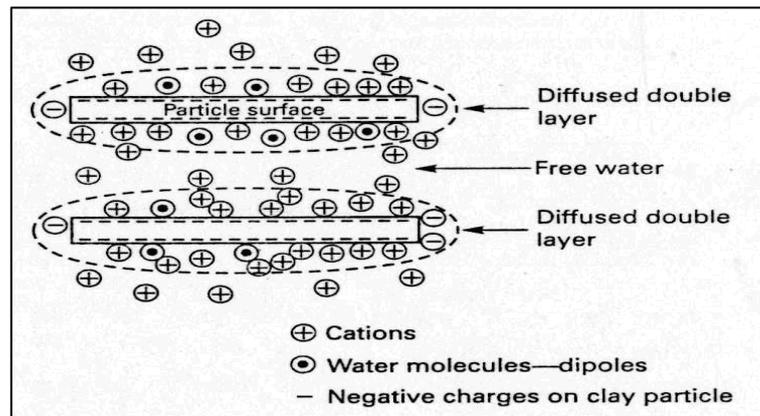


Figure 2.7 Double Layer of Clay Minerals (after Oweis and Khera, 1998).

a) Effect of valence of cations: The lower valences of cations results in increase in DDL thickness. Thus, for the same soil mineralogy, more swelling would occur in a sample having exchangeable sodium (Na^+) cations than in a sample with calcium (Ca^{2+} or magnesium Mg^{2+}) cations (Nelson and Miller, 1992).

b) Effect of concentration of cations: The high concentration of cations near the surface of clay particle creates a repulsive force between the diffuse double layer system (Chen, 1975). In general, a thicker DDL and greater swelling are associated with lower cation concentrations (Mitchell, 1976).

c) Effect of temperature: An increase in temperature cause an increase in DDL thickness, thus temperature change has effect on strength, compressibility and swelling of soils (Mitchell and Soga, 2005).

d) Effect of pH: Hydroxyls (OH^-) are exposed on the surfaces and edges of clay particles. The tendency for hydroxyls to dissociate in water, " $\text{SiOH} \rightarrow \text{SiO} + \text{H}^+$ " is strongly influenced by pH. The higher pH, the greater is the tendency for H to go into solution, and the greater the effective negative charge of the particle. Alumina, exposed at the edges of clay particles, is amphoteric (capable of functioning either as an acid or a base), and it ionizes positively at low pH and negatively at high pH. As a result, positive diffuse layers can develop at the edges of some clay particles in an acid environment which promotes a positive edge to negative surface interaction, often leading to flocculation from suspension (Mitchell and Soga, 2005).

2.2.2 Cation Exchange Capacity (CEC)

Cations that neutralize the net negative charge on the surface of soil particles in water are readily exchangeable with other cations. The exchange reaction depends mainly on the relative concentrations of cations in the water and also on the electrovalence of cations (Terzaghi, Peck and Mesri, 1996). The cation exchange capacity is the quantity of exchangeable cations required to balance the negative charge on the surface of the clay particles. CEC is expressed in milliequivalents per 100 grams of dry clay (Nelson and Miller, 1992).

Table 2.1 CEC of Principle Clay Minerals (modified from Terzaghi, Peck and Mesri, 1996)

MINERAL	CEC (meq/100g)
Kaolinite	3 - 10
Illite	20 - 30
Montmorillonite	80 - 120

2.2.3 Stages of swelling

The swelling phenomenon has two basic mechanisms:

1) Interparticle or intercrystalline swelling, effective for all kinds of clay minerals (Fig. 2.8). 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 (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 (Fig. 2.8) (Popescu, 1986).

2.3 Factors Influencing Swelling

The swell potential of a clayey soil may be affected by either the soil properties influencing the nature of the internal force field, the environmental factors those may change the internal force system or the state of stress present on the soil. These factors are summarized in Table.2.2, Table.2.3 and Table.2.4. Some physical factors such as initial water content, initial density, amount and type of compaction also influence the swell potential and swell parameters of soils.

HYDRATION VOLUME CHANGES

interparticle or intercrystalline

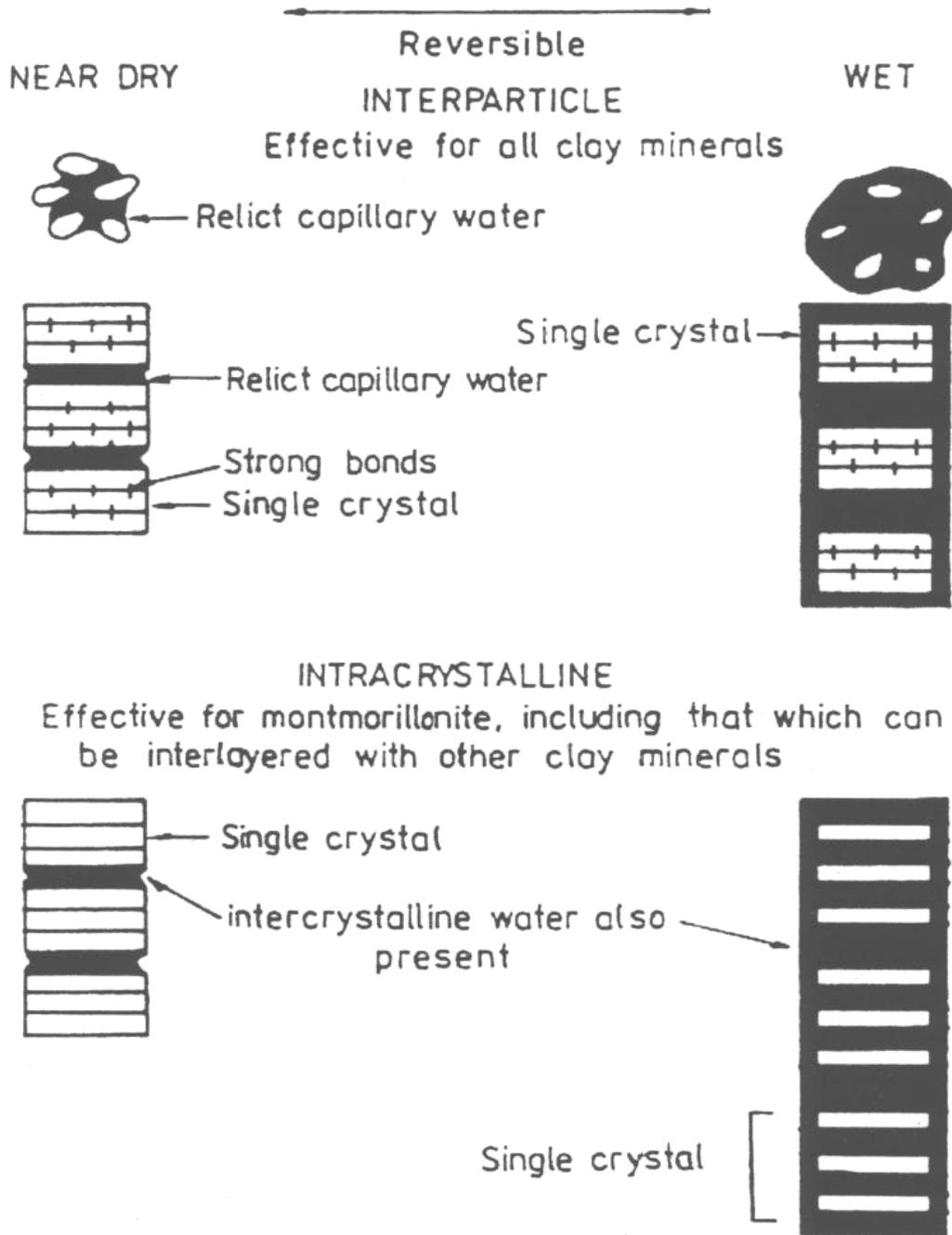


Figure 2.8 Mechanism of Swelling (after Popescu, 1986).

Upon the aforementioned factors, the mineralogical composition and clay content is obviously the most important factor influencing the extent of swell of clayey soils. From the previous investigations, it is a well known fact that; the content of swell or swell potential of a clayey soil varies with the type of clay mineral present in the soil mass (Lambe and Whitman, 1969).

Table 2.2 Soil Properties Influencing Swell Potential (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
Clay Mineralogy	Clay minerals which typically cause soil volume changes are montmorillonites, vermiculates, and some mixed layer minerals. Illites and Kaolinites are frequently in expansive, but can cause volume changes when particle sizes are extremely fine
Soil Water Chemistry	Swelling is repressed by increased cation concentration and increased cation valence. For example, Mg^{2+} cations in the soil water would result in less swelling than Na^+ cations
Soil Suction	Soil suction is an independent effective stress variable, represented by the negative pore pressure in unsaturated soils. Soil suction is related to saturation, gravity, pore size and shape, surface tension, and electrical and chemical characteristics of the soil particles and water.
Plasticity	In general, soils that exhibit plastic behaviour over wide ranges of moisture content and that have high liquid limits have greater potential for swelling and shrinkage. Plasticity is an indicator of swell potential
Soil Structure and Fabric	Flocculated clays tend to be more expansive than dispersed clays. Cemented particles reduce swell. Fabric and structure are altered by compaction at high water content or remolding. Kneading compaction has been shown to create dispersed structures with lower swell potential than soils statically compacted at lower water contents
Dry Density	Higher densities usually indicate closer particle spacing, which may mean greater repulsive forces between particles and larger swelling potential

Table 2.3 Environmental Factors Affecting Swell Potential (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
Initial Moisture Content	A desiccated expansive soil will have high affinity for water, or higher suction than the same soil at higher water content, lower suction. Conversely, a wet soil profile will loose water more readily on exposure to drying influences, and shrink more than a relatively dry initial profile. The initial soil suction must be considered in conjunction with the expected range of final suction conditions
Moisture Variations	Changes in moisture in the active zone near the upper part of the profile primarily define heave, it is in those layers that the widest variation in moisture and volume change will occur
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 source of moisture and fluctuating water tables contribute to moisture
Drainage	Surface drainage features, such as ponding around a poorly graded house foundation, provide sources of water at the surface; leaky plumbing can give the soil access to water at greater depth
Vegetation	Trees, shrubs, and grasses 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 2.4 Stress Conditions Affecting Swell 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. Swell pressures can increase on aging of compacted clays, but amount of swell under light loading has been shown to be unaffected by aging. Repeated wetting and drying tend to reduce swell in laboratory samples, but after a certain number of wetting-drying cycles, swell is unaffected.
In situ Conditions	The initial stress state in a soil must be estimated in order to evaluate the probable consequences of loading the soil mass and/or altering the moisture environment therein. The initial effective stresses can be roughly determined through sampling and testing in a laboratory, or by making in-situ measurements and observations
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 movements. Greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. Less movement will occur if expansive soil is overlain by nonexpansive material or overlies bedrock at shallow depth

2.4 Oedometer Methods to Determine Swell Properties

The most satisfactory and convenient method of determining the swelling properties of an expansive clay is by direct measurement. Direct measurement of expansive soils can be achieved by the use of the conventional one-dimensional consolidometer (Chen, 1975).

According to ASTM D4546 - 03 (Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils), test methods for swell properties can be grouped into three; Method A, Method B and Method C. Initially, the terminology of experiments is presented.

Swell, L = Increase in elevation or dilation of soil column following absorption of water.

Free swell, % = Percent heave, $\Delta h/h \times 100$ following absorption of water at the seating pressure.

Primary swell, L = 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 (Fig. 2.9).

Secondary swell, L = An arbitrary long-term swell usually characterized as the linear portion of a one dimensional change-logarithm of time plot following completion of short-term or primary swell (Fig. 2.9).

Swell Pressure, kPa: A pressure preventing the specimen from swelling.

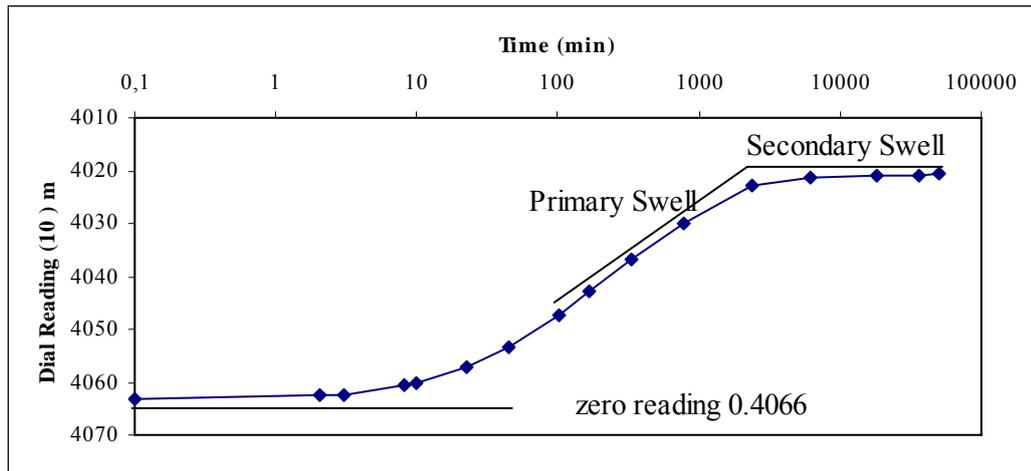


Figure 2.9 Time - Swell Curve (modified from ASTM, 1999)

2.4.1 Method A

After taking initial deformation readings, the seating pressure is applied on specimen and the specimen inundated to swell vertically. While swelling of specimen, deformations are recorded at 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 min and 1, 2, 4, 8, 24, 48, and 72 hours (Fig. 2.9). After primary swell is complete (Fig.2.10, step 3-4), a vertical pressure of approximately 5, 10, 20, 40, 80, etc., kPa is applied till the specimen is recompressed to its initial void ratio and original height complete (Fig.2.10, step 4-6).

Method A may 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 min of placing the seating pressure and securing the zero deformation reading. Then, the deformation is read within 5 min and the vertical stress is removed, except for the seating pressure. The deformation is recorded within 5 min after removal of σ_1 , the specimen is inundated, and the test continues as in stated above (Fig.2.10, step 1-3). Method A measures (a) the free swell, (b)

percent heave for vertical confining pressures up to the swell pressure, and (c) the swell pressure.

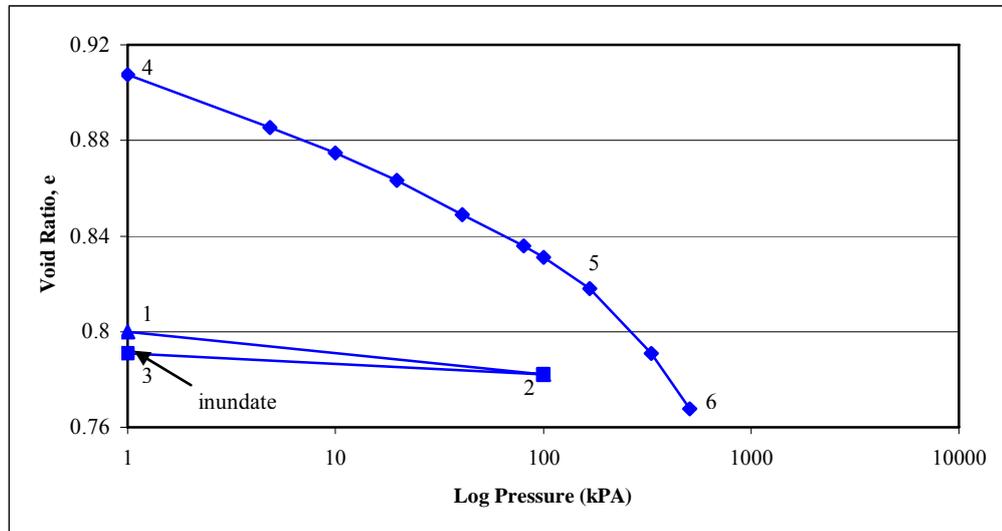


Figure 2.10 Void Ratio - Log Pressure Curve for Method A (modified from ASTM, 1999)

2.4.2 Method B

After applying a vertical pressure exceeding the seating pressure within 5 min of placing the seating pressure, the deformation is read within 5 min of placing the vertical pressure (Fig. 2.11, step 1-2). The specimen is inundated immediately after the deformation is read (Fig. 2.11, step 2) and deformation is recorded after elapsed times similar to Method A until primary swell is complete (Fig. 2.11, step 2-3). After primary swell is complete, vertical pressures of are applied as stated in Method A. Method B 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.

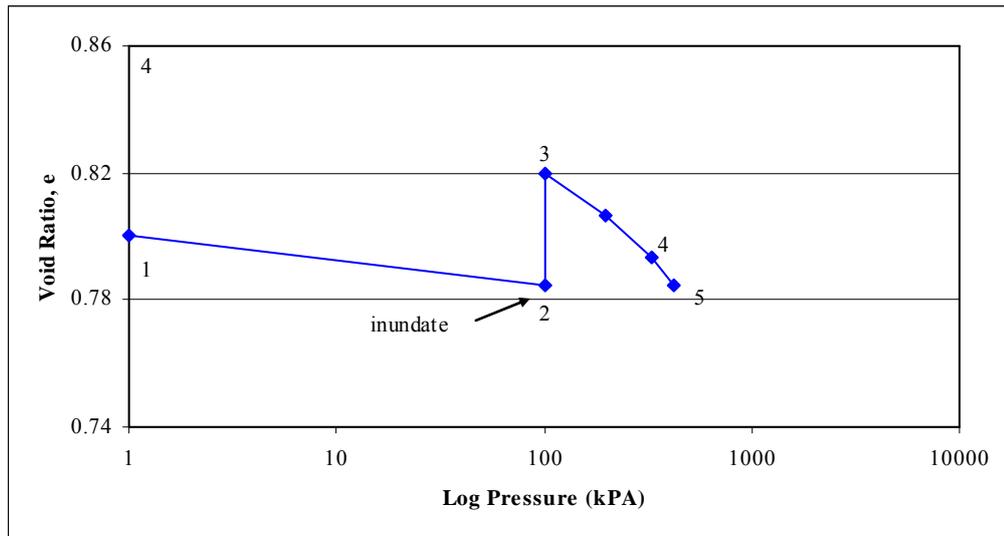


Figure 2.11 Void Ratio – Log Pressure Curve for Method B (modified from ASTM, 1999)

2.4.3 Method C

An initial stress, σ_1 , is applied, equivalent to the estimated vertical in situ pressure or swell pressure within 5 min after placement of the seating pressure. The deformation within 5 min is read after placing σ_1 (Fig. 2.12, step 1), and immediately the specimen is inundated with water (Fig. 2.12, step 2). Increments of vertical stress as needed to prevent swell is applied and final load is recorded (Fig. 2.12, step 3). The specimen is loaded vertically as in Method A (Fig. 2.12, step 4-7). The rebound curve following consolidation is then, determined (Fig. 2.10, after step 7). Method C measures (a) the swell pressure, (b) preconsolidation pressure, and (c) percent heave or settlement within the range of applied vertical pressures.

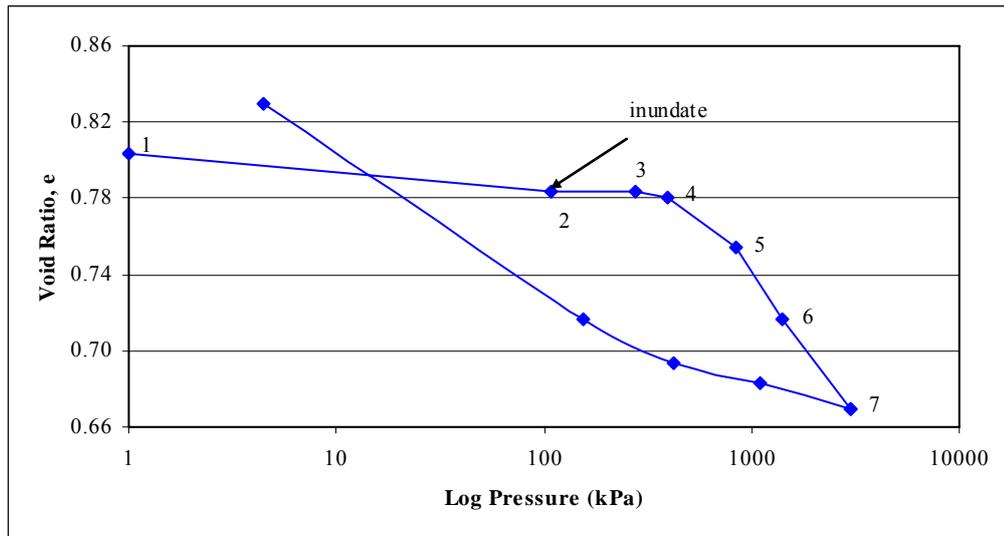


Figure 2.12 Void Ratio - Log Pressure Curve for Method C (modified from ASTM, 1999)

CHAPTER 3

SOIL STABILIZATION

3.1 Chemical Stabilization

Soil stabilization using chemical admixtures is the oldest and most widespread method of ground improvement. Chemical stabilization is mixing of soil with one of or a combination of admixtures of powder, slurry, or liquid for the general objectives of improving or controlling its volume stability, strength and stress-strain behavior, permeability, and durability (Winterkorn and Pamukçu, 1990).

Soil improvement by means of chemical stabilization can be grouped into three chemical reactions; cation exchange, flocculation - agglomeration, pozzolanic reactions.

3.1.1 Cation Exchange

The excess of ions of opposite charge (to that of the surface) over those of like charge present in the diffuse double layer are called exchangeable ions. These ions can be replaced by a group of different ions having the same total charge by altering the chemical composition of the equilibrium electrolyte solution (Winterkorn and Pamukçu, 1991).

Negatively charged clay particles adsorb cations of specific type and amount. The ease of replacement or exchange of cations depends on several factors, primarily the valence of the cation. Higher valence cations easily replace cations of lower valence. For ions of the same valence, the size of the hydrated ion becomes important; the larger the ion, the greater the replacement power. If other conditions are equal, trivalent cations are held more tightly than divalent and divalent cations are held more tightly than monovalent cations (Mitchell and Soga, 2005). A typical replaceability series is



The exchangeable cations may be present in the surrounding water or be gained from the stabilizers.

An example of the cation exchange (Sivapullah, 2006);



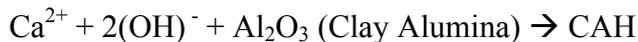
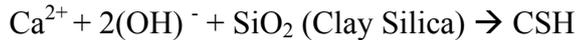
The thickness of the diffused double layer decreases as replacing the divalent ions (Ca^{2+}) from stabilizers with monovalent ions (Na^+) of clay. Thus, swelling potential decreases.

3.1.2 Flocculation and Agglomeration

Cation exchange reactions result in the flocculation and agglomeration of the soil particles with consequent reduction in the amount of clay-size materials and hence the soil surface area, which inevitably accounts for the reduction in plasticity (Terzaghi and Peck, 1967). Due to change in texture, a significant reduction in the swelling of the soil occurs.

3.1.3 Pozzolanic Reactions

Time depending pozzolanic reactions play a major role in the stabilization of the soil, since they are responsible for the improvement in the various soil properties (Show et al., 2003). Pozzolanic constituents produces calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH).



The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure thus, strength of the soils increases (Hadi et al, 2006; Sivapullaiah, 2006).

3.2 Lime Stabilization

The chemical theory involved in the lime reaction is complex (Thompson, 1966, 1968). The main reactions include cation exchange, flocculation and pozzolanic reactions (cited in Nelson and Miller, 1992). The cation exchange and flocculation concepts, primarily effects of stabilizer, were stated in Section 3.1.1 and 3.1.2, respectively. Also, in Section 3.1.3, the pozzolanic reactions for lime stabilized soils were presented. These three stabilization steps are valid for stabilization of expansive soils using waste limestone dust and waste dolomitic marble dust.

3.3. Marble and Production of Waste Marble Dust

Stone-masons often apply the term marble to any rock which can be easily polished (Oates, 1998). Limestones, schistes, travertines or even granites can be considered as marble in the business world (Onargan et al., 2005).

Waste marble dust produced from marble plants can be either of these natural stones' dust. Thus, in order to distinguish the stabilizers used, stabilizers were named as waste limestone dust and waste dolomitic marble dust in this study.

3.3.1 Limestone

Limestone is a sedimentary rock composed principally of calcium carbonate, or of that with dolomite (Kemp, 1965). Calcium carbonate rocks are formed by re-crystallization of preexisting lime sediments (Dietrich and Skinner, 1979). Due to its color, durability and application possibilities, limestone has wide application field. In this study, waste limestone dust used consists of mainly calcite.

3.3.2 Marble (Real Marble)

Marble or real marble is a metamorphic rock that consists predominantly of calcite and/or dolomite (cited in Dietrich and Skinner, 1979). Marble may be considered as metamorphosed limestone (i.e. limestone which has been fully re-crystallized and hardened under hydrothermal conditions) (Oates, 1998). In this study waste dolomitic marble dust was used.

3.3.3 Production of Waste Marble Dust

The production of fine particles (<2 mm) while cutting marble is one of the major problems for the marble industry. When 1 m³ marble block is cut into 2 cm thick slabs, the proportion of fine particle production is approximately 25 % (Kun, 2000).

While cutting of marble blocks water is used as cooler. But, the fine particles can be easily dispersed after losing humidity, under atmospheric

conditions, such as wind and rain. Thus, fine particles can cause more pollution than other forms of marble waste (cited in Çelik and Sabah, 2007).



Figure 3.1 Views from waste marble dust disposed sites (after Yıldız, 2008)

3.4 Soil Stabilization Using Waste Marble Dust

Extensive literature is available on soil improvement by the application of additives, notably cement and lime. Lately, many researchers have reported on additives that could substitute lime as a soil modifier. Such materials include fly ash (Çokça, 1999; Indraratna et al. 1991, 1995), rice husk (Muntohar, 1999); (Muntohar and Hantoro 2000), marble dust (Okagbue and Onyeobi, 1999), and limestone ash (Okagbue and Yakubu, 2000) (Cited in Okagbue, 2007).

Many researchers (Çelik and Sabah, 2007; Zorluer and Usta, 2003; Oates, 1998; Almedia et al., 2007; Tegethoff, 2001) have reported that marble has very high lime (CaO) content up to 55 % by weight. Thus, stabilization characteristics of waste limestone dust and waste dolomitic marble dust is mainly due to their high lime (CaO) content.

CHAPTER 4

EXPERIMENTAL STUDY

4.1. Purpose

The purpose of this experimental study is to investigate the effects of the addition of waste limestone dust and waste dolomitic marble dust on Atterberg limits, grain size distribution, swell percentage, and rate of swell of an expansive soil sample; and also, to investigate the effect of curing on swell percentage and rate of swell of an expansive soil stabilized with waste limestone dust and waste dolomitic marble dust.

4.2. Material

In this study, bentonite, kaolinite, waste limestone dust and waste dolomitic marble dust were used.

Bentonite: Bentonite (Na - Montmorillonite) was product of Karakaya Bentonite Factory in the form of fine sized grains.

Kaolinite: Kaolinite was product of ESAN Industrial Minerals Company of Eczacıbaşı. Kaolinite was sieved through # 40 sieve before usage.

Waste limestone dust: Waste limestone dust was obtained from ÇMK Marble Co Inc. located in Ankara (Fig. 4.1). Waste limestone dust was passed through # 40 sieve before usage. Specific gravity of this material was determined as 2.68.

Waste dolomitic marble dust: Waste dolomitic marble dust was obtained from ÇMK Marble Co Inc. located in Ankara (Fig. 4.1). Waste dolomitic marble dust was passed through # 40 sieve before usage. Specific gravity of this material was determined as 2.80.



(Limestone Dust)



(Dolomitic Marble Dust)

Figure 4.1 Views from stabilizers

The chemical analysis of waste limestone dust and waste dolomitic marble dust were done by METU Central Laboratory and the results are presented in Table 4.1. Also, X-Ray diffraction patterns were analyzed by General Directorate of Mineral Research and Exploration (MTA). The results of X-Ray analysis results are presented in Table 4.2 and in Fig.4.2 and Fig.4.3.

Table 4.1 Chemical Analysis of the Waste Limestone Dust and Waste Dolomitic Marble Dust

Element	Chemical Composition (%)	
	Waste limestone dust	Waste dolomitic marble dust
C	41.5	37.9
O	47.6	47.0
Ca	10.9	8.5
Mg	-	6.6

Table 4.2 Mineral Composition of the Waste Limestone Dust and Waste Dolomitic Marble Dust

Mineral	Waste limestone dust	Waste dolomitic marble dust
Calcite	+	+
Dolomite	-	+

+: Present; -: Not Present

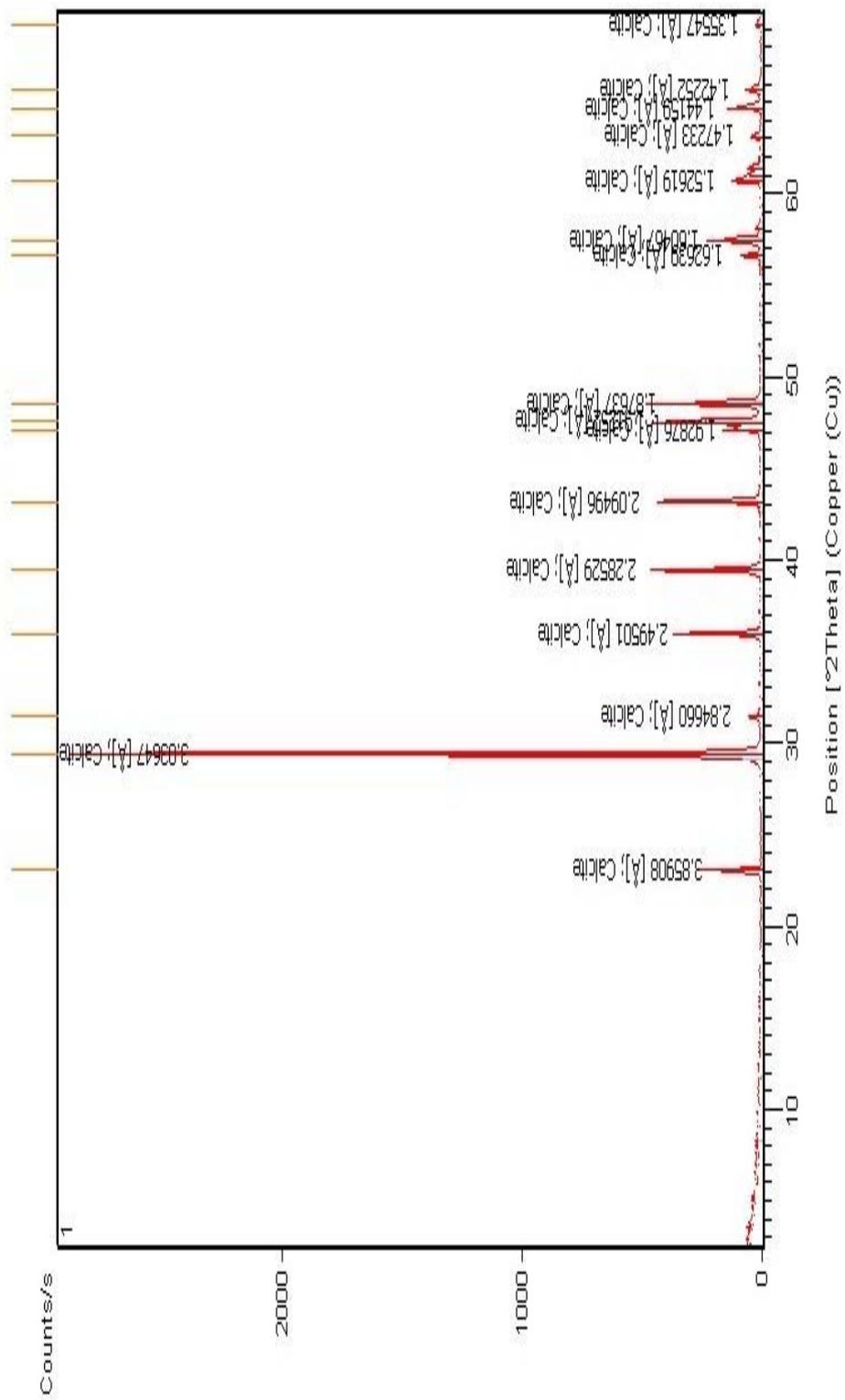


Figure 4.2 X-Ray Diffraction Patterns for Waste Limestone Dust

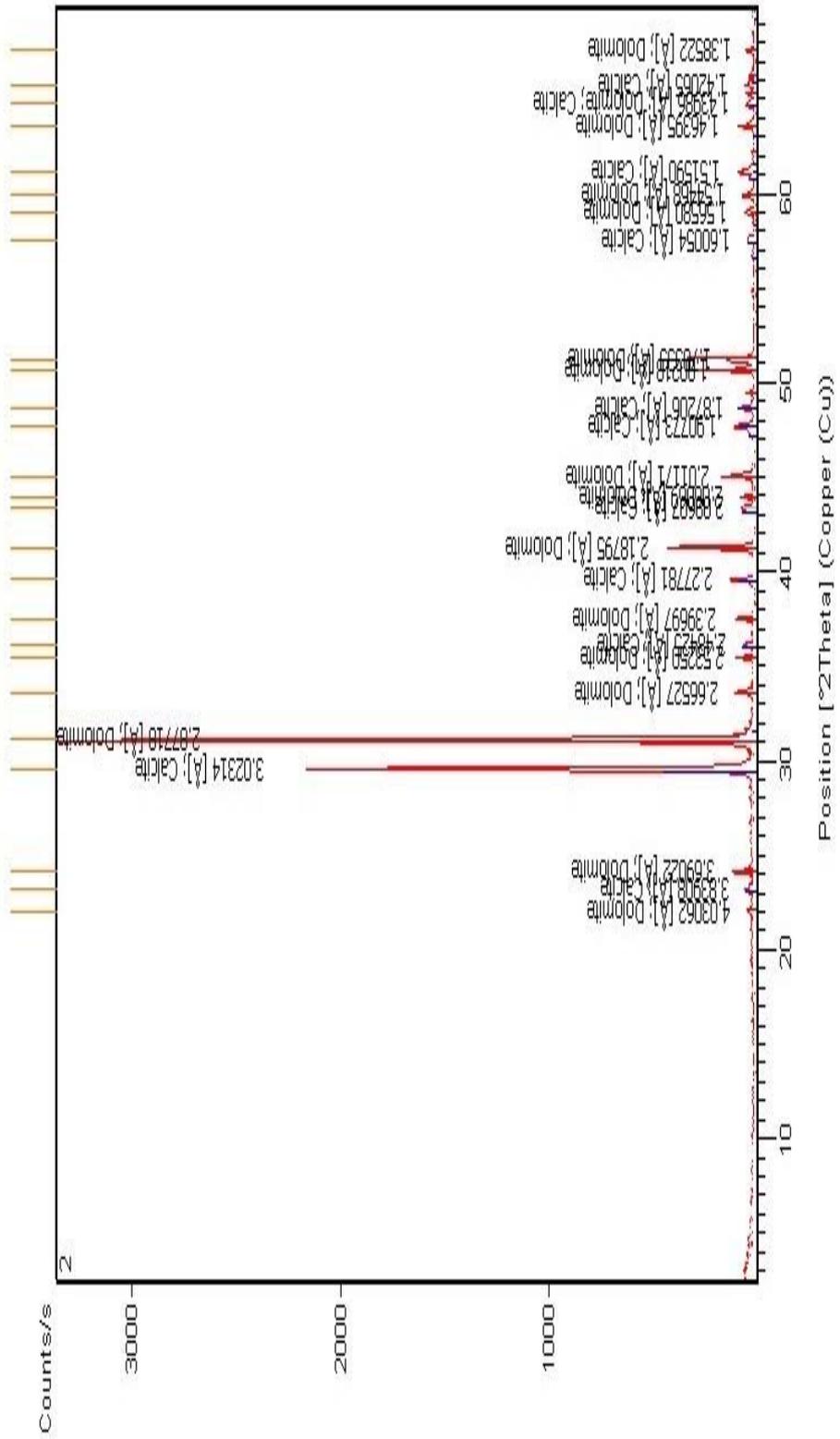


Figure 4.3 X-Ray Diffraction Patterns for Waste Dolomitic Marble Dust

4.3. Preparation of the Test Samples

In nature, expansive soils are widely present. However, possible non-homogeneity or disturbance of these soils may prevent to show actual effects of the stabilizers. Thus, an artificially expansive soil sample (Sample A) was prepared using kaolinite and bentonite in laboratory.

By dry mass, Sample A was composed of 85% kaolinite and 15% bentonite. In the beginning of the preliminary studies, waste limestone dust and waste dolomitic marble dust as stabilizer, were pre-tested and the results showed that these materials could be considered as stabilizing agents for Sample A.

Each sample was prepared by addition of waste limestone dust or waste dolomitic marble dust to Sample A with different percentages to obtain a sample with predetermined percentage of stabilizer varying from 0 to 30 percent (by dry weight of the sample).

Firstly, kaolinite, bentonite, waste limestone dust and waste dolomitic marble dust were oven-dried at 50° C for 1 day. Then, all materials were sieved through # 40 sieve. For each sample, the predetermined amount of material was mixed with trowel. Then, to mix well this fine grained soil samples thoroughly; the materials were sieved two times through # 30 sieve. Each time, only 150 g sample were prepared to make particles distribute homogenously. Then, water was added. The amount of water was 10 % of mass of dry mixture (15 g). Lastly, the mixture was mixed with a trowel and passed through # 30 sieve (Figure 4.4).

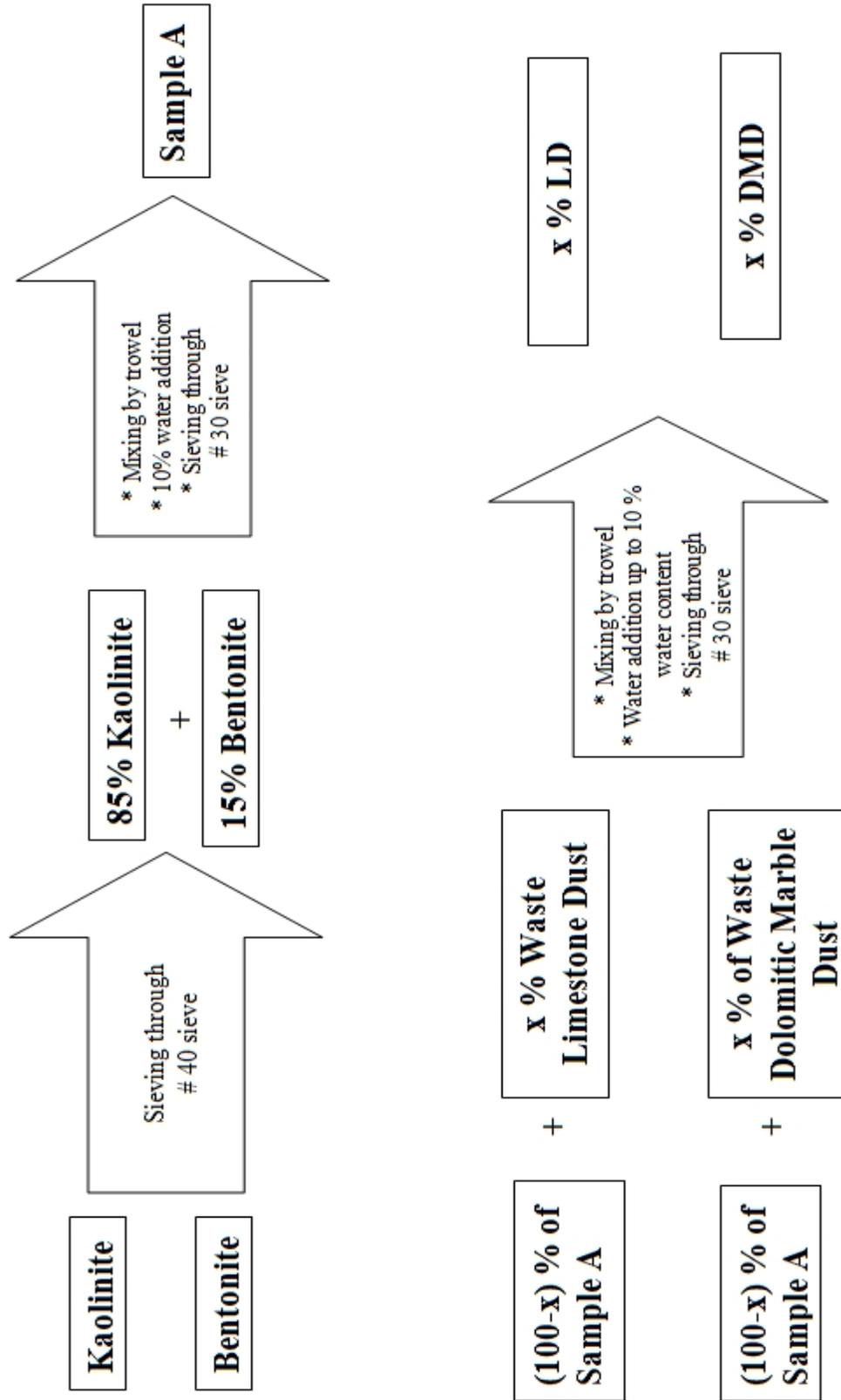


Figure 4.4 Preparation of the Test Samples

4.4. Sample Properties

To determine the index properties of samples hydrometer test, Atterberg limit tests and specific gravity test were performed according to ASTM D 2435. Clay percentages were determined by grain size distribution curves from hydrometer test.

Soil classification was done according to Unified Soil Classification System (USCS) by plotting test results on plasticity chart (Fig 4.5).

Swelling potentials were calculated for each sample using PI and clay percentages according to classification chart of Seed et al. (1962) (Fig 4.6).

Grain size distribution curves of samples were grouped according to stabilizer added and plotted, also, on the same graph, grain size distribution curves of Sample A and waste limestone dust were plotted to determine effect of stabilizer addition on grain size distribution curves. However, hydrometer analysis of pure waste dolomitic marble dust did not give any results due to immediate settlement of this dust at the bottom of hydrometer flask. Nevertheless, there was no problem occurred in waste dolomitic marble dust added samples (Fig. 4.7 and Fig. 4.8).

The sample properties, swell potential and soil classification according to USCS are tabulated in Table 4.3.

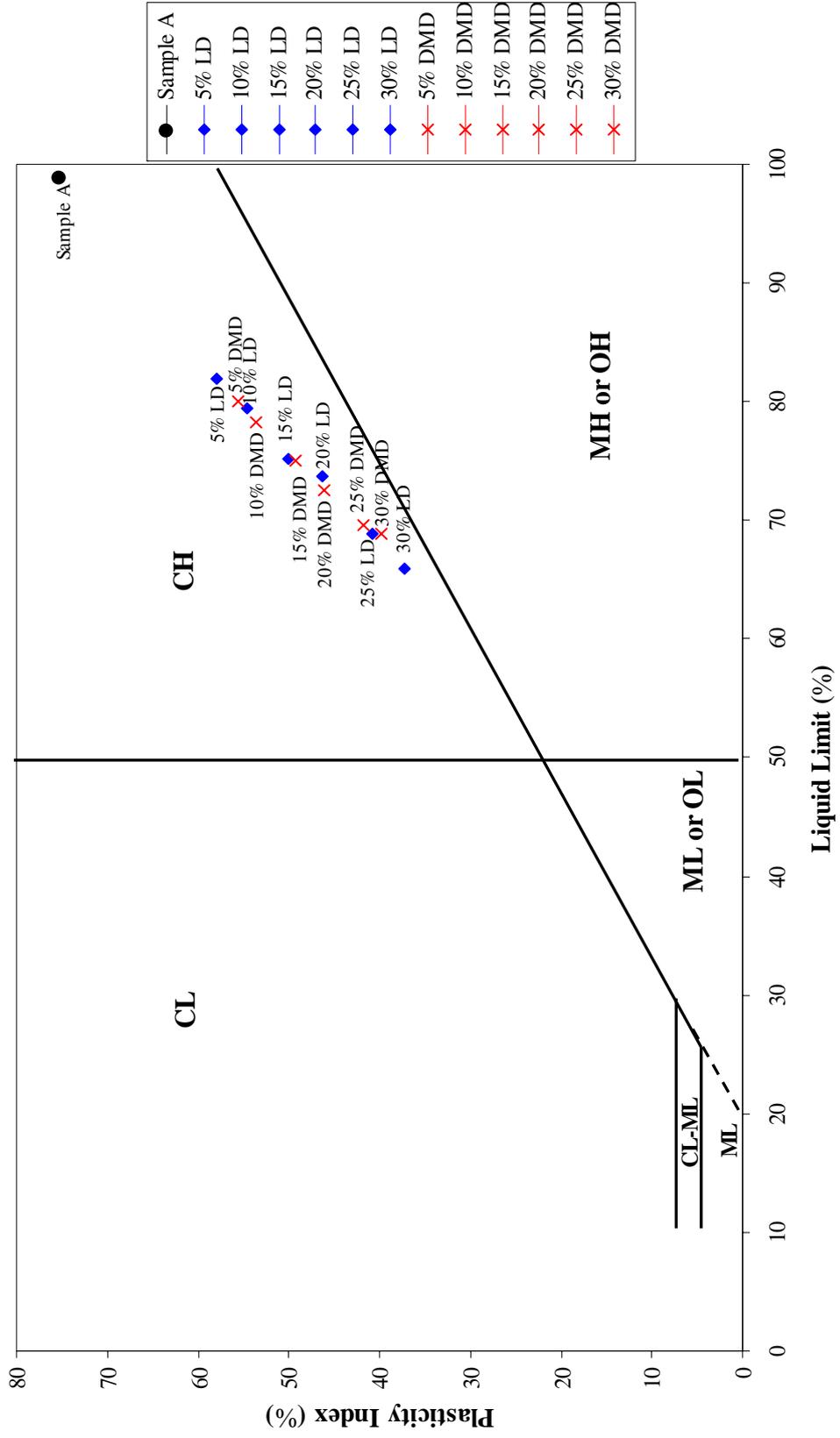


Figure 4.5 Plasticity Chart: Unified System

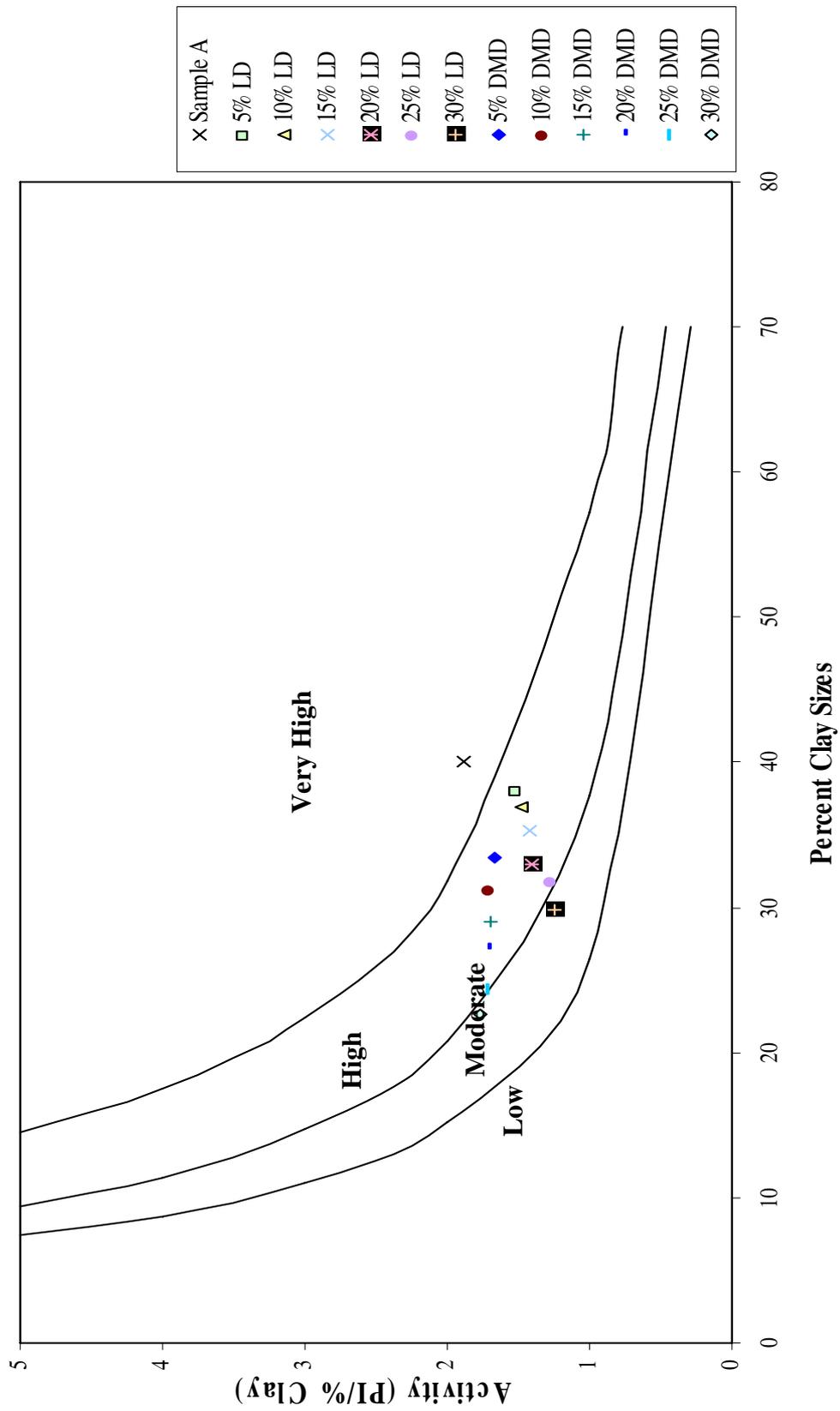


Figure 4.6 Classification Chart for Swelling Potential (After Seed, et al., 1962)

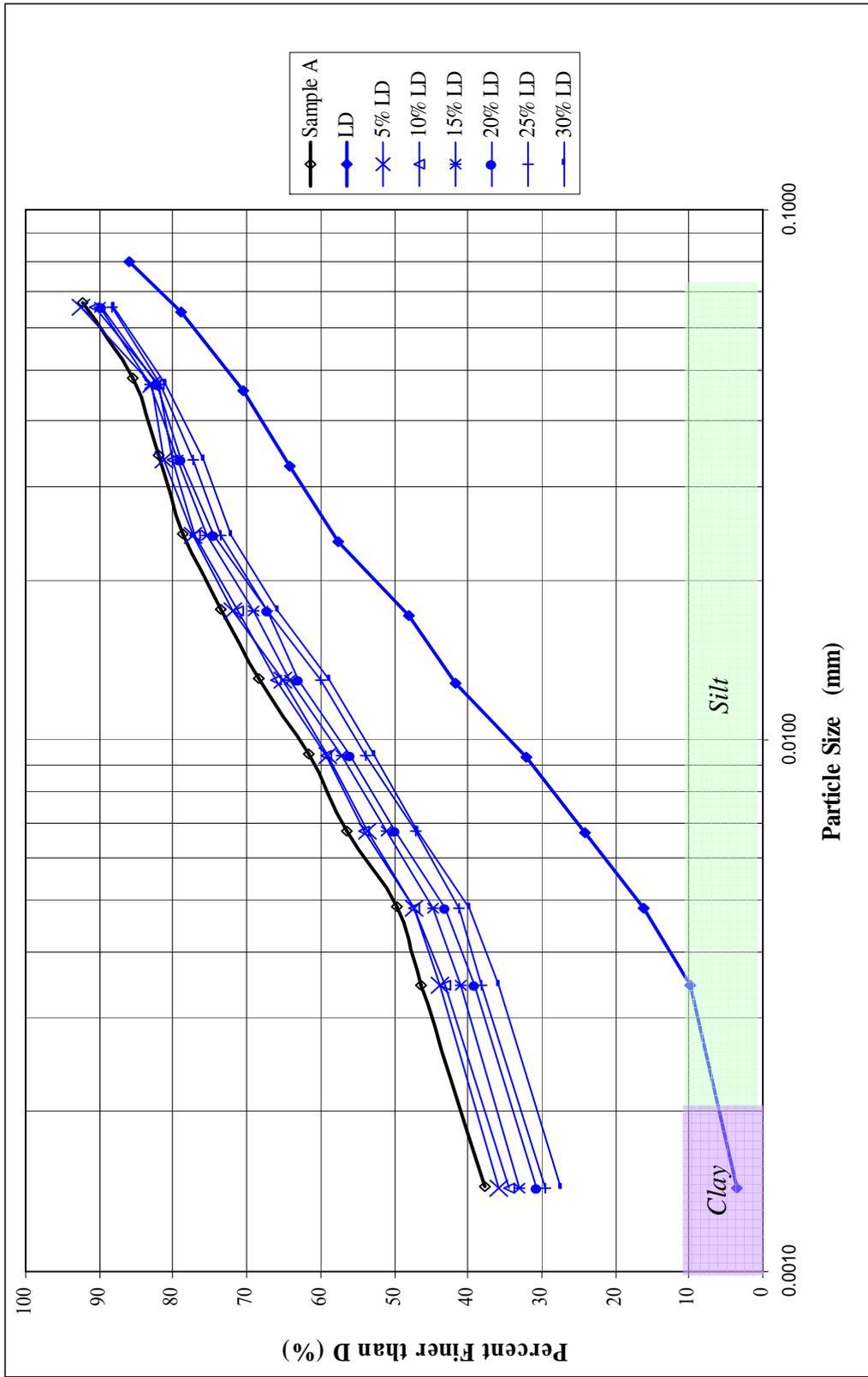


Figure 4.7 Grain Size Distribution Curves of Waste Limestone Dust Added Samples

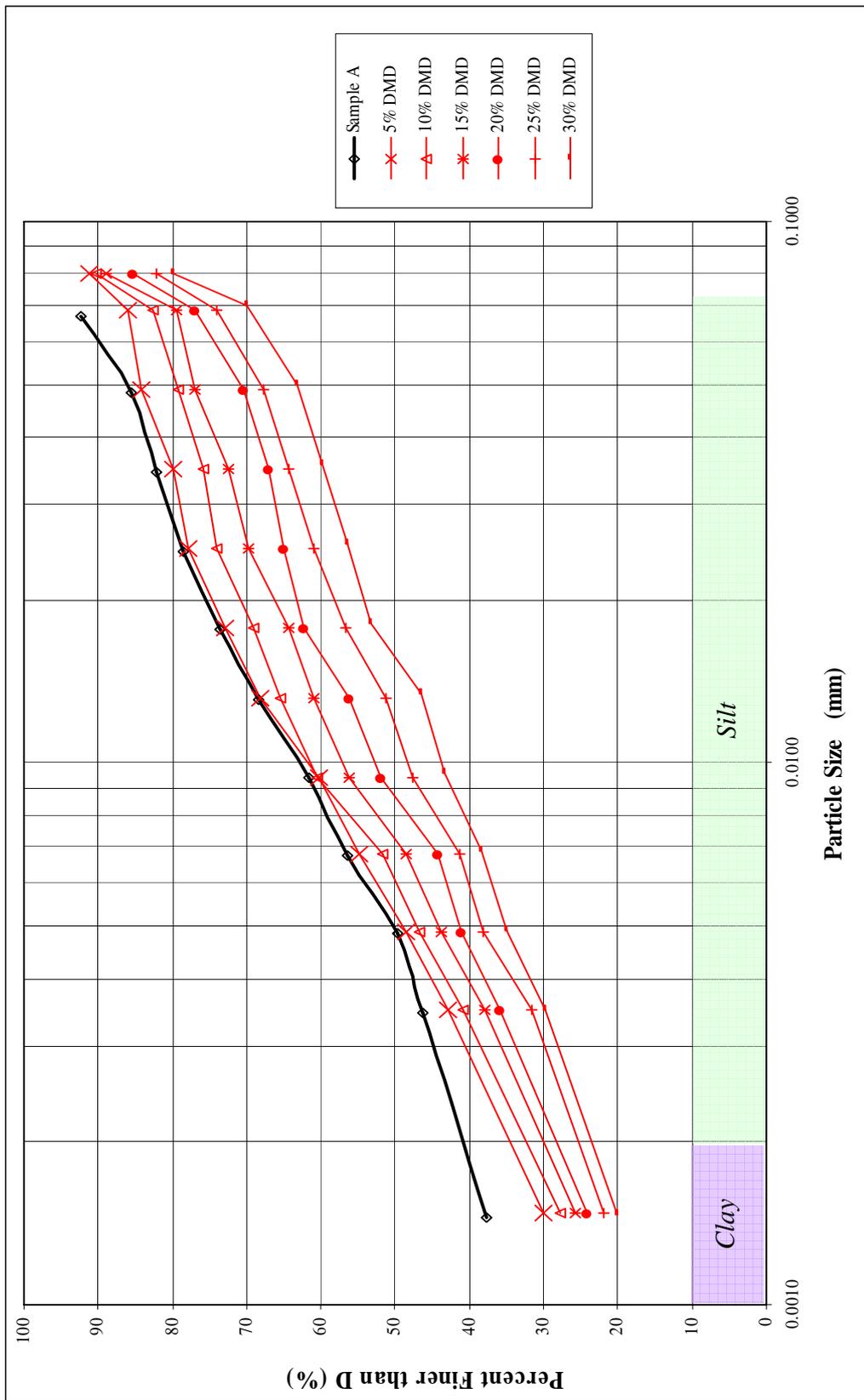


Figure 4.8 Grain Size Distribution Curves of Waste Dolomitic Marble Dust Added Samples

Table 4.3 Properties of the samples

Sample	Clay (%)	Silt & Fine Sand (%)	Gs	LL (%)	PL (%)	PI (%)	SL (%)	SI (%)	USCS	Activity	Swelling Potential
A	40.1	59.9	2.42	98.9	23.6	75.3	18.2	80.7	CH	1.88	Very High
5% LD	38.1	61.9	2.44	81.9	23.9	58.0	19.9	62.0	CH	1.52	High
10% LD	36.9	61.2	2.46	79.4	24.8	54.6	20.1	59.3	CH	1.48	High
15% LD	35.3	63.1	2.46	75.2	25.2	50.0	23.6	51.6	CH	1.42	High
20% LD	33.0	64.7	2.48	73.7	27.3	46.4	25.2	48.5	CH	1.41	High
25% LD	31.8	67.0	2.49	68.8	28.1	40.7	29.2	39.6	CH	1.28	High
30% LD	29.8	68.2	2.50	65.9	28.7	37.2	30.1	35.8	CH	1.25	Medium
5% DMD	33.4	66.6	2.45	80.0	24.4	55.7	20.1	59.9	CH	1.67	High
10% DMD	31.2	68.8	2.47	78.3	24.8	53.6	24.6	53.7	CH	1.72	High
15% DMD	29.0	71.0	2.49	75.1	25.9	49.2	26.9	48.2	CH	1.70	High
20% DMD	27.2	72.8	2.51	72.6	26.5	46.1	27.8	44.8	CH	1.69	High
25% DMD	24.4	75.6	2.52	69.6	27.8	41.8	28.1	41.5	CH	1.71	High
30% DMD	22.6	77.4	2.54	68.9	29.1	39.8	28.7	40.2	CH	1.76	Medium

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

LL: Liquid Limit, PL: Plastic Limit, PI: Plasticity Index, SL: Shrinkage Limit, SI: Shrinkage Index, Activity: PI / % Clay

LD: Waste limestone dust; DMD: Waste dolomitic marble dust

Naming is explained with the following two examples:

5% LD: 95% Sample A + 5% Waste Limestone Dust

5% DMD: 95% Sample A + 5% Waste Dolomitic Marble Dust

4.5. Test Procedure

4.5.1 Free Lime Content Test on Stabilizers

Pozzolanic activity is the one of the main factors using of waste limestone dust and waste dolomitic marble dust. Lime content of these stabilizers plays major role for pozzolanic activity.

ASTM C 25 (Standard Test Methods for Chemical Analysis of Limestone, Quicklime and Hydrated Lime) was used as a reference to determine the hydrated lime content of test samples.

To determine hydrated lime in stabilizers, the following steps can be summarized;

1) Neutralized sugar solution is prepared. (40 g sugar is dissolved in 100 ml CO₂-free water, several drops of 4% phenolphthalein indicator and 0.1 N NaOH is added to sugar solution).



Figure 4.9 A view of neutralized sugar solution

2) 2.804 g of sample is brushed in 40 ml of CO₂-free water and 100 ml of neutralized sugar solution is added to this mixture.

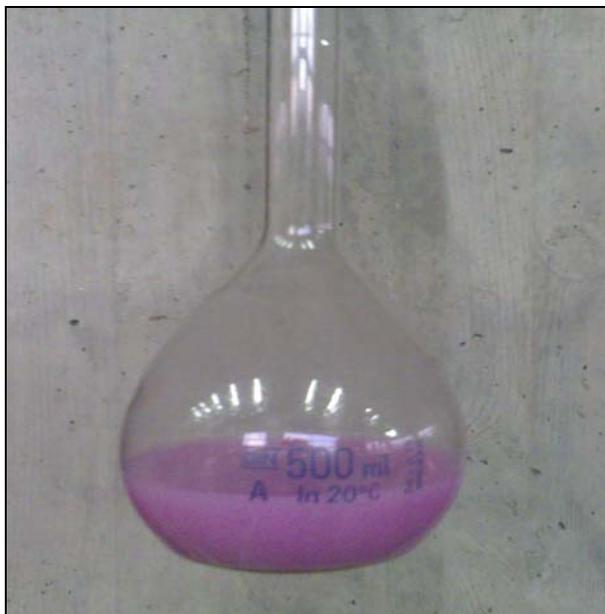


Figure 4.10 A view of neutralized sugar solution added to mixture of sample and water

3) The mixture is left for reactions for 15 min. and in 5 min. intervals it is swirled.

4) After 15 min. the mixture is titrated with HCl, until the first disappearance of the pink color, which persists for 3 s.

$$\text{Available lime } [\text{Ca}(\text{OH})_2], \% = \frac{N \times V \times 3.704}{W}$$

where; N: normality of acid solution (= 1 in this study)

V: standard HCl (1.0 N), ml

W: weight of sample, g (= 2.804 g in this study)

4.5.2. Free Swell Ratio Test

To have information on the swell property of soil, Sridharan and Prakash (2000-b) proposed the free swell ratio method of characterizing the soil expansivity. Free swell ratio (FSR) defined as the ratio of equilibrium sediment volume of 10-g oven dried soil passing a 425-µm sieve in distilled water (Vd) to that of carbon tetra chloride or kerosene (Vk) (Sridharan and Prakash, 2004).

$$FSR = \frac{Vd}{Vk}$$

In this study, the recommended procedure of Sridharan and Prakash for FSR was followed and samples were prepared as stated in section 4.3 without addition of water.

10 g of each sample was added to 100 ml of water in a graduated jar. Also, another 10 g of each sample was added to 100 ml of commercially available kerosene in a graduated jar. After 24 hours, sediment volumes of samples were measured to calculate Free Swell Ratio, FSR (Fig.4.11). Classification of soils based on free swell ratio is listed in Table 4.4.

Table 4.4 Classification of soils based on free swell ratio (after Prakash and Sridharan,2004)

Free Swell Ratio	Soil Expansivity	Clay Type	Dominant Clay Mineral Type
<1.0	Negligible	Non-swelling	Kaolinitic
1.0-1.5	Low	Mixture of Non-swelling and Swelling	Kaolinitic and Montmorillonitic
1.5-2.0	Moderate	Swelling	Montmorillonitic
2.0-4.0	High	Swelling	Montmorillonitic
>4.0	Very High	Swelling	Montmorillonitic



Figure 4.11 Free Swell Test for 5% DMD

4.5.3 Free Swell Test

4.5.3.1 Compaction of Samples for Free Swell Test

In this study, “Free swell method” was used for the determination of the effectiveness of stabilizers on the swell potential of samples. To apply this method, the samples were prepared as stated in Section 4.3 and compacted in a guide ring obtaining a bulk density of 1.80 g/cm^3 and dry density of 1.64 g/cm^3 . Then, compacted samples were transferred into consolidation rings. This compaction procedure was done in two steps. Firstly, some of the calculated amount of sample was placed into guide ring until it contacted with Part C of the piston (Fig. 4.12). Then, with help of a hydraulic jack, pressure applied on the top of piston, Part A. This process was finished until the entire calculated amount of sample was compressed.

After compaction of sample in the guide ring, the guide ring and compacted sample was put on the consolidation ring. Then, longer part of the piston, Part A, placed on the guide ring and the piston was pushed, so the compacted sample was placed into consolidation ring.

For experiments on cured samples, compacted sample was pushed in another guide ring with a larger diameter, then, the sample was took and left to cure at 22°C and 70% relative humidity in the dessicator for 7 and 28 days. After, 7 or 28 days passed, sample was placed in consolidation ring.

The set up for compaction and transference of samples is shown in Figure 4.12.

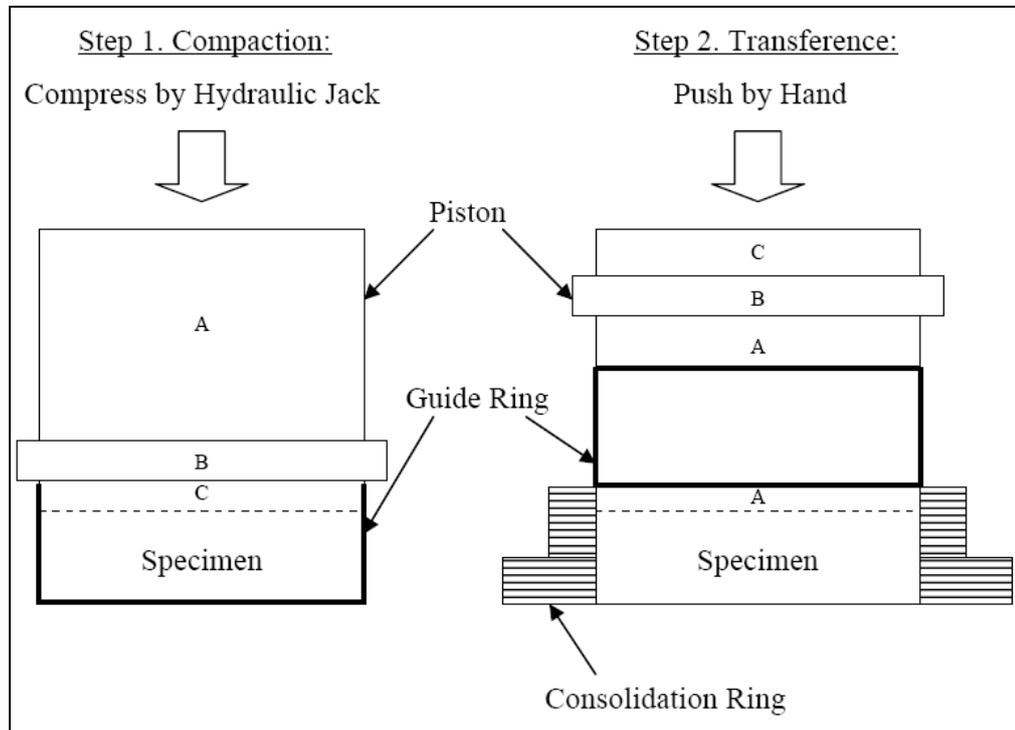


Figure 4.12 The Set Up for Compaction and Transference of Samples (after Çetiner, 2004)

4.5.3.2 Procedure for Free Swell Test

The compaction procedure is explained in section 4.5.3.1. Consolidation ring with compacted sample 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. Then, the oedometer was mounted, and the dial gauge measuring the vertical deflection was set to zero. The sample was inundated by pouring water through standpipes and water directly from the top of the oedometer. Distilled water was preferred to eliminate effects of ions in water. Swelling of the sample started right after the pouring of water.

Deflection values were read in dial gauge, and time were recorded. Deflection values at 0.5 min, 1 min, 2 min, 4 min, 15 min, 30 min, 1 hour, 2 hours, 4 hours, 8 hours, 1 day, 2 days, 3 days and 4 days were used to plot swelling percentage vs. time graphs.

When the sample had no further tendency to swell, last reading of swell was recorded and oedometer was dismantled. Then, the final water content was determined in accordance with ASTM D2435 - 90, after filter papers were separated from the surface of the sample.

The percent of free swell can be expressed as;

$$\text{Free Swell} = \frac{\Delta h}{h} \times 100$$

where ΔH : Change in initial height (H) of the specimen

H: Initial height of the specimen

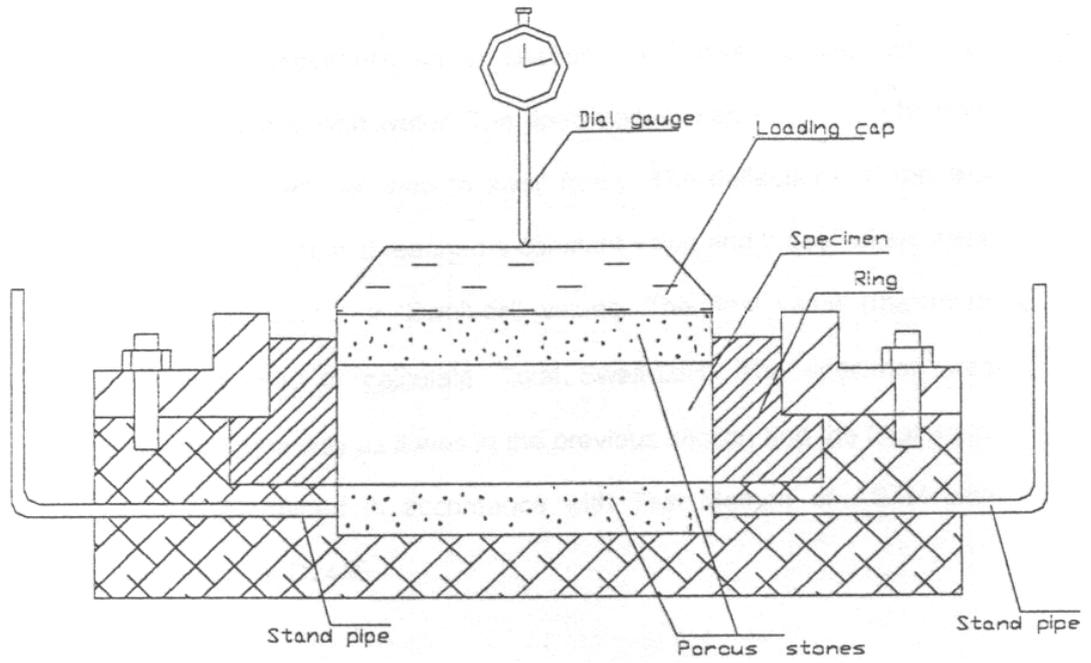


Figure 4.13 Experimental Setup Drawing for Free Swell Test (İpek, 1998)



Figure 4.14 The Oedometer

4.6. Experimental Program

Experimental study was conducted in seven phases:

- 1) Free lime content test was performed on stabilizers
- 2) Analyses for stabilizers were performed by METU Central Laboratory and General Directorate of Mineral Research and Exploration (MTA).
- 3) Hydrometer tests, Atterberg limit tests, and specific gravity tests were applied to the samples.
- 4) Free Swell Ratio Test was applied to the samples.
- 5) Free swell tests were applied to the samples without cure.
- 6) Free swell tests were applied to the 7 days cured samples.
- 7) Free swell tests were applied to the 28 days cured samples

Table 4.5 Mixtures for experimental study

Sample	Expansive Soil (%)	Waste limestone dust (%)	Waste dolomitic marble dust (%)
Sample A	100	-	-
5% LD	95	5	-
10% LD	90	10	-
15% LD	85	15	-
20% LD	80	20	-
25% LD	75	25	-
30% LD	70	30	-
5% DMD	95	-	5
10% DMD	90	-	10
15% DMD	85	-	15
20% DMD	80	-	20
25% DMD	75	-	25
30% DMD	70	-	30

4.7. Test Results

Results of available lime test are presented in Figure 4.14. In Figure 4.15, free swell ratio test results are presented.

Effect of addition of waste limestone dust and waste dolomitic marble dust on liquid limit (LL), plastic limit (PL), plasticity index ($PI = LL - PL$), shrinkage limit (SL) and shrinkage index ($SI = LL - SL$) are presented in Fig.4.16, 4.17, 4.18, 4.19 and 4.20, respectively. Also, specific gravities of each sample are presented in Figure 4.21.

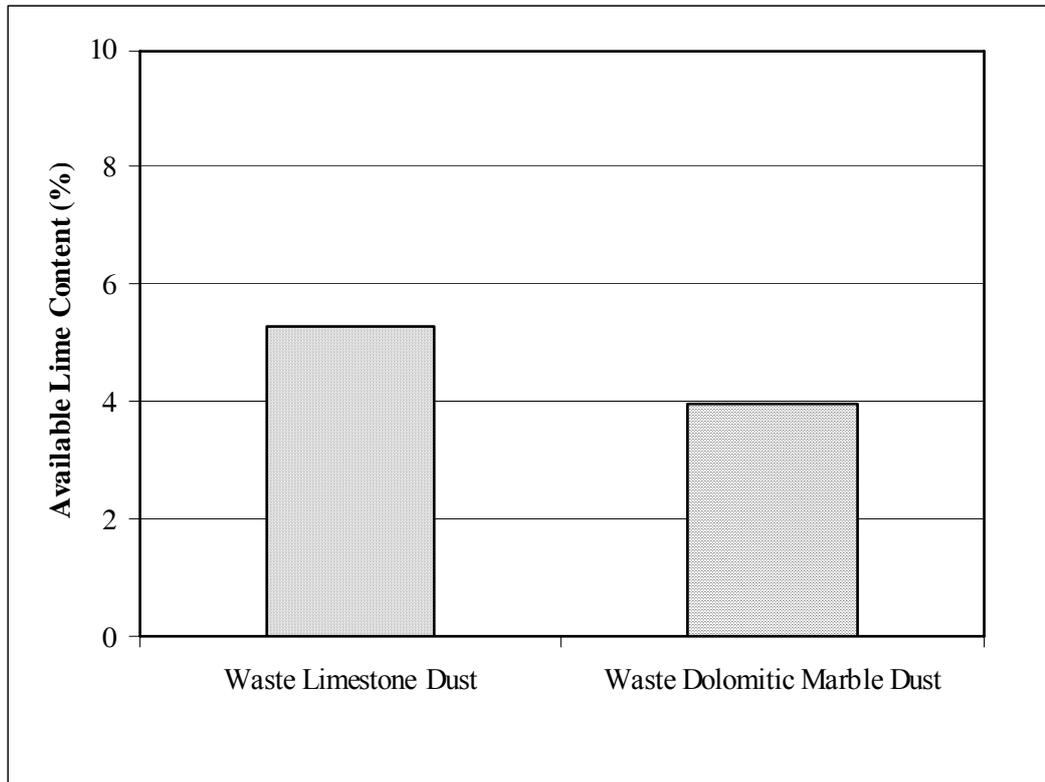


Figure 4.14 Results of Available Lime Test

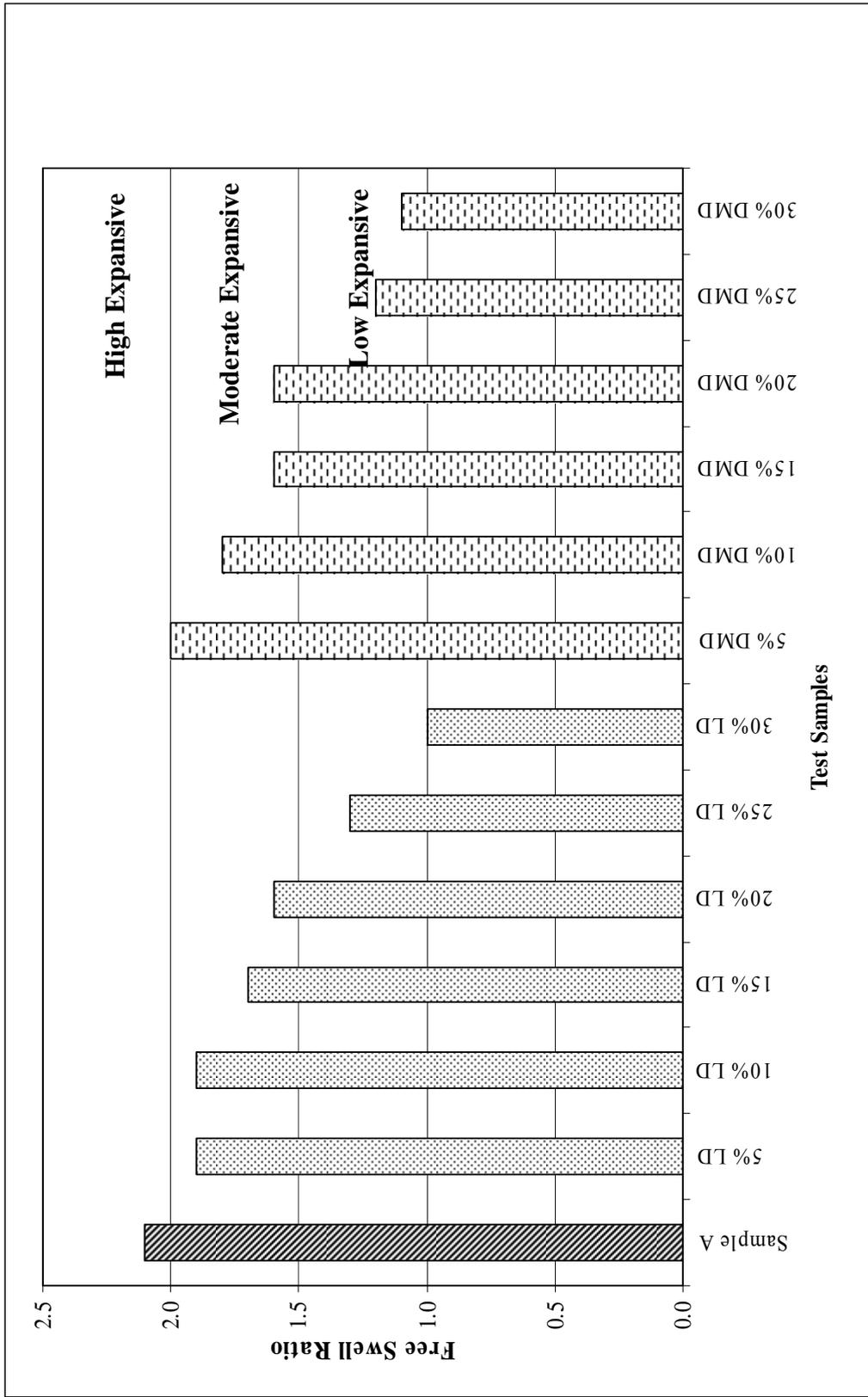


Figure 4.15 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on Free Swell Ratio of the Samples

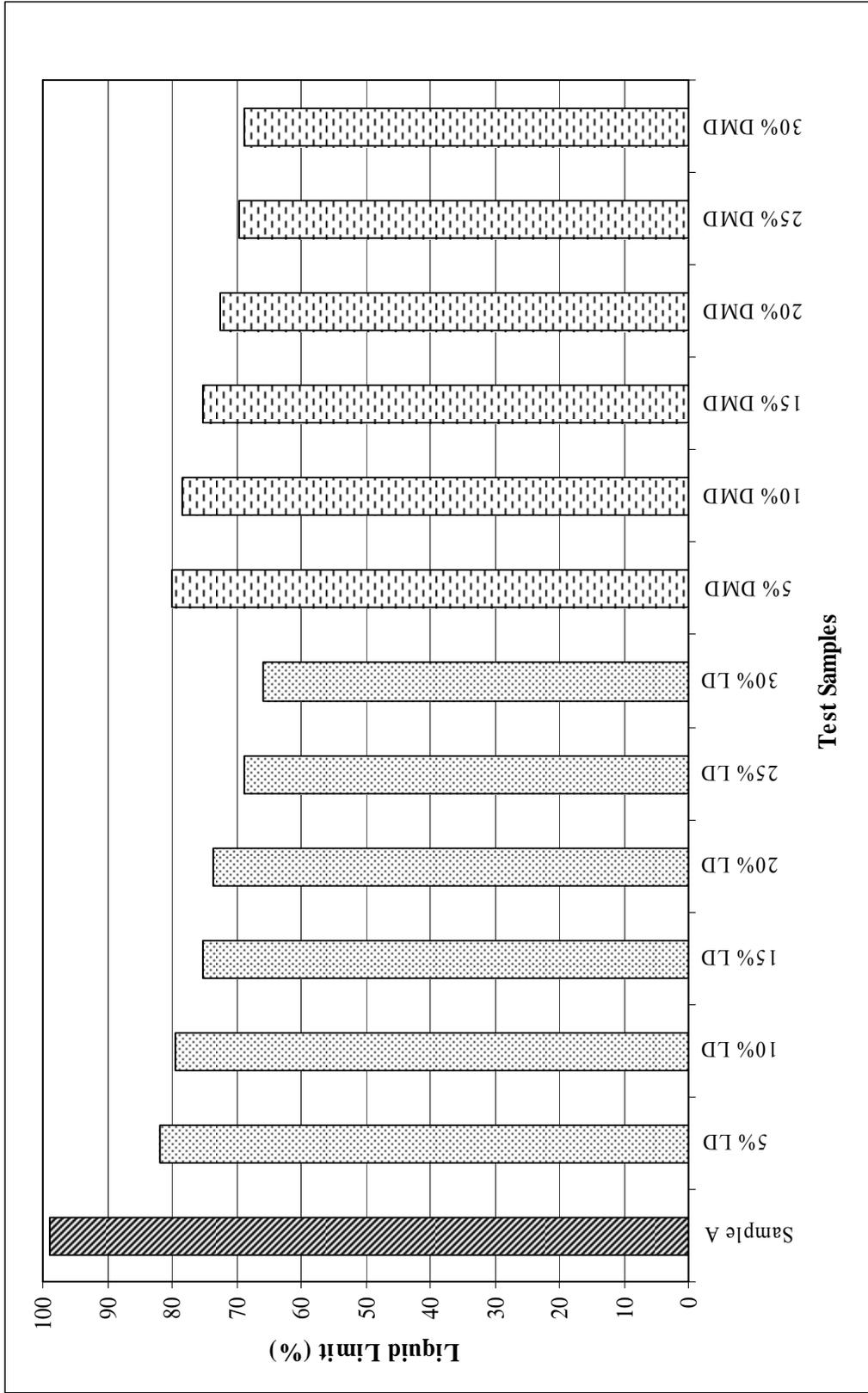


Figure 4.16 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the Liquid Limit (LL) of the Samples

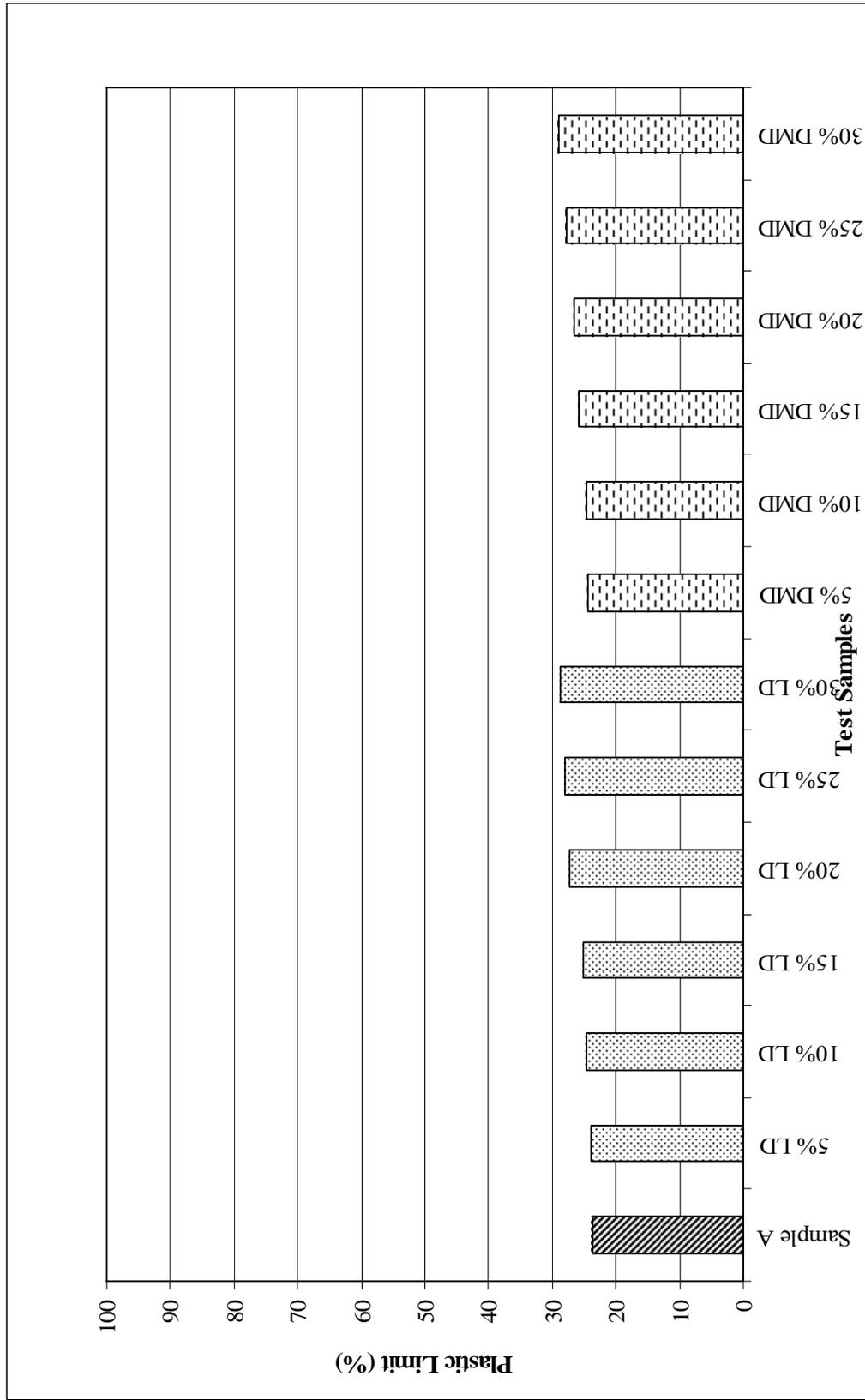


Figure 4.17 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the Plastic Limit (PL) of the Samples

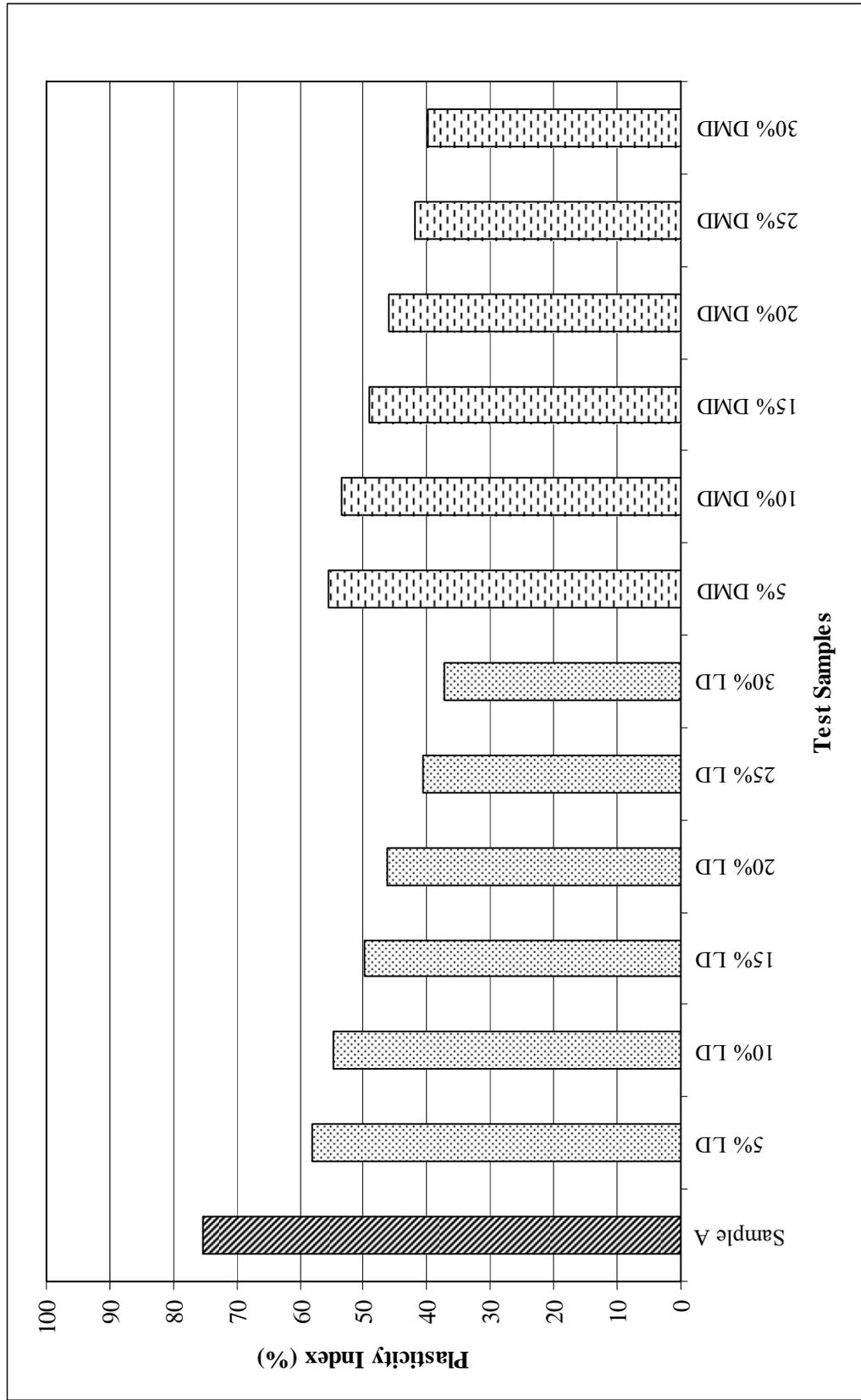


Figure 4.18 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the Plasticity Index (PI) of the Samples

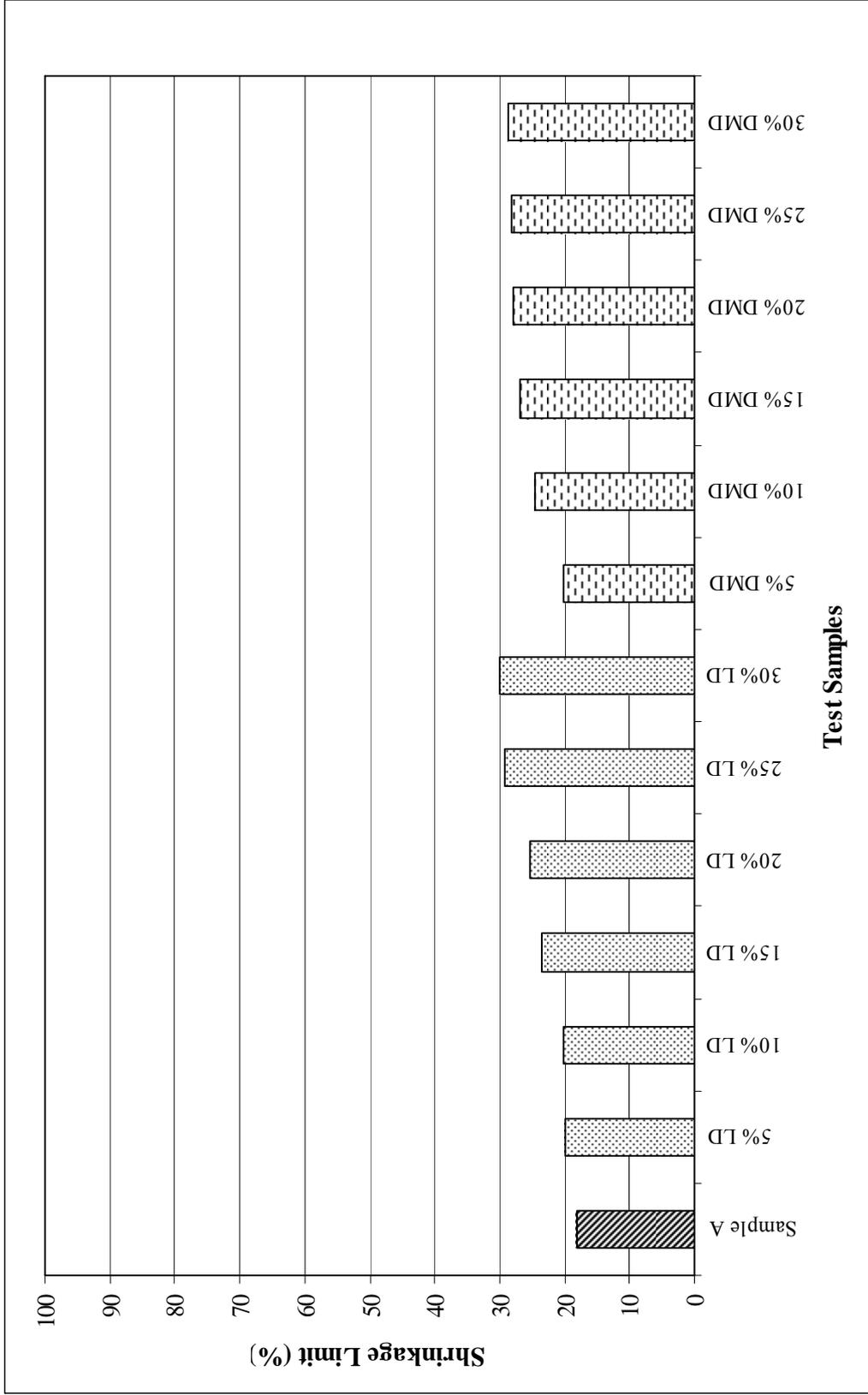


Figure 4.19 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the Shrinkage Limit (SL) of the Samples

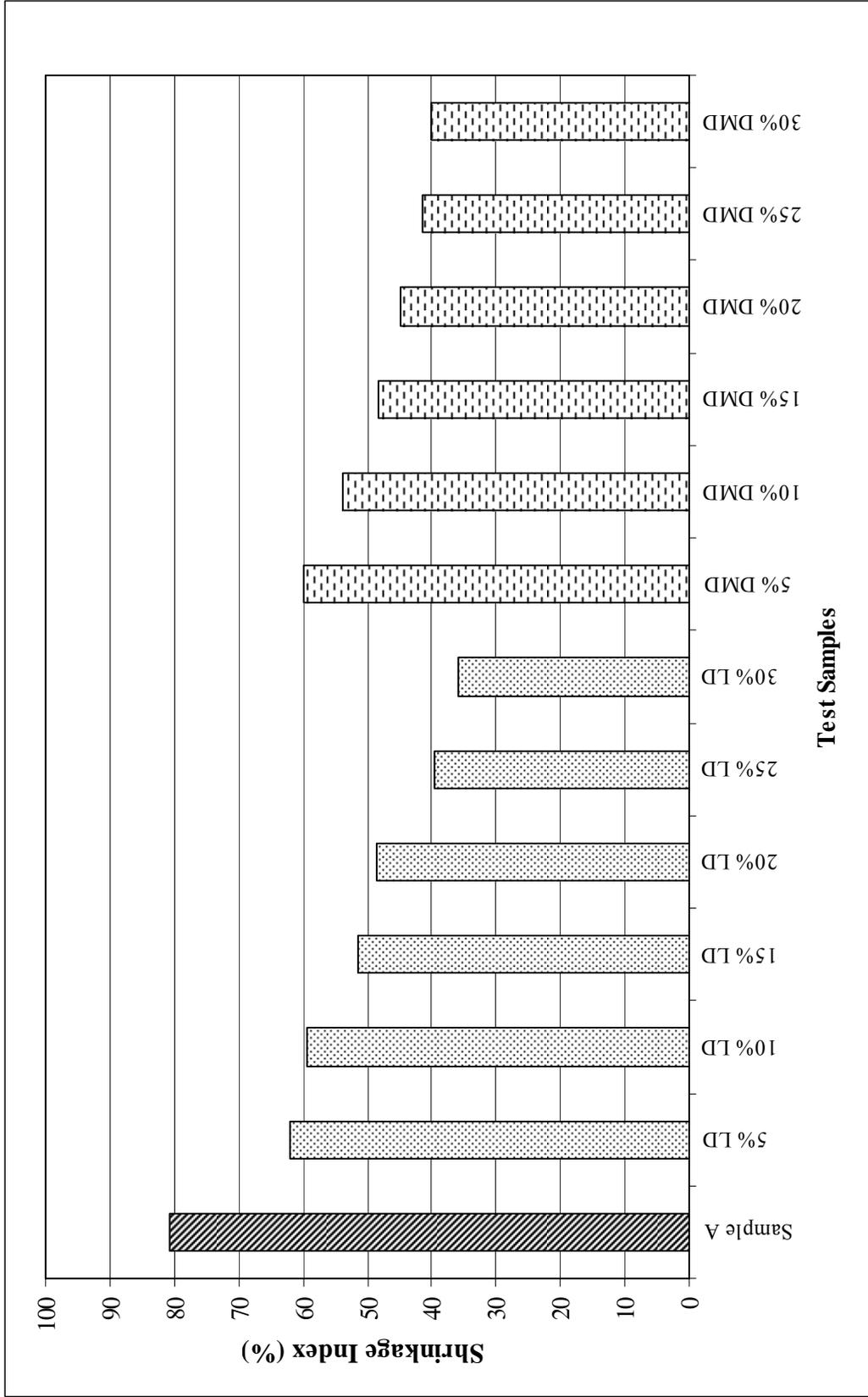


Figure 4.20 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the Shrinkage Index (SI) of the Samples

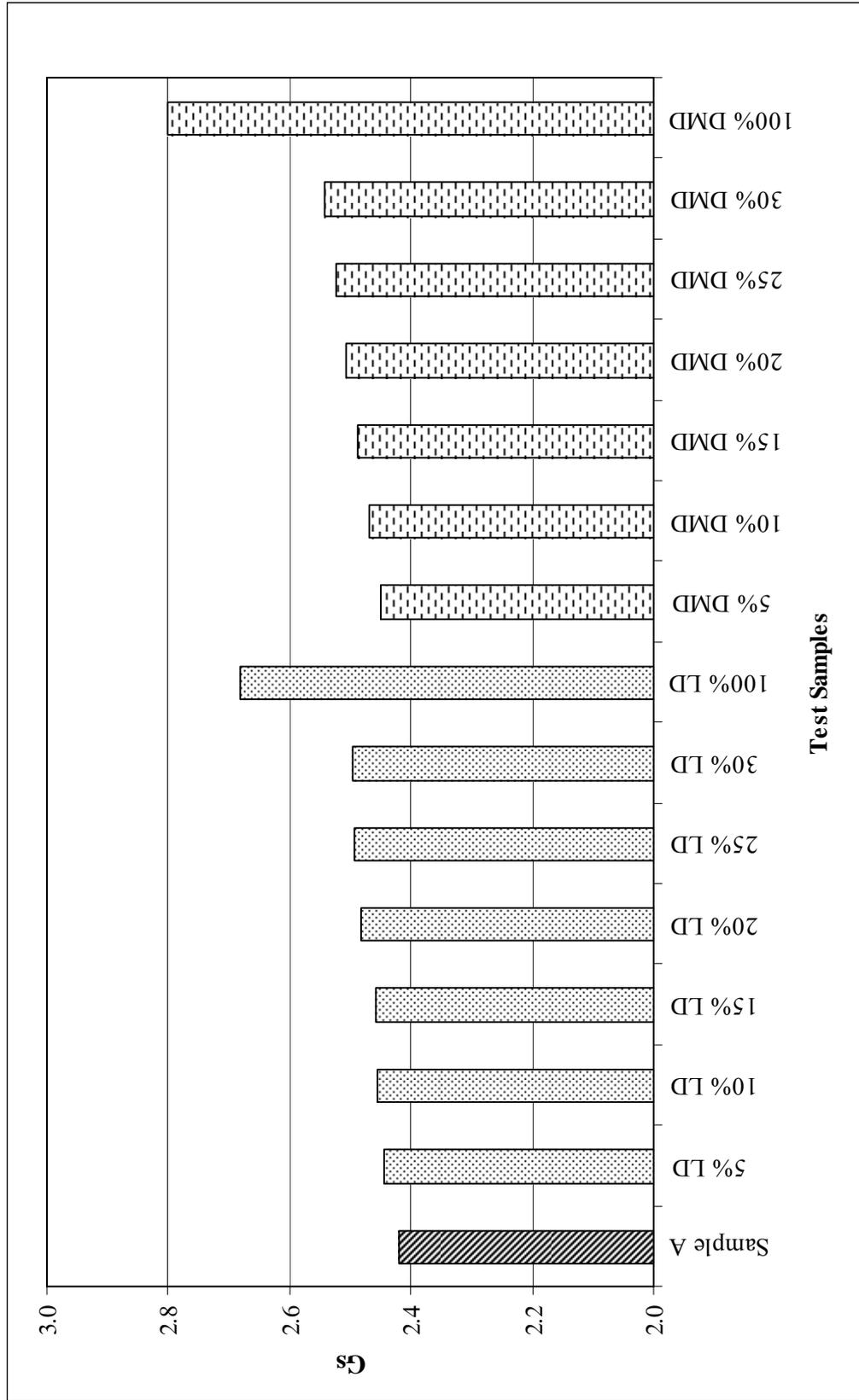


Figure 4.21 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the Specific Gravity (Gs) of the Samples

Swell percentage vs. time relationship for Sample A, a typical swell percentage vs. time graph, is presented in Figure 4.22. For swell percentage calculations dial readings recorded in free swell test were subtracted by initial readings and $\Delta H/H$ vs. time graph was plotted. Swelling percentage vs. time relationships of the waste limestone dust and waste dolomitic marble dust added samples are presented in Appendix B.

Effect of addition of waste limestone dust and waste dolomitic marble dust on free swell for samples without cure, 7 days cured and 28 days cured are presented in Figure 4.23, 4.24 and 4.25, respectively. Also, Figure 4.26 shows effect of cure on free swell.

There is no readily available method for measuring rate of swell. Thus, to evaluate the rate of swell of experiments, time elapsed for fifty percent swell, t_{50} is considered. Effect of addition of waste limestone dust and waste dolomitic marble dust on rate of swell for samples without cure, 7 days cured and 28 days cured are presented in Figure 4.27, 4.28 and 4.29, respectively. Also, Figure 4.30 shows effect of cure on rate of swell.

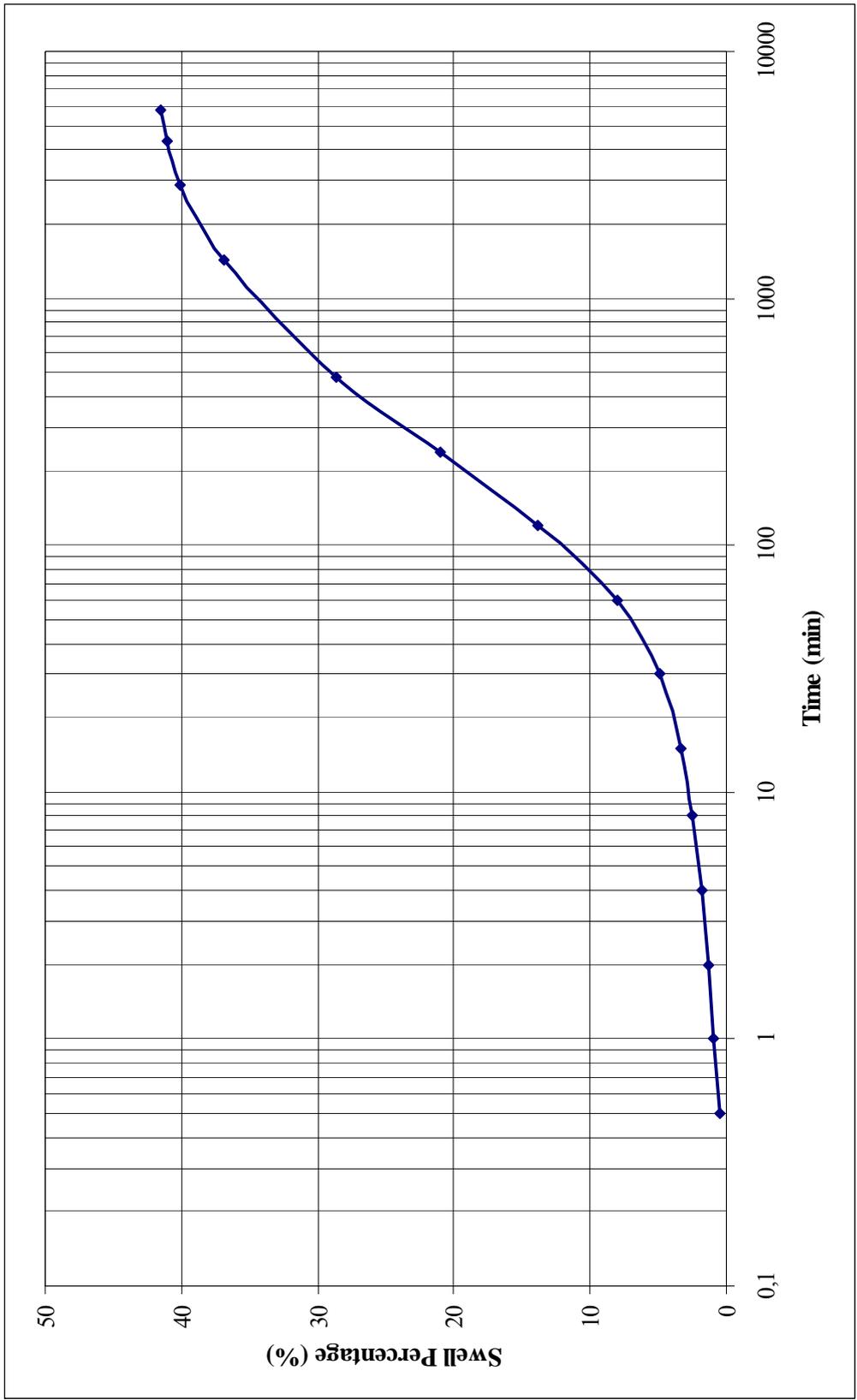


Figure 4.22 Swelling Percentage versus Time Relationship for Sample A

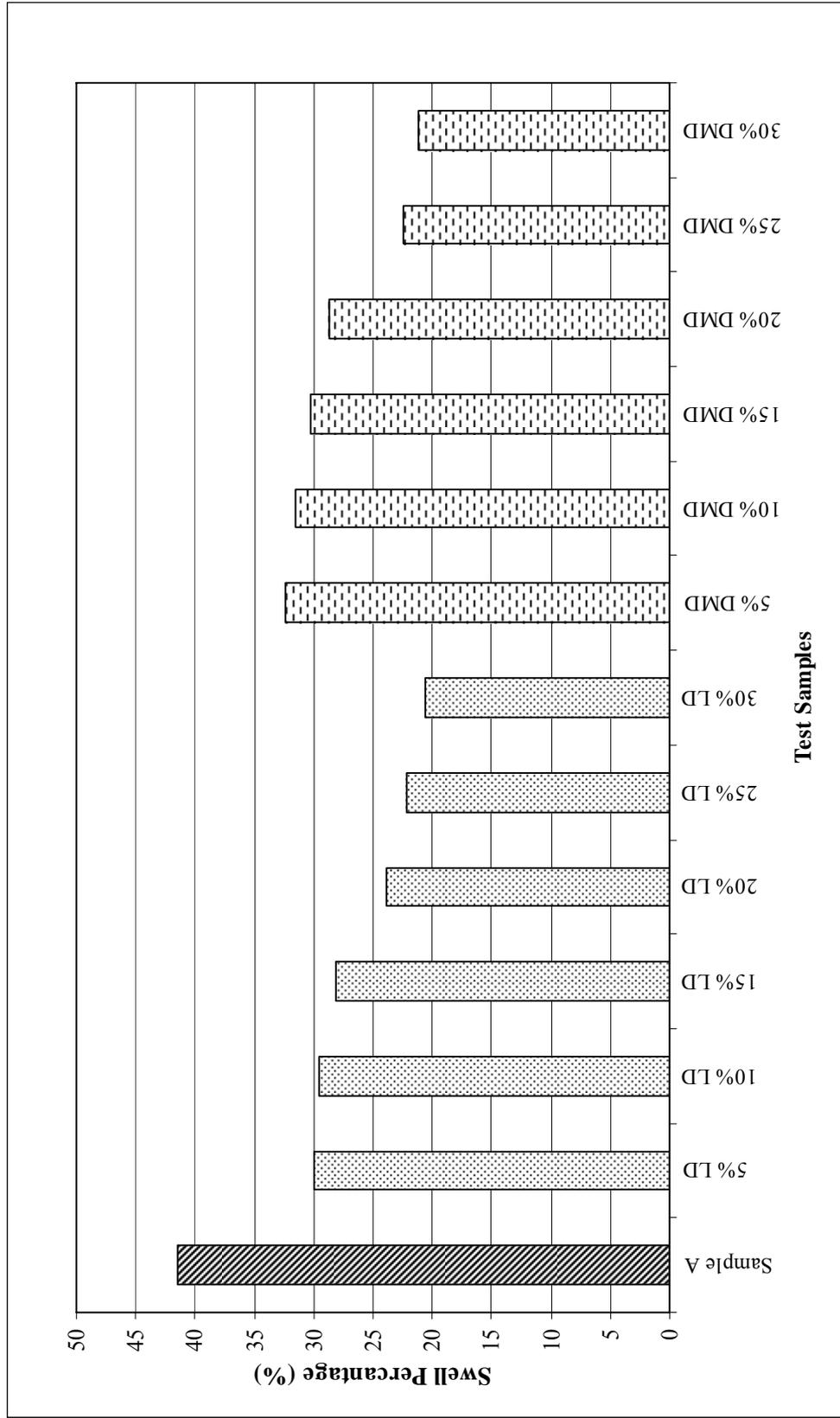


Figure 4.23 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the Swelling Percentage of Samples without Cure

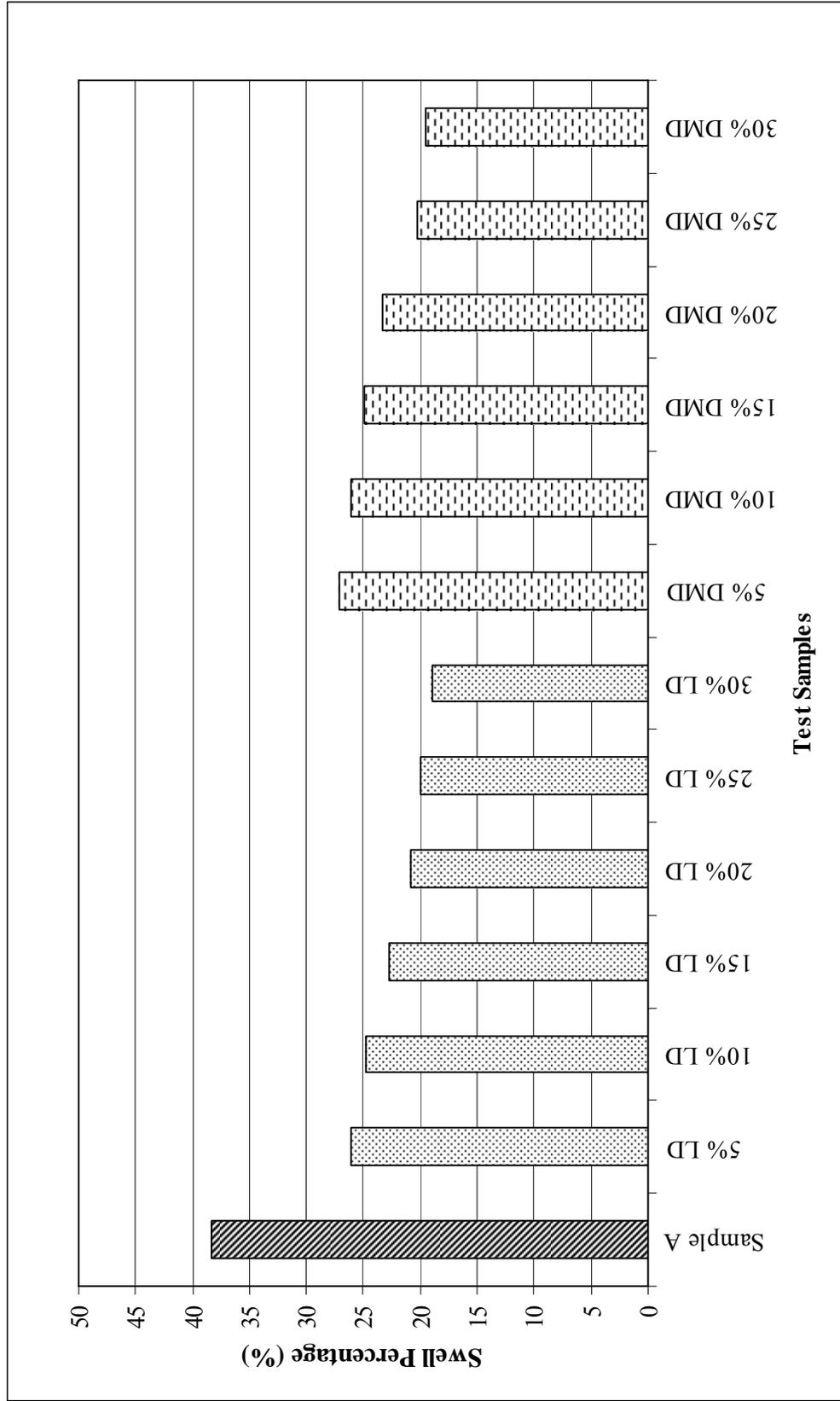


Figure 4.24 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the Swelling Percentage of 7 days Cured Samples

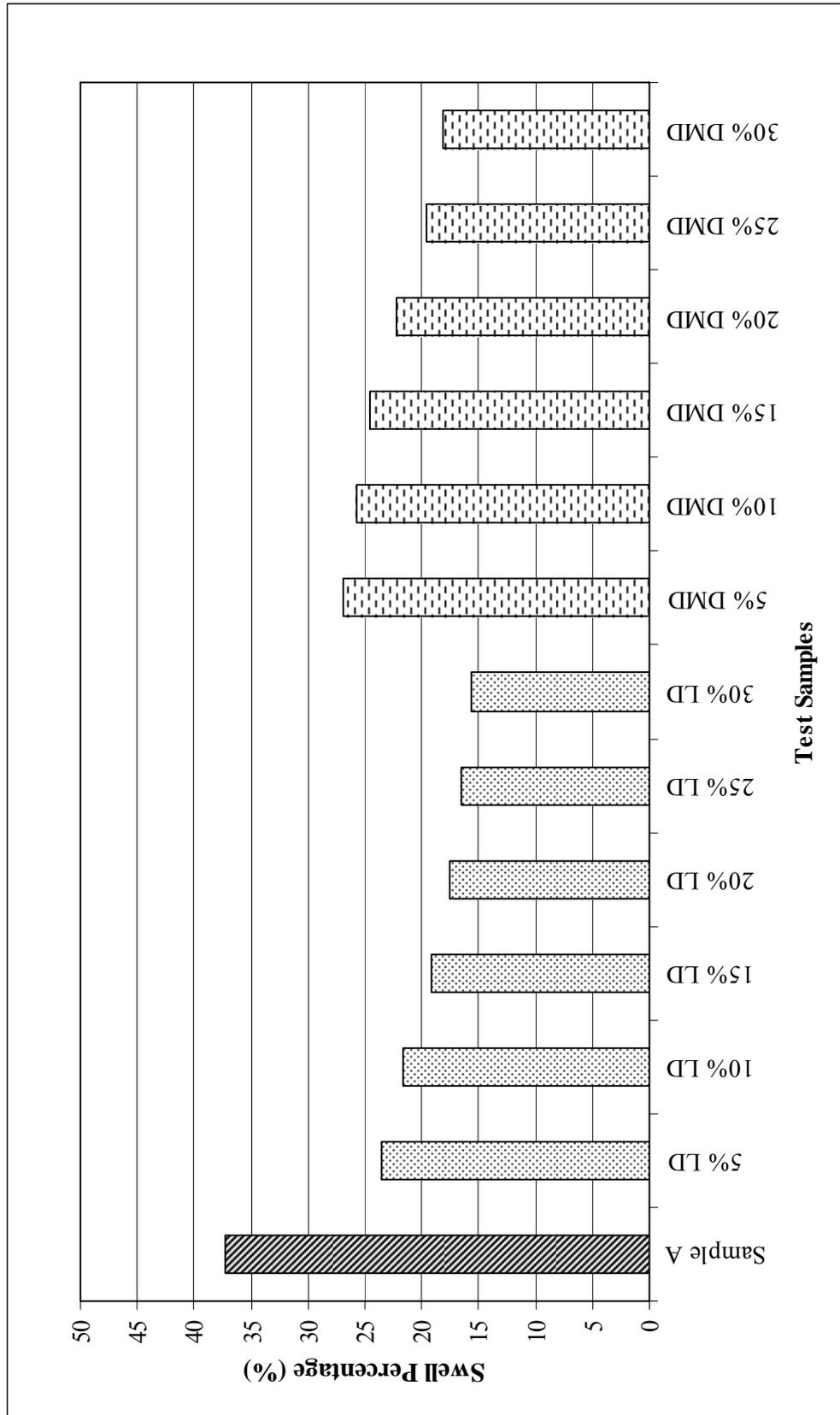


Figure 4.25 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the Swelling Percentage of 28 days Cured Samples

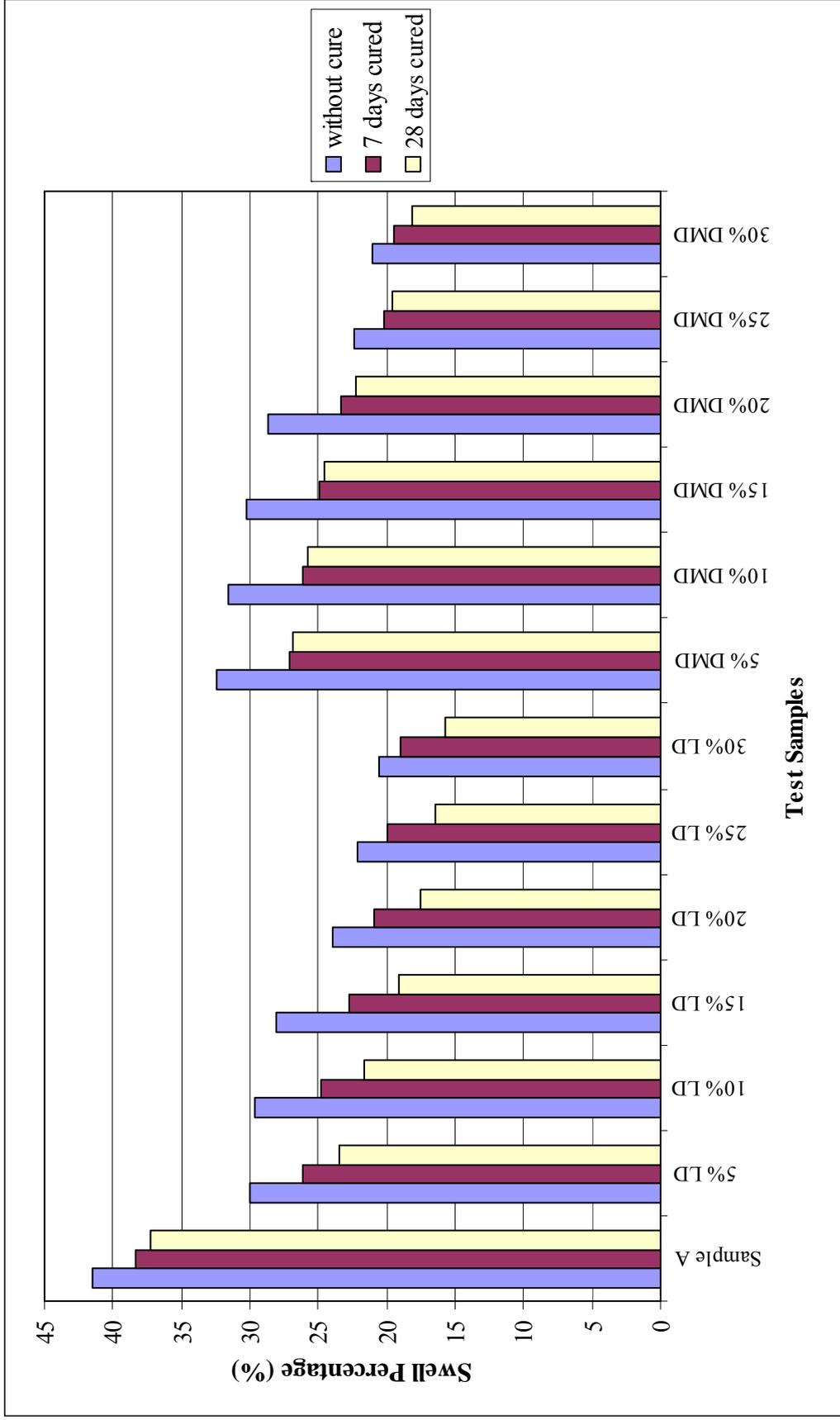


Figure 4.26 Effect of Cure on Swelling Percentage of the Samples

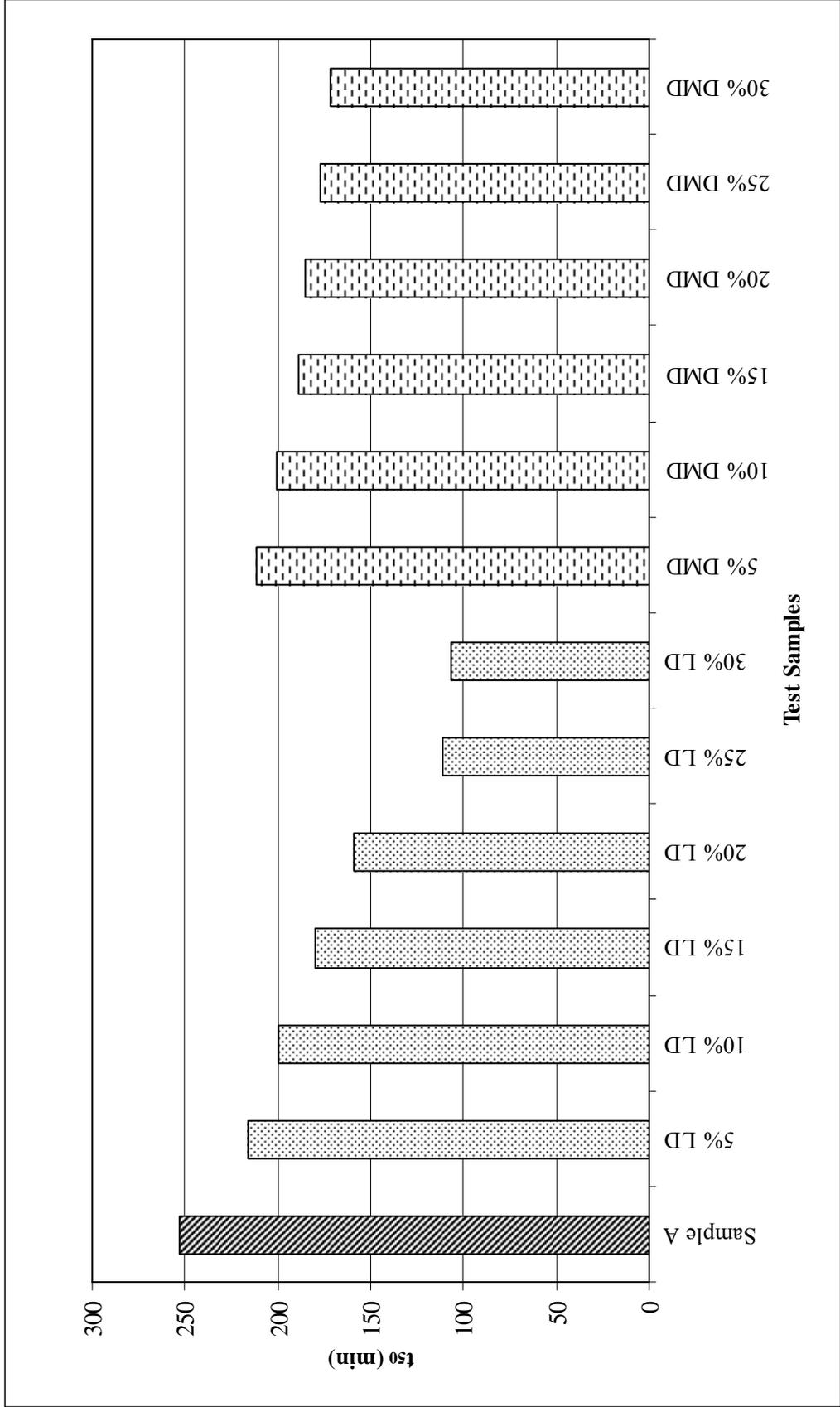


Figure 4.27 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the t_{50} of Samples without Cure

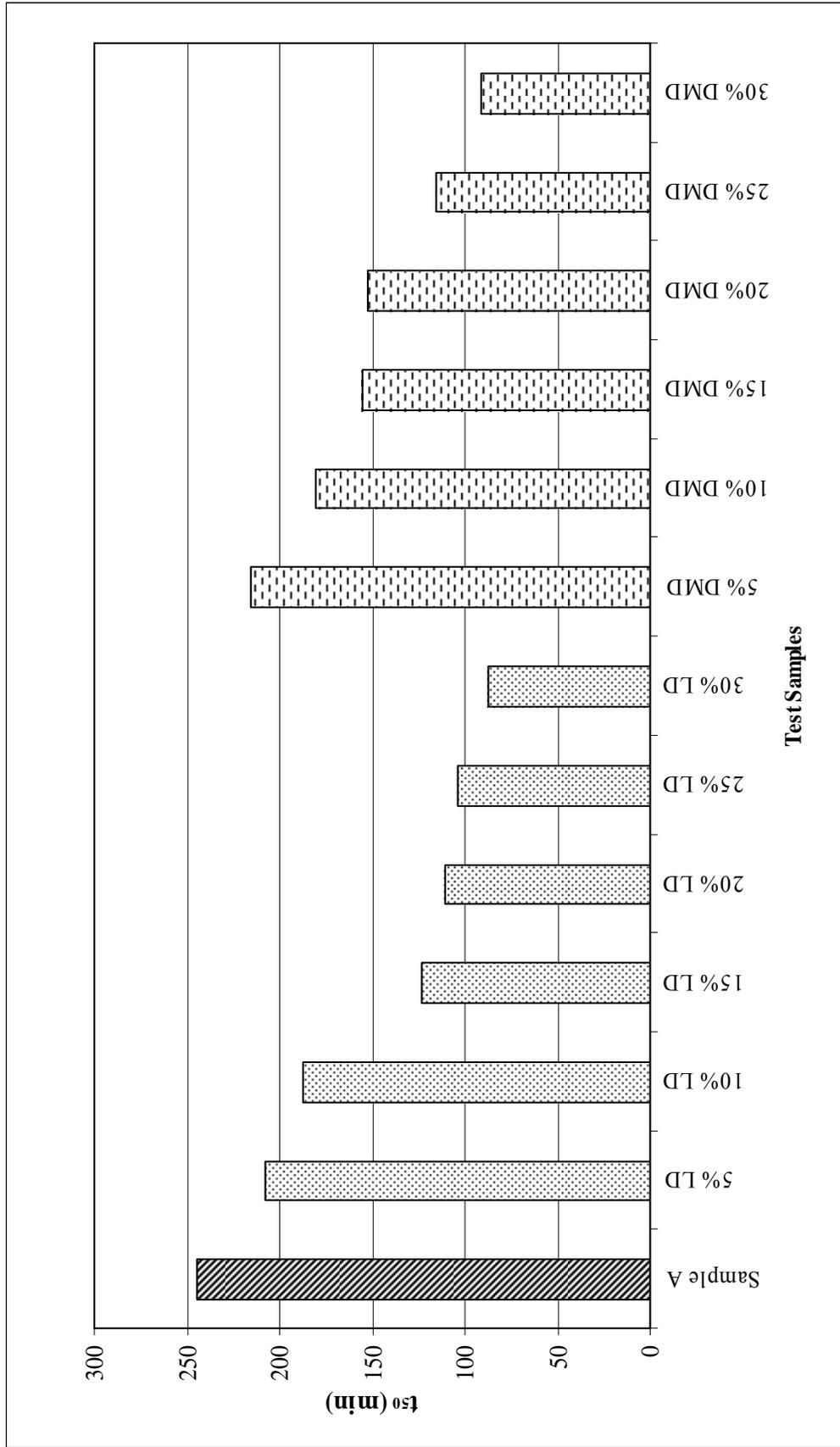


Figure 4.28 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the t_{50} of 7 days Cured Samples

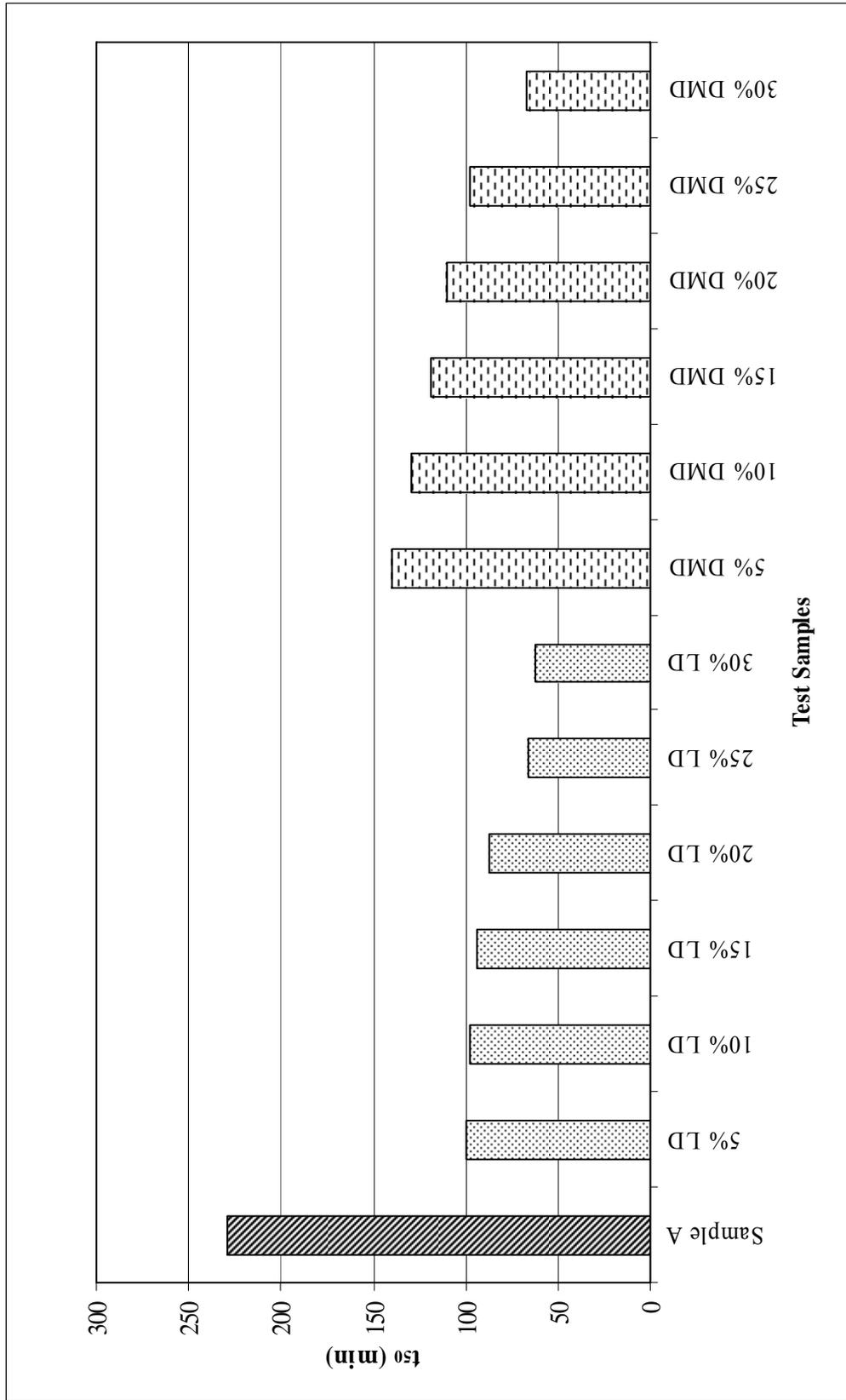


Figure 4.29 Effect of Addition of Waste Limestone Dust and Waste Dolomitic Marble Dust on the t₅₀ of 28 Days Cured Samples

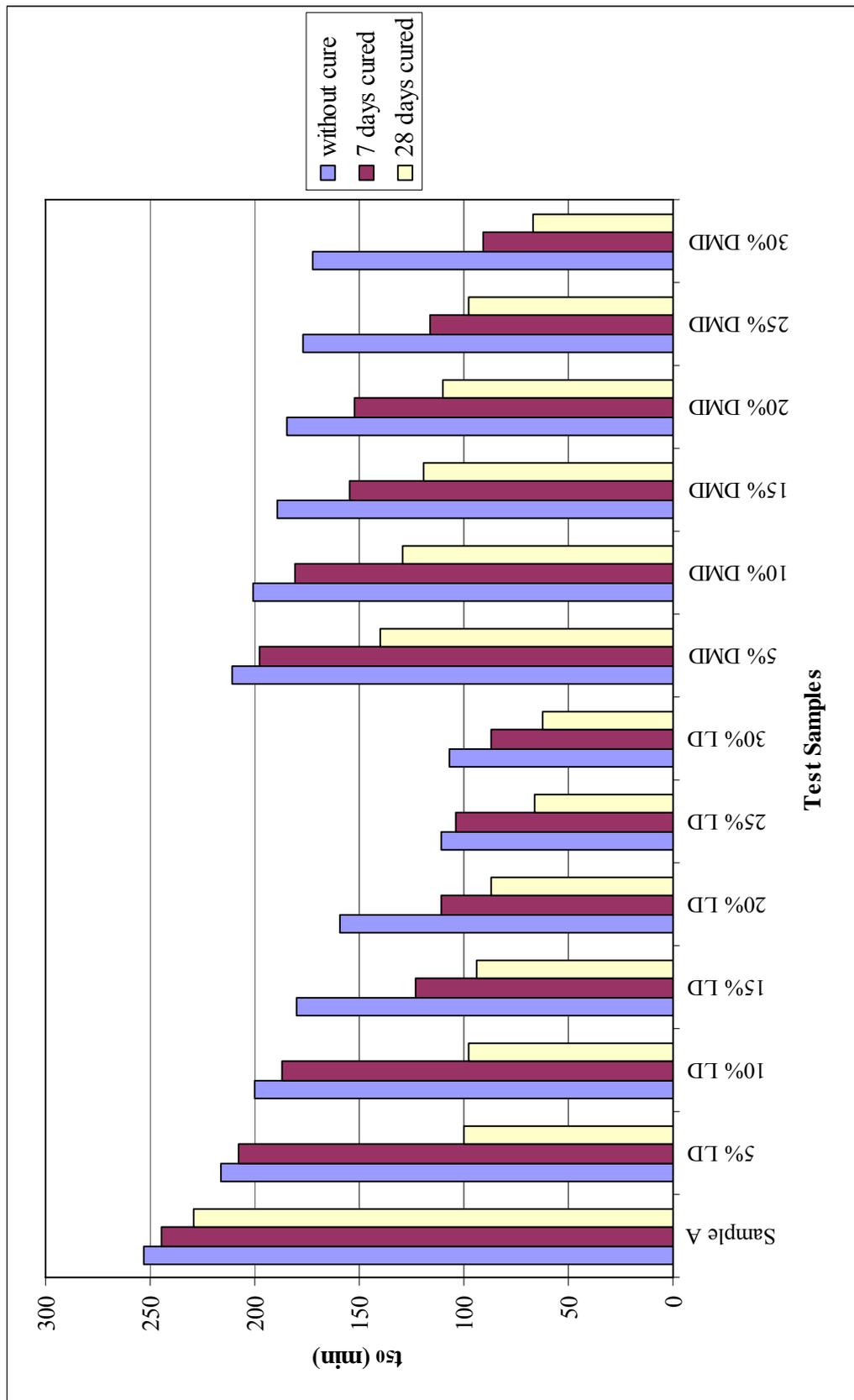


Figure 4.30 Effect of Cure on t_{50} of the Samples

4.8. Calculations for Prediction of Swell Percentage

A typical log time vs. percent swell relationship generally follows a standard “S” shape. This “S” shape curve can be divided into 3 phases; Initial, primary and secondary swelling. Initial swelling is generally less than 10% of total swelling. This is essentially due to swelling of the bentonite clay particles within the voids of the coarser non-swelling fractions. This swelling of particles does not cause an increase in the volume of the sample. Primary swelling develops when the voids can no longer accommodate further clay particle swelling. After the primary swelling complete slow continued swelling, secondary swelling occurs. In time/free swell vs. time graphs after some time elapsed, a straight-line relationship is apparent (Sivapullaiah, et.al, 1996). The slope of straight-line part of time/free swell vs. time graphs can be used to predict maximum swell.

Dakshanamurthy (1978) proposed a relationship for free swell,

$$\% \text{ swell} = \frac{t}{m * t + c}, \text{ where, } m \text{ is the slope of straight-line part of time/free swell}$$

vs. time graphs, c is the constant and t is the time at which swelling is required.

For maximum swell percent, time goes to infinity. Then,

$$\% \text{ maximum swell} = \lim_{t \rightarrow \infty} \frac{t}{m * t + c} = \lim_{t \rightarrow \infty} \frac{t}{t(m + c/t)} = \frac{1}{m}$$

Swelling percentage vs. time graph and time/free swell vs. time graph for 7 days cured 25% LD sample as examples are presented in Fig. 4.31 and 4.32, respectively. Predicted and measured free swell percentages and difference between them are presented in Table 4.6 and those graphs are presented in Figure 4.33, Figure 4.34 and Figure 4.35.

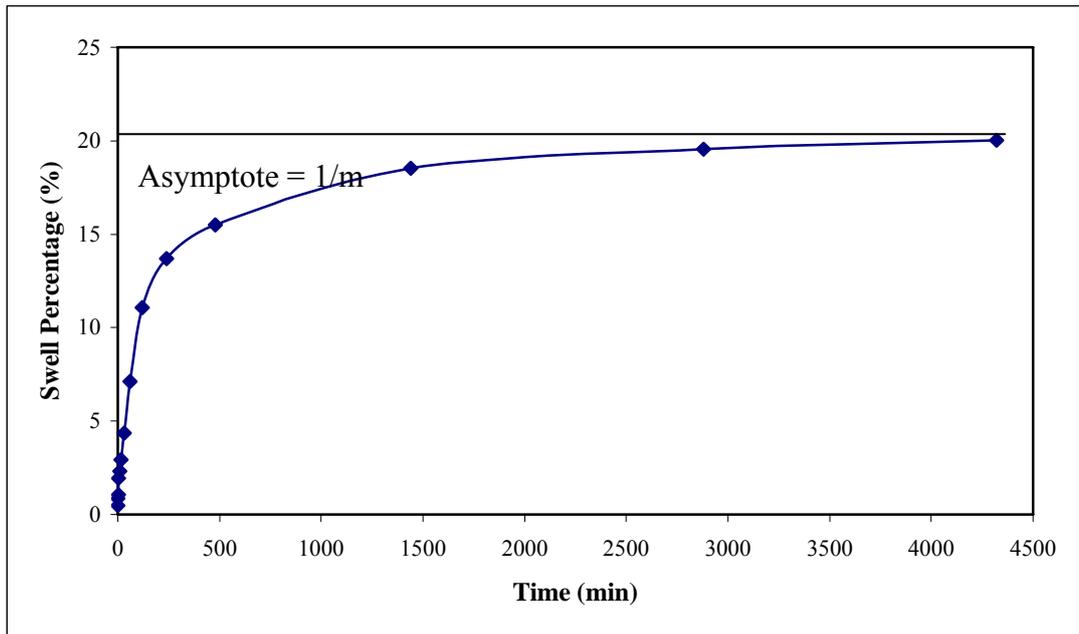


Figure 4.31 Swell Percentage vs. Time Graph of 7 days Cured 25 % LD Sample

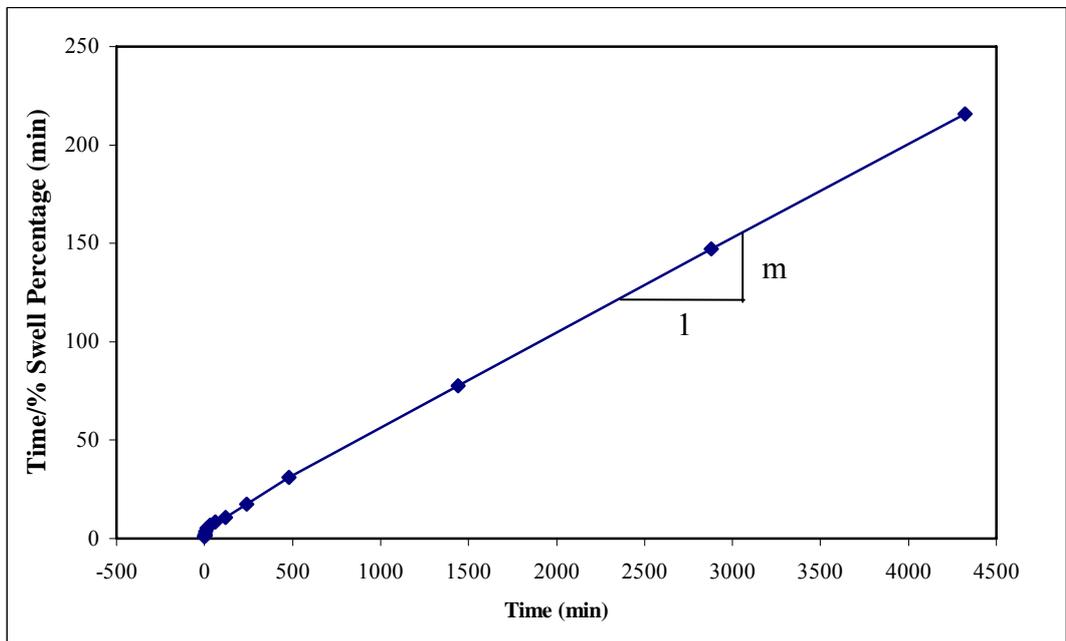


Figure 4.32 Time/%Swell Percentage vs. Time Graph of 7 days Cured 25 % LD Sample

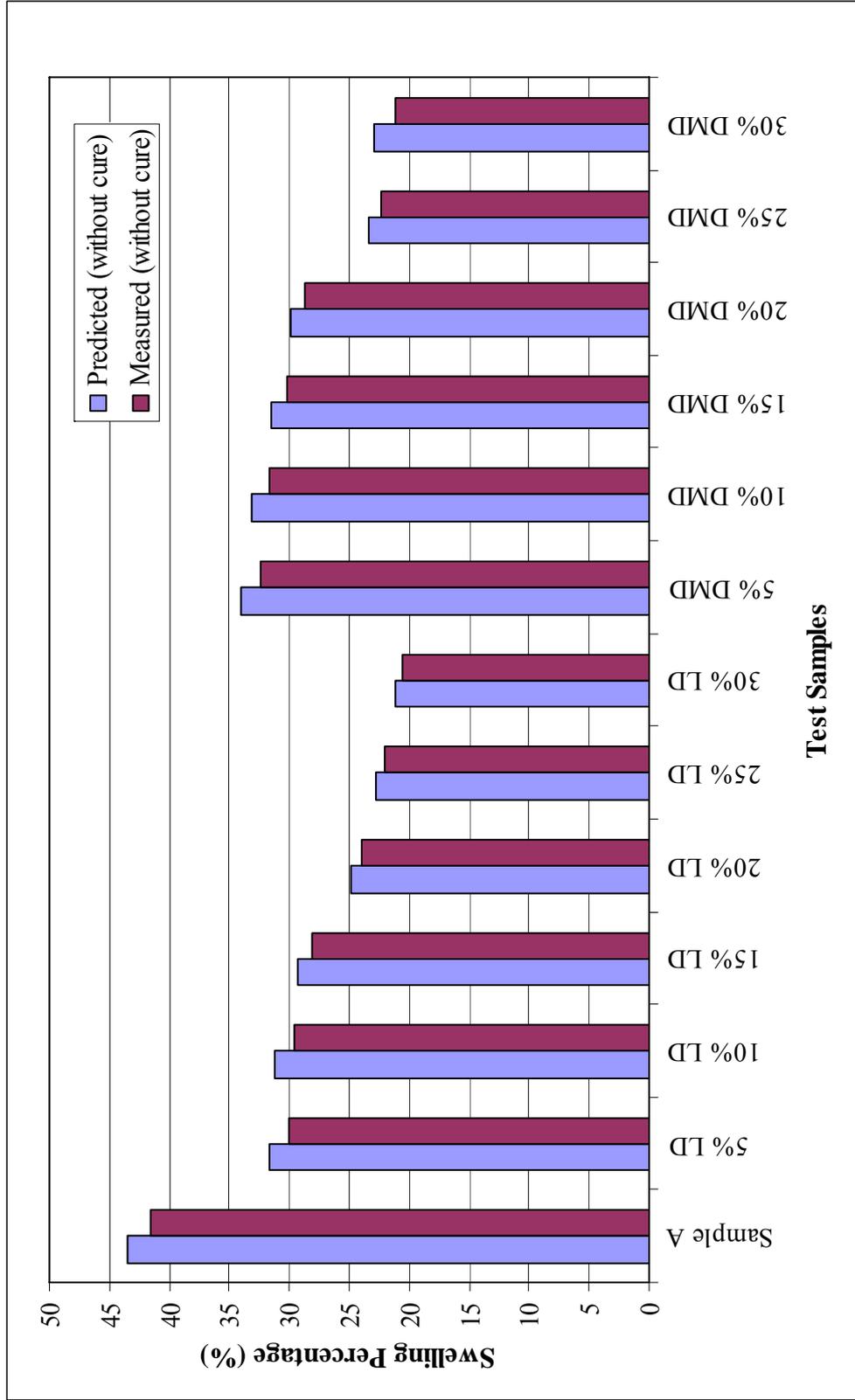


Figure 4.33 Measured and Predicted Swelling Percentage Graph for Test Samples

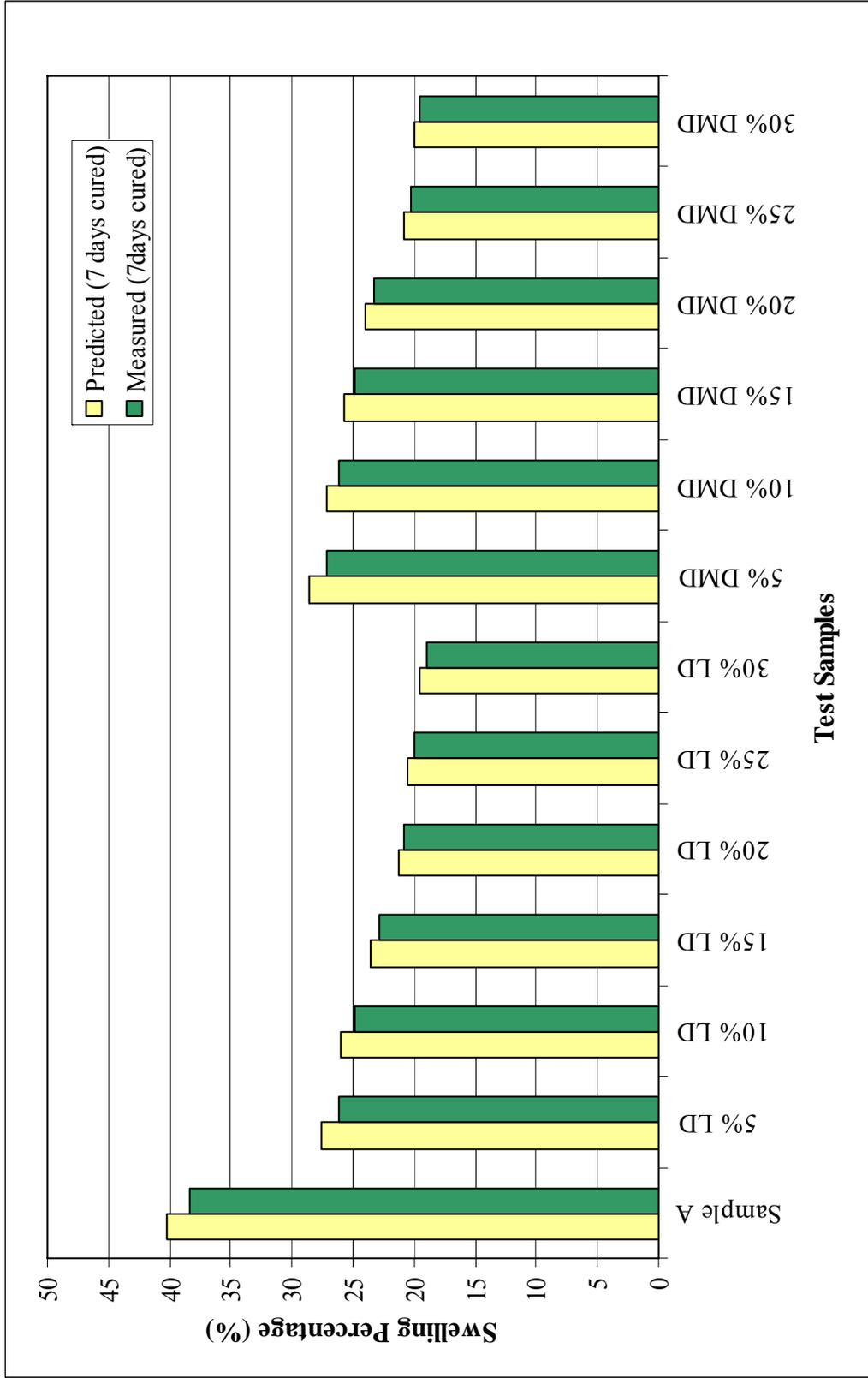


Figure 4.34 Measured and Predicted Swelling Percentage Graph for 7 days Cured Test Samples

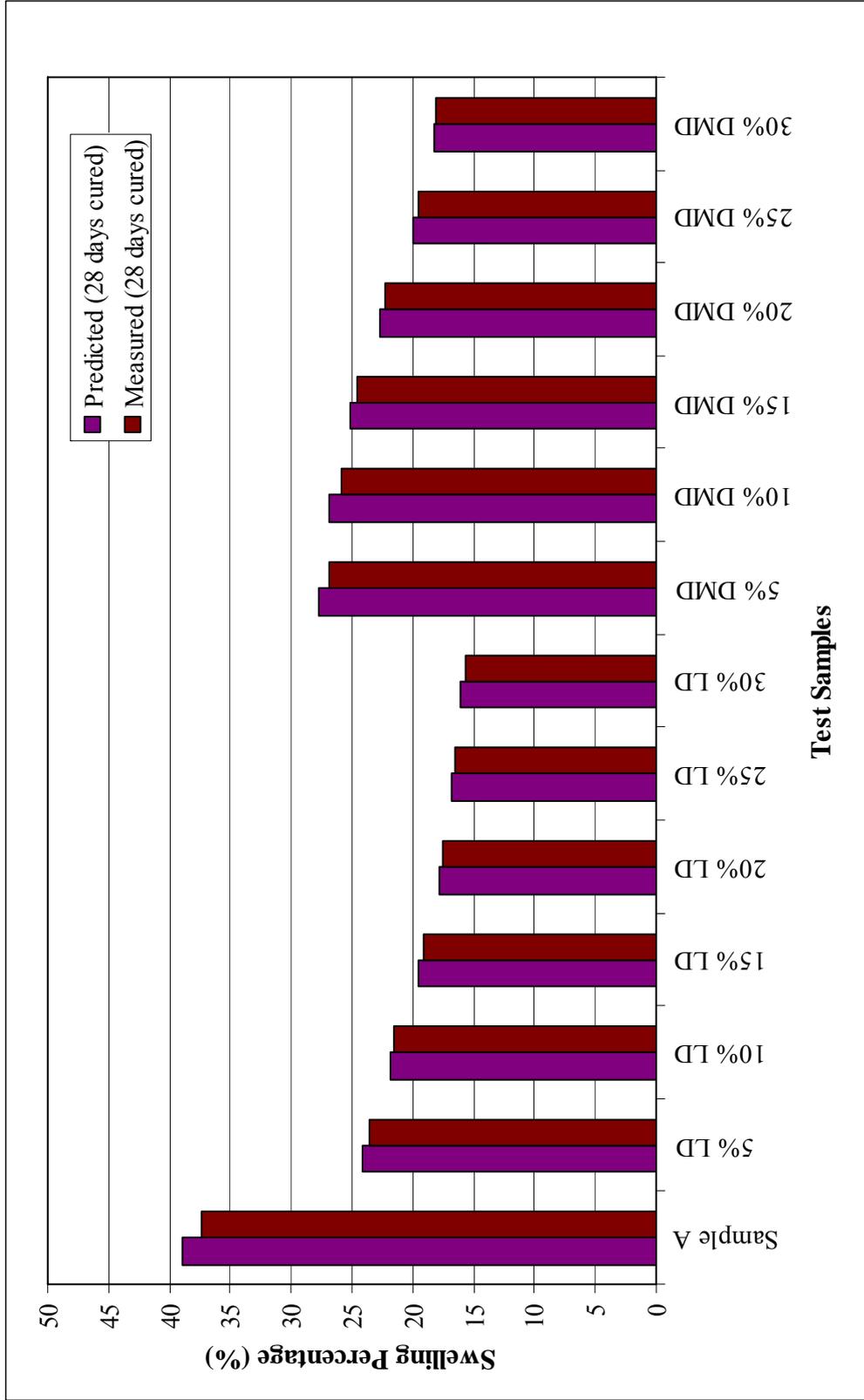


Figure 4.35 Measured and Predicted Swelling Percentage Graph for 28 days Cured Test Samples

Table 4.6 Predicted and Measured Swell Values for the Test Samples

Sample	Samples without Cure			7 Days Cured Samples			28 Days Cured Samples		
	Measured Swell (%)	Predicted Swell (%)	Difference (%)	Measured Swell (%)	Predicted Swell (%)	Difference (%)	Measured Swell (%)	Predicted Swell (%)	Difference (%)
Sample A	41.5	43.5	2.0	38.4	40.2	1.8	37.3	38.9	1.6
5% LD	30.0	31.6	1.6	26.1	27.6	1.5	23.5	24.2	0.7
10% LD	29.6	31.2	1.6	24.8	26	1.2	21.6	21.9	0.3
15% LD	28.1	29.3	1.2	22.8	23.5	0.7	19.1	19.6	0.5
20% LD	23.9	24.9	1.0	20.9	21.3	0.4	17.5	17.8	0.3
25% LD	22.1	22.8	0.7	20.0	20.6	0.6	16.5	16.8	0.3
30% LD	20.6	21.1	0.5	19.0	19.5	0.5	15.7	16.1	0.4
5% DMD	32.4	34.0	1.6	27.1	28.6	1.5	26.9	27.8	0.9
10% DMD	31.6	33.1	1.5	26.1	27.1	1.0	25.8	26.8	1.0
15% DMD	30.2	31.5	1.3	24.9	25.7	0.8	24.5	25.1	0.6
20% DMD	28.7	29.9	1.2	23.3	24.0	0.7	22.2	22.7	0.5
25% DMD	22.4	23.3	0.9	20.2	20.9	0.7	19.6	20.0	0.4
30% DMD	21.1	22.9	1.8	19.5	19.9	0.4	18.1	18.2	0.1
(%) Mean of Differences	1.3			0.9			0.6		
(%) Standard Deviation of Differences	0.440			0.457			0.396		

CHAPTER 5

DISCUSSIONS OF TEST RESULTS

5.1. Free Lime Content Test on Stabilizers

Free Lime Content Test was considered as a preliminary test for lime content of waste limestone dust and waste dolomitic marble dust before chemical analysis from METU Central Laboratory. This test resulted in that both of the stabilizers have considerable amount of lime. Lime content of waste limestone dust was higher than that of waste dolomitic marble dust, according to free lime content test. Chemical analysis test results from METU Central Laboratory states same relationship.

5.2. Effect of Stabilizers on Free Swell Ratio

According to Free Swell Ratio Test proposed by Prakash and Sridharan (2004), Free Swell Ratio, FSR, of Sample A was 2.1. According to classification of soils based on free swell ratio, Sample A could be considered as High Swelling soil and dominant clay mineral type was Montmorillonite. FSR decreased up to 1.0 for waste limestone dust and 1.1 for waste dolomitic marble dust, with increasing stabilizer percent. After 20 % addition of stabilizers a dramatically decrease of Free Swell Ratio occurred.

5.3. Effect of Stabilizers on Grain Size Distribution of Expansive Soil

Addition of stabilizers shifted the grain size distribution curve of Sample A to coarser side. In order to distinguish whether this shifting due to the addition of silt-sized particles or pozzolanic reactions between Sample A and stabilizers, calculated and measured grain size distribution curves of waste limestone dust sample are presented in Appendix C. As it can be seen in Figure C.1 and C.2 for grain size distribution curves of waste limestone dust samples, the calculated curves are on generally, finer side. However, these curves are similar showing that there is an effect of pozzolanic reactions causing the flocculation of clay particles but not so much. Concluding, the shifting of grain size distribution curves of mainly due to addition of coarser material to Sample A.

5.4. Effect of Stabilizers on the Liquid Limit of Expansive Soil

Liquid limit of Sample A decreased with addition of stabilizers. Liquid limit of Sample A decreased as the stabilizer percentage increased.

Decrease in liquid limit of Sample A was in the order of 18 % by minimum addition of stabilizers. The maximum reduction was 33.4 % for waste limestone dust added samples and that of 30.3 % for waste dolomitic marble dust added samples (Table 5.1).

5.5. Effect of Stabilizers on the Plastic Limit of Expansive Soil

Plastic Limit of Sample A increased with addition of stabilizers. Plastic Limit of Sample A increased as the stabilizer percentage increased. Increase in plastic limit of Sample A was in the order of only 1-3 % by minimum addition of stabilizers. The maximum increase was 21.5 % for waste limestone dust

added samples and that of 23.3 % for waste dolomitic marble dust added samples (Table 5.1).

Table 5.1 Percentage Changes in Liquid Limit (LL), Plastic Limit (PL), Plasticity Index (PI), Shrinkage Limit (SL), Shrinkage Index (SI) and Activity

Samples	(%) Change in LL	(%) Change in PL	(%) Change in PI	(%) Change in SL	(%) Change in SI	(%) Change in Activity
Sample A	0	0	0	0.0	0.0	0.0
5% LD	-17.2	1.3	-23.0	9.3	-23.2	-19.0
10% LD	-19.7	4.9	-27.4	10.4	-26.5	-21.2
15% LD	-24.0	6.8	-33.6	29.7	-36.1	-24.7
20% LD	-25.5	15.7	-38.4	38.5	-39.9	-25.2
25% LD	-30.4	19.1	-45.9	60.4	-50.9	-31.9
30% LD	-33.4	21.5	-50.6	65.4	-55.6	-33.6
5% DMD	-19.1	3.2	-26.1	10.4	-25.8	-11.2
10% DMD	-20.8	4.9	-28.9	35.2	-33.4	-8.5
15% DMD	-24.1	9.9	-34.7	47.8	-40.3	-9.6
20% DMD	-26.6	12.4	-38.8	52.7	-44.5	-10.1
25% DMD	-29.6	17.8	-44.5	54.4	-48.6	-9.0
30% DMD	-30.3	23.3	-47.1	57.7	-50.2	-6.4

+: Increase; -: Decrease

5.6. Effect of Stabilizers on the Plasticity Index of Expansive Soil

Plasticity index is expressed as the difference between plastic limit and liquid limit. Since liquid limit decreased and plastic limit increased, plasticity index considerable decreased with the addition of stabilizers increased. The maximum reduction was 50.6 % for waste limestone dust added samples and that of 47.1 % for waste dolomitic marble dust added samples (Table 5.1).

5.7. Effect of Stabilizers on the Shrinkage Limit of Expansive Soil

Shrinkage limit of Sample A increased as the stabilizer percentage increased. Relatively, smooth increase occurred in waste dolomitic marble dust added samples. On the other hand, for waste limestone added samples, shrinkage limit jumped between 20% LD and 25% LD samples.

The minimum increase was in the order of 10.0 % for both stabilizers and the maximum increase 65.4 % for waste limestone dust added samples, and that of 57.7 % for waste dolomitic marble dust added samples (Table 5.1).

5.8. Effect of Stabilizers on the Shrinkage Index of Expansive Soil

Shrinkage index of Sample A decreased with addition of stabilizers. Shrinkage index of Sample A decreases as the stabilizer percentage increased.

For only 5 % stabilizer added samples, the reduction of shrinkage index is approximately 24.5 %. The maximum reduction of shrinkage index is 55.6 % for waste limestone dust added samples, and 50.2 % for waste dolomitic marble dust added samples (Table 5.1).

5.9. Effect of Stabilizers on the Activity and Swelling Potential of Expansive Soil

Activity of Sample A decreased with addition of stabilizers. Activity of Sample A decreased as waste limestone dust percentage increased. However, there is no consistent relationship between activity and the percentage of waste dolomitic marble dust (Table 5.1). Also, according to Seed et.al (1962), swelling potential of Sample A decreased with addition of stabilizers (Figure 4.6).

5.10. Effect of Stabilizers on the Specific Gravity of Expansive Soil

Since the specific gravity of the samples were higher than that of Sample A ($G_{sA}=2.42$), specific gravity of Sample A increases with addition of stabilizers. Specific gravity of waste dolomitic marble dust ($G_{sDMD}=2.80$) was higher than specific gravity of waste limestone dust ($G_{sLD}=2.68$). Thus, the specific gravities of waste dolomitic marble dust added samples were higher than those waste limestone dust added samples.

5.11. Effect of Stabilizers on the Swell Percentage of Expansive Soil

Swell percentages of samples decreased with addition of stabilizers. Firstly, this was because of replacing expansive soil with non-expansive material. Obviously, with increasing amount of stabilizer, the proportion of expansive soil decreased.

For waste limestone dust added samples, the reduction of swelling percentage is 27.7 % for only 5 % addition of stabilizer, and maximum reduction is 50.4 % for 30 % addition (Table 5.2).

For waste dolomitic marble dust added samples, the reduction of swelling percentage is 21.9 % for 5 % addition of stabilizer, and maximum reduction is 49.2 % for 30 % addition (Table 5.2).

Waste limestone dust having higher lime content is more effective than waste dolomitic marble dust on swell percentage. The reduction in swell percentage is due to chemical reactions between lime and expansive soil.

Table 5.2 Percentage Changes in Swelling Percentage (%) and t_{50} (min) for samples without Cure, 7 days cured and 28 days cured

Sample	Samples Without Cure		7 Days Samples		28 Days Samples	
	(%) Change in Swelling Percentage	(%) Change in t_{50}	(%) Change in Swelling Percentage	(%) Change in t_{50}	(%) Change in Swelling Percentage	(%) Change in t_{50}
Sample A	0.0	0.0	0.0	0.0	0.0	0.0
5% LD	-27.7	-14.6	-32.0	-15.1	-37.0	-56.3
10% LD	-28.7	-20.9	-35.4	-23.7	-42.1	-57.2
15% LD	-32.3	-28.9	-40.6	-49.8	-48.8	-59.0
20% LD	-42.4	-37.2	-45.6	-54.7	-53.1	-62.0
25% LD	-46.7	-56.1	-47.9	-57.6	-55.8	-71.2
30% LD	-50.4	-57.7	-50.5	-64.5	-57.9	-72.9
5% DMD	-21.9	-16.6	-29.4	-19.2	-27.9	-38.9
10% DMD	-23.9	-20.6	-32.0	-26.1	-30.8	-43.7
15% DMD	-27.2	-25.3	-35.2	-36.7	-34.3	-48.0
20% DMD	-30.8	-26.9	-39.3	-38.0	-40.5	-52.0
25% DMD	-46.0	-30.0	-47.4	-52.7	-47.5	-57.2
30% DMD	-49.2	-32.0	-49.2	-62.9	-51.5	-70.7

+: Increase; -: Decrease

5.12. Effect of Stabilizers on the Rate of Swell of Expansive Soil

t_{50} , the time necessary for 50 % swell of total swell, decreased with addition of stabilizers, and thus, rate of swell of samples decreased.

For waste limestone dust added samples, the reduction of rate of swell was higher than waste dolomitic marble dust added samples. The maximum decrease in rate of swell is 57.7 % for waste limestone dust added samples and 32.0 % for waste dolomitic marble dust added samples. The reduction was nearly same for 25 % LD and 30 % LD samples (Table 5.2).

5.13. Effect of Cure on the Swell Percentage

Considerable reductions of swell percentage occurred in each sample with addition of stabilizers after curing. Also, swell percentages of Sample A were tested to normalize all samples.

Especially, limestone dust added samples show higher reduction in swelling percentage after cured. The maximum decrease in swell percentage is 57.9 % for waste limestone dust added samples and 51.5 % for waste dolomitic marble dust added samples after curing (Table 5.2). Even, 5 % addition of waste limestone dust resulted in 37.0 % reduction in swelling percentage after 28 days curing (Table 5.2). Except 5 %, 10 % and 15 % DMD samples, gradual decrease of swelling percentage can be summarized as;

$$(\Delta H/H)_{\text{without cure}} > (\Delta H/H)_{7 \text{ days cured}} > (\Delta H/H)_{28 \text{ days cured}}$$

5.14. Effect of Cure on Rate of Swell

Reductions of rate of swell occurred in each sample with addition of stabilizers. In other words, time passed for 50 % of total swelling was lowered by curing. Also, rate of swell of Sample A was tested to normalize all samples. Especially, 28 days cured samples had much tendency to decrease in rate of swell.

The maximum decrease in t_{50} is 72.9 % for waste limestone dust added samples and 70.7 % for waste dolomitic marble dust added samples after curing (Table 5.2). Even 5 % addition of waste limestone dust resulted in 56.3 % reduction in t_{50} after 28 days curing (Table 5.2). Gradual decrease of swelling percentage can be summarized as;

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

5.15. Discussion on Prediction of Swell Percentage

The mean of differences between predicted and measured swell percentages of samples are 1.3 %, 0.9 % and 0.6 %, for samples without cure, 7 days cured samples and 28 days cured samples, respectively. These small differences show that the prediction of swell percentages methodology, proposed by Sivapullaiah, et.al (1996), gives consistent results for this study. Also, the standard deviation of 28 days cured samples has the minimum value, 0.396 %, thus, 28 days cured samples have the most consistent results.

5.16. General Discussion on Test Results

The addition of waste limestone dust and dolomitic marble dust to the expansive clay: (a) reduces the clay content and a corresponding increase in the percentage of coarse particles; (b) reduces the liquid limit (LL), (c) raises the shrinkage limit (SL) and (d) reduce the plasticity index (PI) of soil, and hence swelling potential. In one process, a base exchange occurs with the calcium ions of waste limestone dust and dolomitic marble dust replacing sodium on the surface of the expansive clay particle. The net result is a low base exchange capacity for the particle with a resulting lower swelling potential. Addition of waste limestone dust and dolomitic marble dust resulted in the formation of aggregations which reduced the swelling potential of the soil.

CHAPTER 6

CONCLUSION

In this study, the suitability of waste limestone dust and waste dolomitic marble dust as stabilizers for swelling potential of an expansive soil was studied. Waste limestone dust and waste dolomitic marble dust added to Sample A. According to test results, the following outcomes can be summarized:

1. Addition of waste limestone dust and waste dolomitic marble dust decreased liquid limit, plasticity index and shrinkage index, increased plastic limit and shrinkage limit of Sample A.
2. Activity of samples decreased by addition of stabilizers. Activity of Sample A decreased more as waste limestone dust percentage increased. However, there is no consistent relationship between activity and the percentage of waste dolomitic marble dust.
3. Free swell ratio of Sample A decreased with addition of stabilizers. As the percentage of stabilizer increased, free swell ratio decreased.
4. By addition of stabilizers, the swelling percentage decreased considerably. The reduction was higher for waste limestone added samples having more lime content.

5. By addition of stabilizers, the t_{50} values were decreased. In other words, samples having more stabilizers reached the 50 % of total swell quicker.
6. Swelling percentage and rate of swell of samples decreased by curing. Curing was more effective for waste limestone dust added samples. Also, curing was more effective for rate of swell of samples than swelling percentage.
7. The predicted swell and measured swell percentages of samples were close to each other.
8. Both waste limestone dust and dolomitic marble dust can be used as stabilizers for expansive soils. But, the effectiveness of stabilizers depends on their chemical content.

Recommendations for Future Research

The swell potential tests were performed on samples treated with waste limestone dust and dolomitic marble dust in this study. The swell potential test to measure the swell percent is the most reliable method for accessing the actual swell potential, but for the full scale characterization of the soil behavior, the microfabric and mineralogical characteristics of untreated and treated soil should be determined by scanning electron microscope and X-Ray diffraction studies.

REFERENCES

1. Al-Rawas, A.A., Taha R., Nelson, J.D., Al-Shab, T.B. and Al-Siyabi, H., A Comparative Evaluation of Various Additives Used in the Stabilization of Expansive Soils, *Geotechnical Testing Journal*, Vol. 25, No. 2, pp. 199-209, 2002
2. Almeida N., Branco, F., Santos, J.R., Recycling of Stone Slurry in Industrial Activities: Application to Concrete Mixtures, *Building and Environment*, Vol. 42, pp. 810–819, 2007.
3. ASTM, Standard Test Method for One-Dimensional Consolidation Properties of Soils, *Annual Book of ASTM Standards*, D 2435-90, Vol. 04 - 08, pp. 313-323, 1993.
4. ASTM, Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils, *Annual Book of ASTM Standards*, D4546 - 03, Vol. 04 -08, pp. 1-7,1999
5. ASTM, Standard Test Methods for Chemical Analysis of Limestone, Quicklime and Hydrated Lime, *Annual Book of ASTM Standards*, C 25 -93a, Vol.04.01, pp. 9-36, 1993
6. Bohn, H. L., McNeal, B.L. and O'Conner, G. A., *Soil Chemistry*, 2nd Edition, John Wiley Inc., New York, 1985
7. Craig, R.F., *Soil Mechanics*, 6th Edition, Spon Press, London, 1997.
8. Chen, F. H., *Foundations on Expansive Soils*, Elsevier Scientific Pub. Co., Amsterdam, 1975
9. Çelik, M.Y., Mermer Atıklarının (Parça-Tozların) Değerlendirilmesi, *M. S. Thesis*, Afyon Kocatepe University, Natural and Applied Sciences, 119 pages, 1996.

10. Çelik, M.Y., Sabah, E., Geological and Technical Characterization of İscehisar (Afyon-Turkey) Marble Deposits and the Impact of Marble Waste on Environmental Pollution, *Journal of Environmental Management*, Vol. 87, No.1, pp. 106-116, 2008.
11. Çetiner, S., I., Stabilization of Expansive Soils by Çayırhan Fly Ash and Desulphogypsum, *M.S. Thesis*, METU, Turkey, 107 pages, 2004.
12. Çokça, E., 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, 2001.
13. Dakshanamurthy, V., A new method to predict swelling using a hyperbolic equation, *Geotechnical Engineering*, Vol. 9, pp.29-38, 1978.
14. Dietrich, R. V. and Skinner, B. J., *Rocks and Rock Minerals*, 6th Edition, John Wiley and Sons, New York, 1979.
15. Fredlund, D.G. and Rahardjo, H., *Soil Mechanics for Unsaturated Soils*, John Wiley and Sons Inc., New York, 1993
16. Hadi, N. A. R. A., Khoury, H. N., Suliman, M. R., Utilization of bituminous limestone ash from EL-LAJJUN area for engineering applications, *Acta Geotechnica*, Vol. 3, No. 2, pp.139-151, 2008
17. İpek, T., Stabilization of Expansive Soils Using Lime, Cement and Fly Ash, *M.S. Thesis*, METU, Turkey, 119 pages, 1998
18. Jones, D. E., and Holtz, W. G., Expansive Soils - the hidden disaster, *Civil Engineering*, ASCE 43(8), pp. 49-51, 1973
19. Kemp, J. F., *A Handbook of Rocks*, D. Van Nostrand Co. Inc., New York, 1965.
20. Kun, N., *Mermer Jeolojisi ve Teknolojisi*, Tezer Matbaası, İzmir, 2000.
21. Lal, R. and Shukla, M.K., *Principles of Soil Physics*, Marcel Decker Inc., New York, 2004
22. Lambe, T.W. and Whitman, R.V., *Soil Mechanics*, John Wiley and Sons Inc., New York, 1969.

23. Mishra, A.K., Dhawan S., Rao, S.M., Analysis of Swelling and Shrinkage Behavior of Compacted Clays, *Geotechnical and Geological Engineering*, Vol. 26, No. 3, pp. 289-298, 2008
24. Mitchell, J. K. and Soga, K., *Fundamentals of Soil Behavior*, 3rd Edition, John Wiley and Sons Inc., New York, 2005.
25. Mitchell, J. K., *Fundamentals of Soil Behavior*, John Wiley and Sons Inc., New York, 1976
26. Muntohar, A. S., and Hantoro, Influence of rice husk ash and lime on engineering properties of clayey subgrade, *Electronical Journal of Geotechnical Engineering*, 2000.
27. Muntohar, A. S., Behaviour of engineering properties of the clay blended with LRHA (lime rice husk ash). *Research Rep. from Grant*, Muhammadiyah Univ. of Yogya-Karta, Yogya-Karta, Indonesia, 1999
28. Nelson, J.D. and Miller, D.J., *Expansive Soils, Problems and Practice in Foundation and Pavement Engineering*, John Wiley and Sons Inc., New York, 1992.
29. Oates, J. A. H., *Lime and Limestone*, Wiley-VCH, Weinheim, 1998.
30. Okagbue, C. O., Stabilization of Clay Using Woodash, *Journal of Material Science in Civil Engineering*, ASCE, Vol. 19, No. 1, pp. 14-18, 2007
31. Okagbue, C. O., and Onyeobi, T. U. S., Potentials of marble dust to stabilize red tropical soils for road construction, *Engineering Geology*, 53, pp. 371-380, 1999.
32. Okagbue, C. O., and Yakubu, J. A., Limestone ash waste as a substitute for lime in soil improvement for engineering construction, *Bulletin of Engineering Geology and the Environment*, Vol. 58, No.2, 107-113, 2000.
33. Onargan, T., Köse, H., Deliormanlı, A., H., *Mermer*, TMMOB Maden Mühendisleri Odası, İzmir, 2005.
34. Oweis, I. S., Khera, R. P., *Geotechnology of Waste Management*, 2nd Edition, PWS Publishing Company, Boston, 1998.
35. Popescu, M.E., (1986), "A Comparison between the Behaviour of Swelling and of Collapsing Soils", *Engineering Geology*, Vol. 23, pp. 145-163.

36. Prakash, K. and Shidhara, A., Free Swell Ratio and Clay Mineralogy of Fine-Grained Soils, *Geotechnical Testing Journal*, Vol. 27, No.2, pp.1-6, 2004.
37. Seed, H.B., Woodward, R.J. and Lungren, R., *Prediction of Swelling Potential for Compacted Clays*, Journal of the Soil Mechanics and Foundations Division, Proc. Of the ASCE, pp. 53-87, 1962.
38. Show, K. Y., Tay, J., H., and Goh, A., T., C., Reuse of Incinerator Fly Ash in Soft Soil Stabilization, *Journal of Materials in Civil Engineering*, Vol. 15, No. 4, 2003.
39. Sivapullaiah, P. V., Pozzolanic Stabilization of Expansive Soils, *Expansive Soils: Recent Advances in Characterization and Treatment Edited By Ali-Rawas, A.A. and Goosen, M. F. A*, Taylor and Francis Group, London, 1996
40. Sivapullaiah, P.V., Shidharan, A. And Stalin, V. K., Swelling behaviour of soil-bentonite mixtures, *Canadian Geotechnical Journal*, Vol. 33, pp. 808-814, 1996.
41. Tegethoff, F. W., *Calcium Carbonate: From the Cretaceous Period into the 21st Century*, Birkhauser, Switzerland, 2001.
42. Terzaghi, K. and Peak R.B., *Soil Mechanics in Engineering Practice*, 2nd Edition, Wiley Inc., New York, 1967.
43. Terzaghi, K., Peak R.B. and Mesri, G., *Soil Mechanics in Engineering Practice*, 3rd Edition, John Wiley and Sons Inc., New York, 1996.
44. Türker, D., Stabilization of Expansive Soils with C/F Type Fly Ash and Sand, *M.S. Thesis*, METU, Turkey, 103 pages, 2000
45. USGS, <http://pubs.usgs.gov/of/2001/of01-041/htmldocs/clay.htm>, last visited on 2nd January 2009
46. Wayne, A.C., Mohamed, A.O. and El-Fatih, M.A., Construction on Expansive Soils in Sudan, *Journal of Construction Engineering and Management*, Vol. 110, No. 3, pp. 359-374, 1984
47. Winterkorn, H., F., and Pamukçu, S., Soil Stabilization and Grouting, *Foundation Engineering Handbook Edited by Hsai-Yang Fang*, 2nd Edition, Van Nostrand Reinhold, New York, 1990

48. Yıldız, A. H., Mermer Toz Atıklarının Yol İnşaatında Değerlendirilmesi, *Ph.D. Thesis*, Süleyman Demirel University, Civil Engineering Dept., 2008
49. Zorluer, I., and Usta, M, Zeminlerin Atık Mermer Tozu ile İyileştirilmesi, *Türkiye IV. Mermer Sempozyumu (MERSEM '2003) Bildiriler Kitabı*, pp. 305 - 311, 2003

APPENDIX A

SWELLING PERCENTAGE vs. TIME GRAPHS

Swelling Percentage vs. Time Graphs for Waste Limestone Dust and Waste Dolomitic Marble Dust added samples without cure, 7 days cured and 28 days cured are presented in Figure A.1, Figure A.2, Figure A.3, Figure A.4, Figure A.5, Figure A.6.

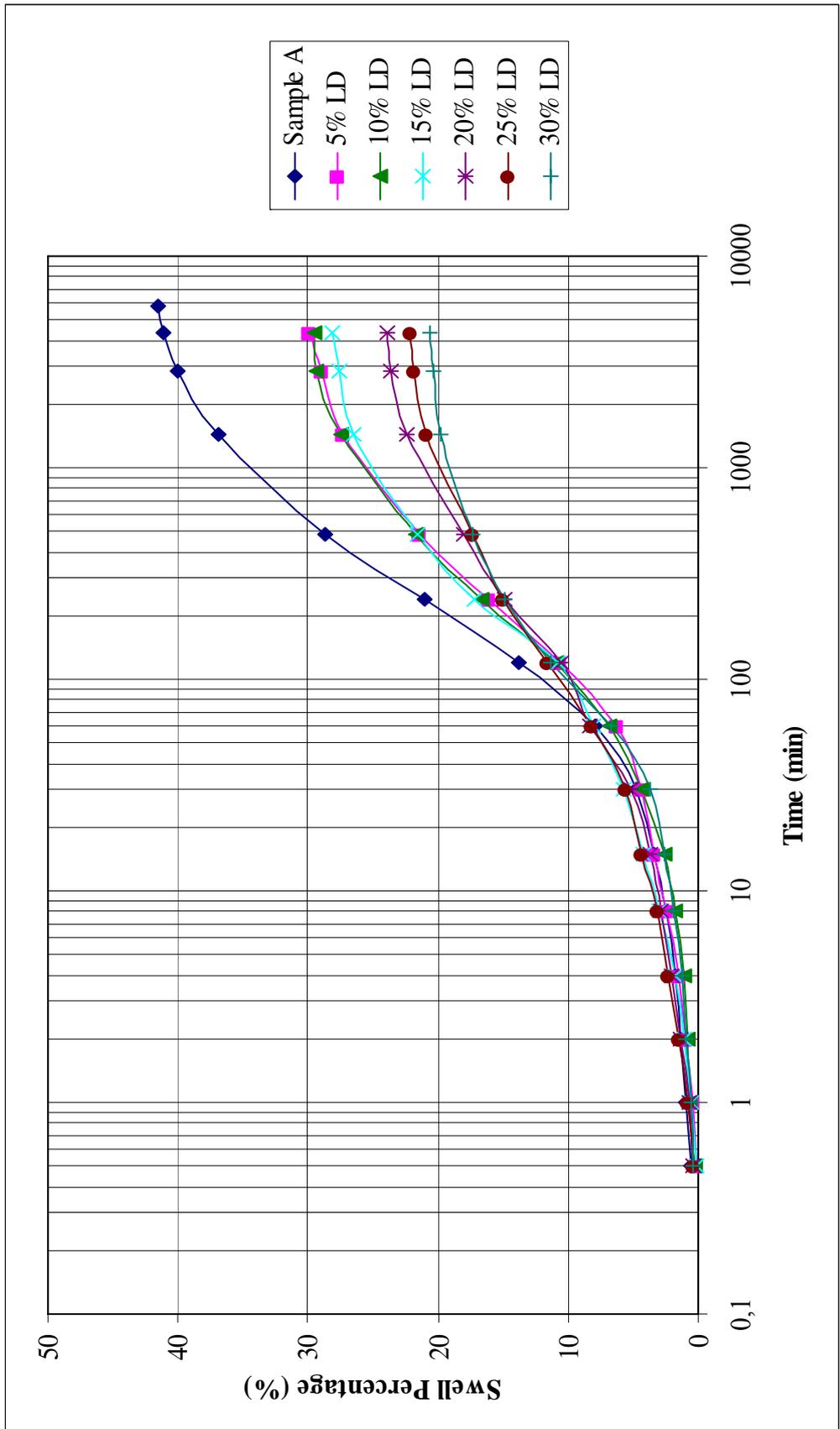


Figure A.1 Swelling Percentage vs. Time Graph for Waste Limestone Dust Added Samples

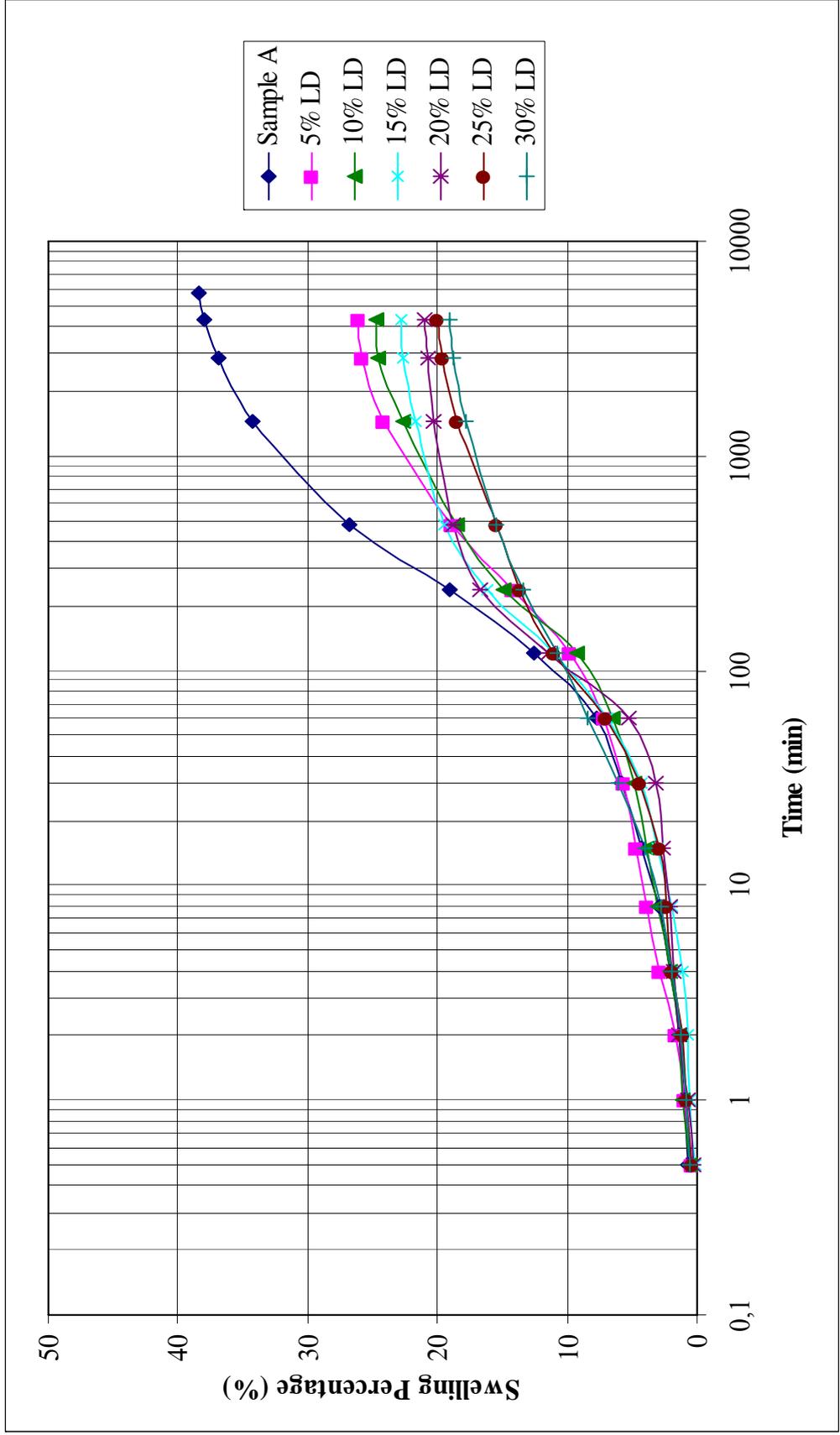


Figure A.2 Swelling Percentage vs. Time Graph for Waste Limestone Dust Added and 7 days Cured Samples

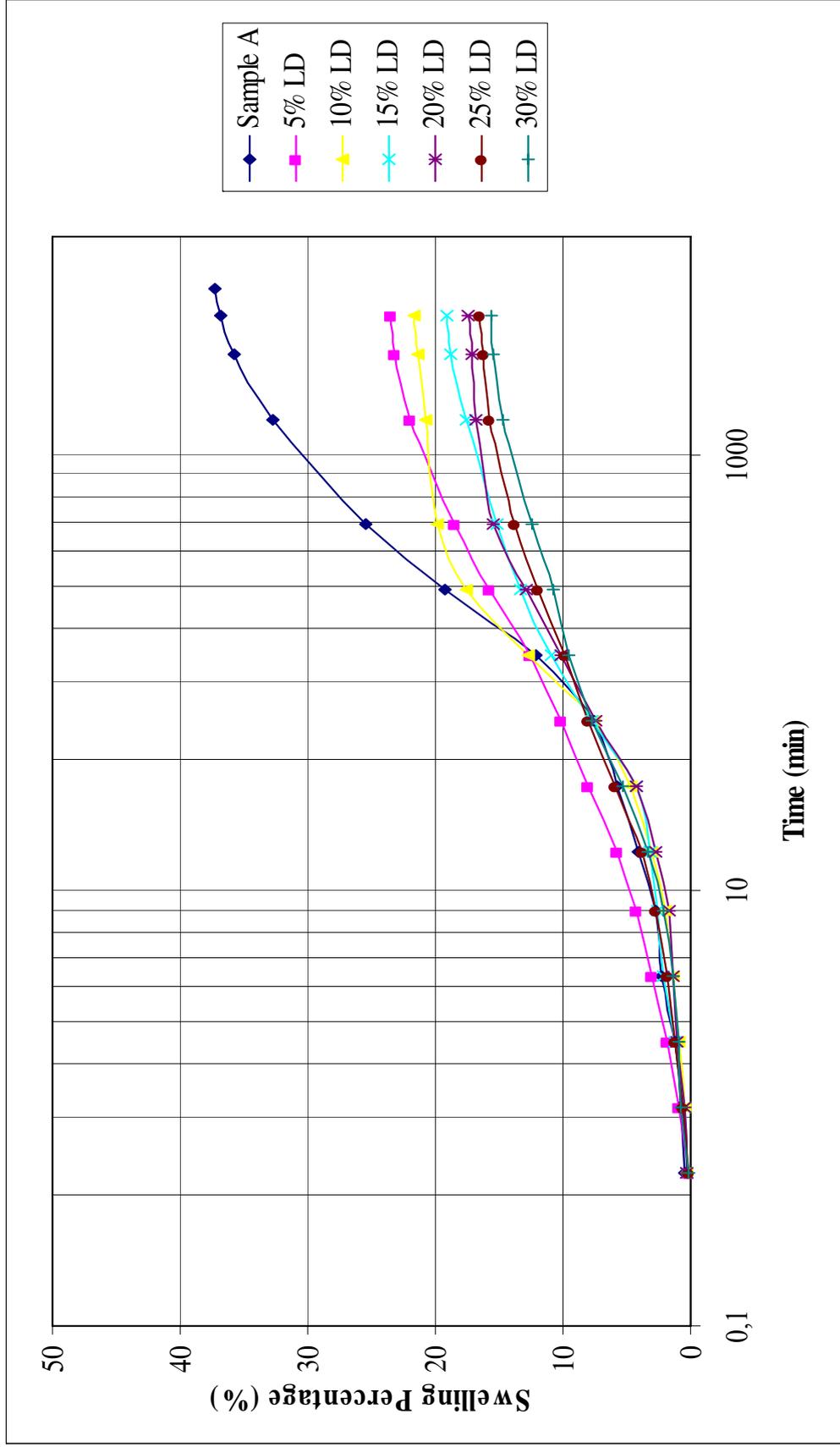


Figure A.3 Swelling Percentage vs. Time Graph for Waste Limestone Dust Added and 28 days Cured Samples

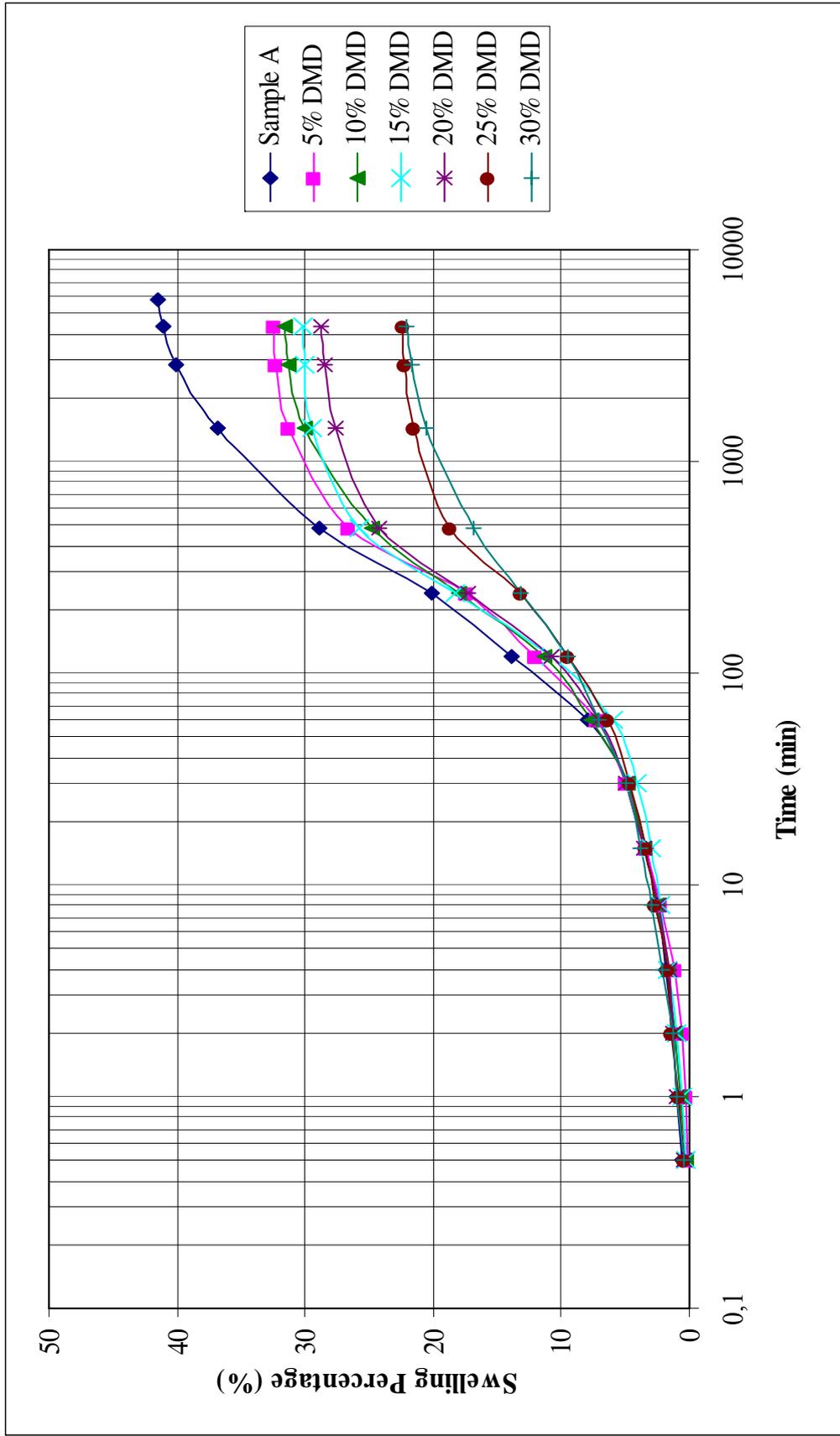


Figure A.4 Swelling Percentage vs. Time Graph for Waste Dolomitic Marble Dust Added Samples

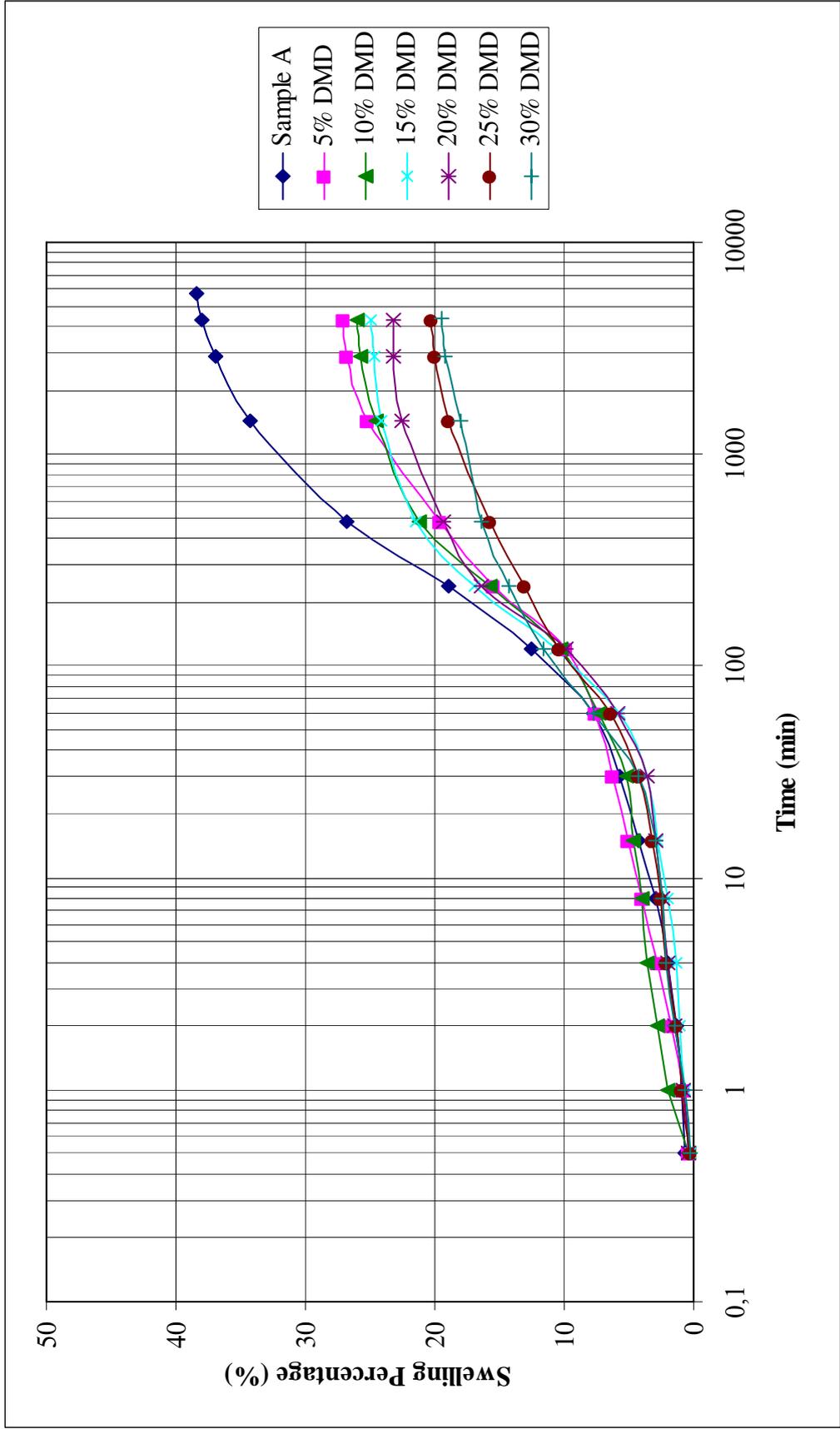


Figure A.5 Swelling Percentage vs. Time Graph for Waste Dolomitic Marble Dust Added 7 Days Cured Samples

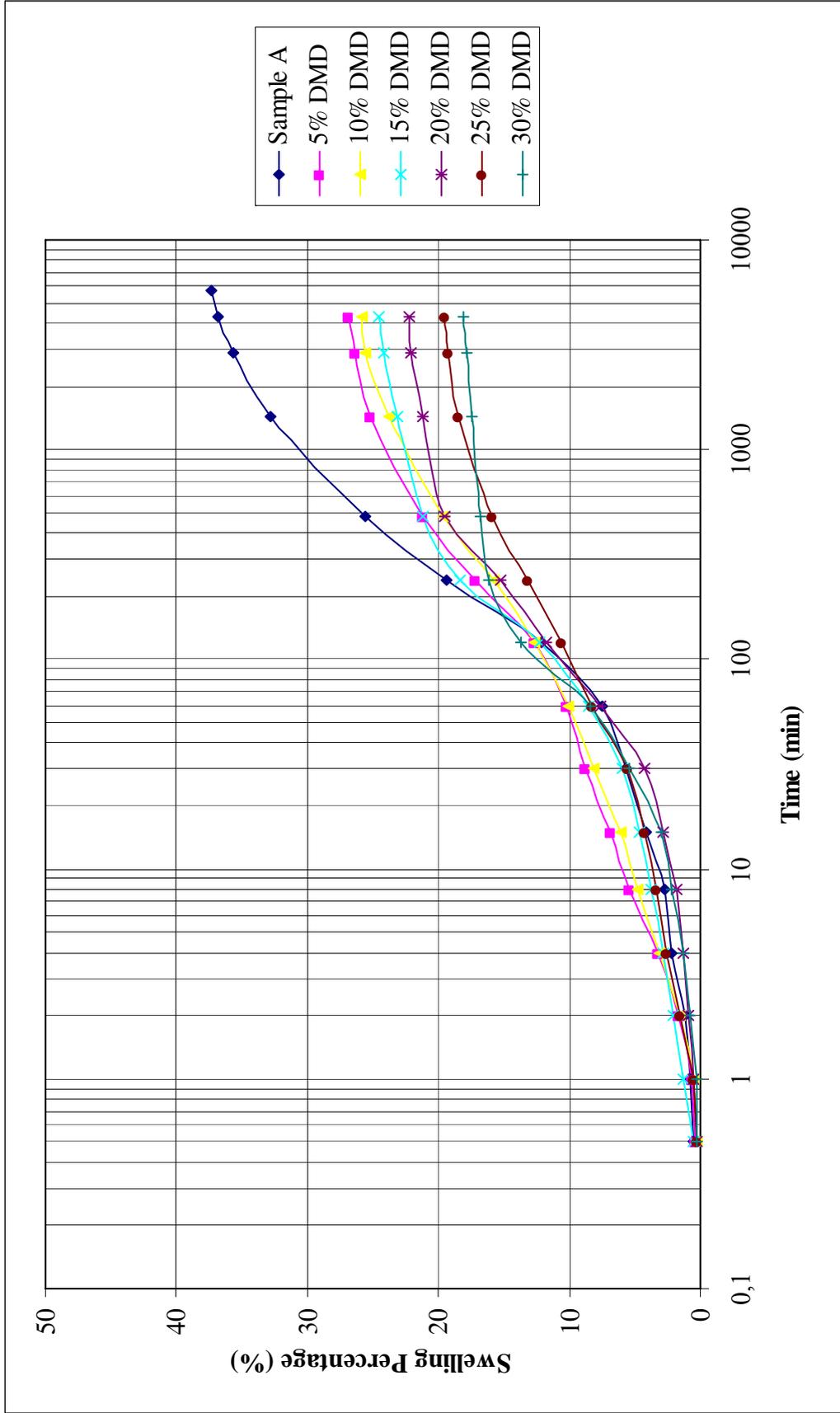


Figure A.6 Swelling Percentage vs. Time Graph for Waste Dolomitic Marble Dust Added 28 Days Cured Samples

APPENDIX B

TIME / SWELLING PERCENTAGE vs. TIME GRAPHS

Time / Swelling Percentage vs. Time Graphs for Waste Limestone Dust and Waste Dolomitic Marble Dust added samples without cure, 7 days cured and 28 days cured are presented in Figure B.1, Figure B.2, Figure B.3, Figure B.4, Figure B.5, Figure B.6.

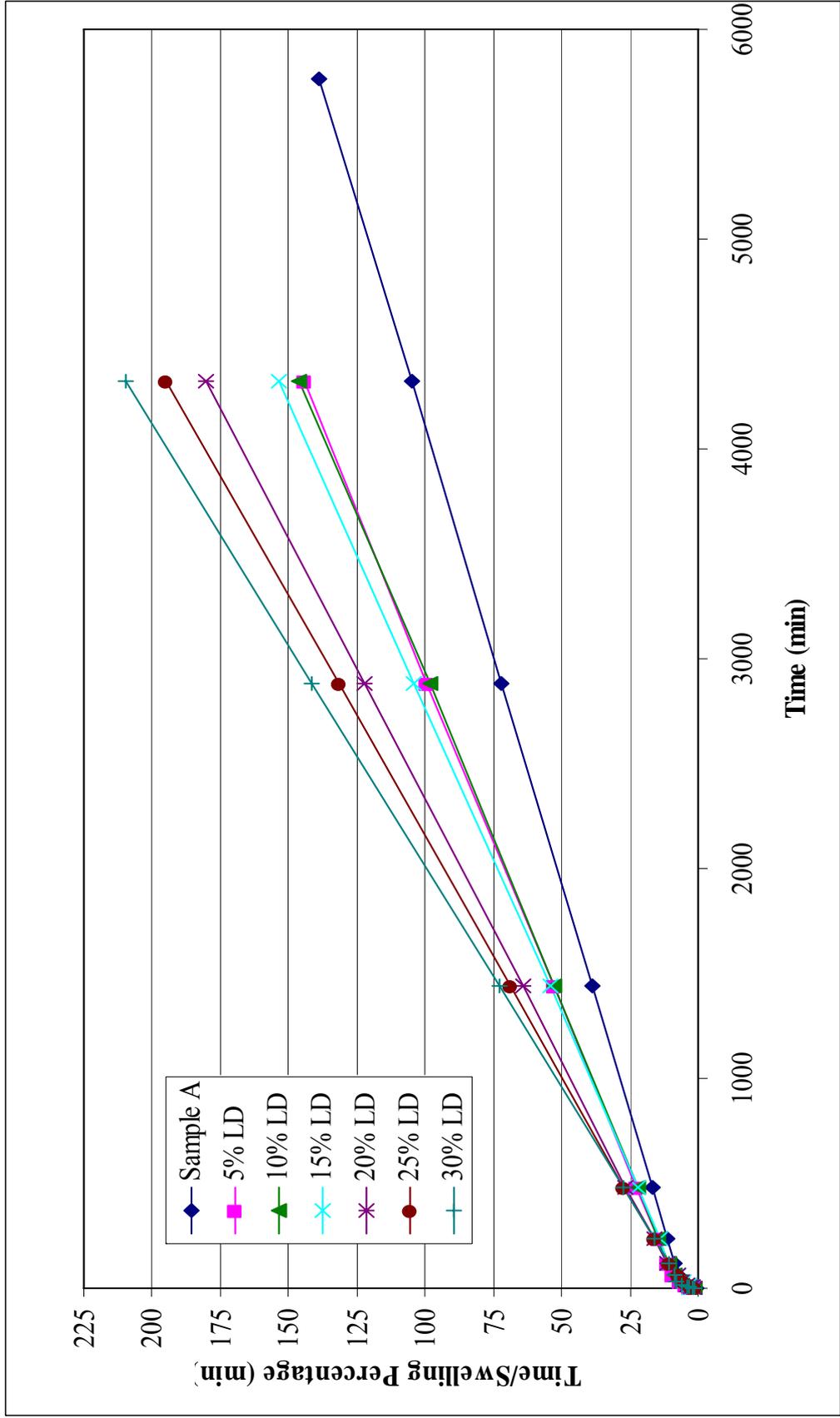


Figure B.1 Time / Swelling Percentage vs. Time Graph for Waste Limestone Dust Added Samples

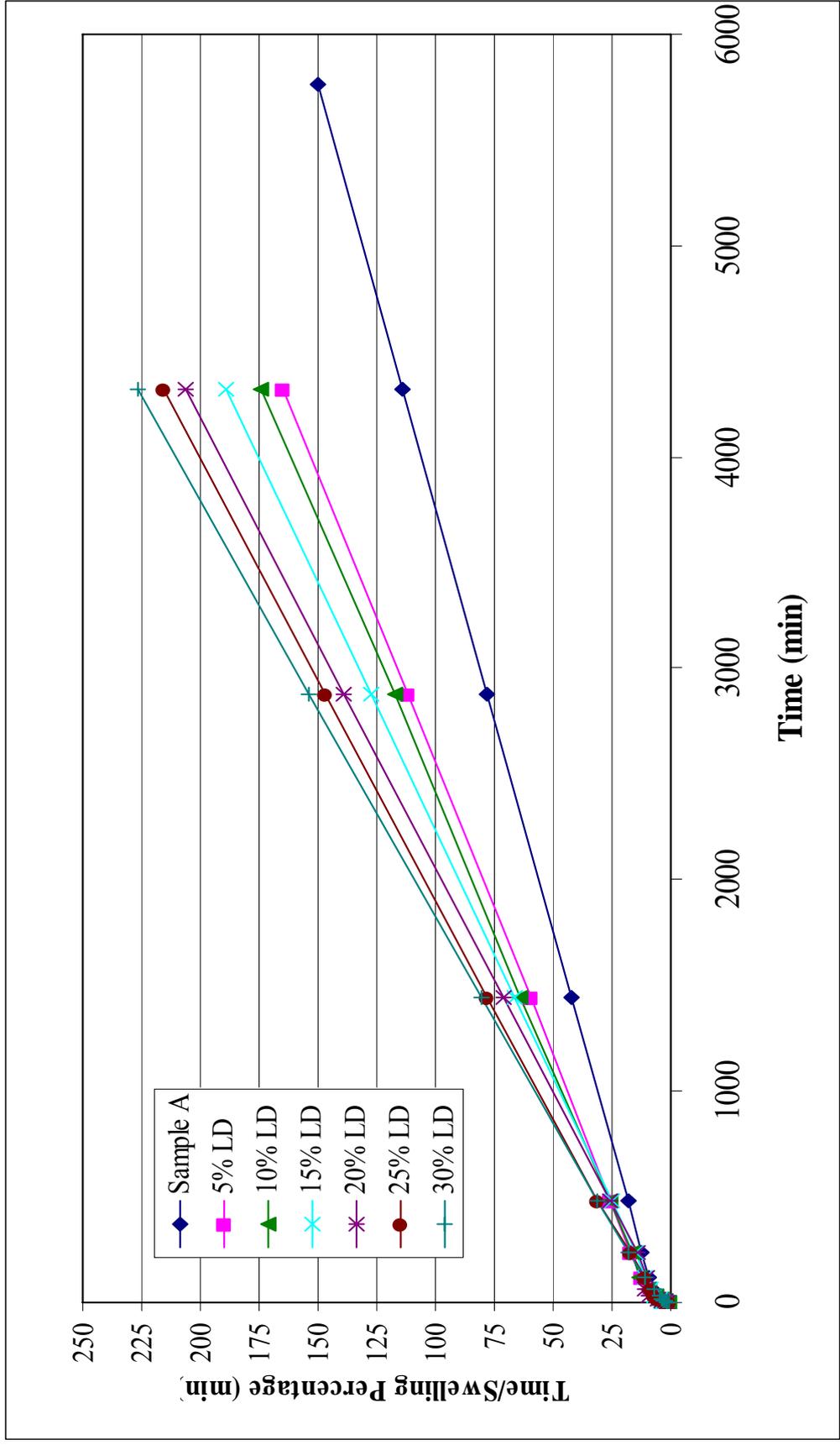


Figure B.2 Time / Swelling Percentage vs. Time Graph for Waste Limestone Dust Added and 7 Days Cured Samples

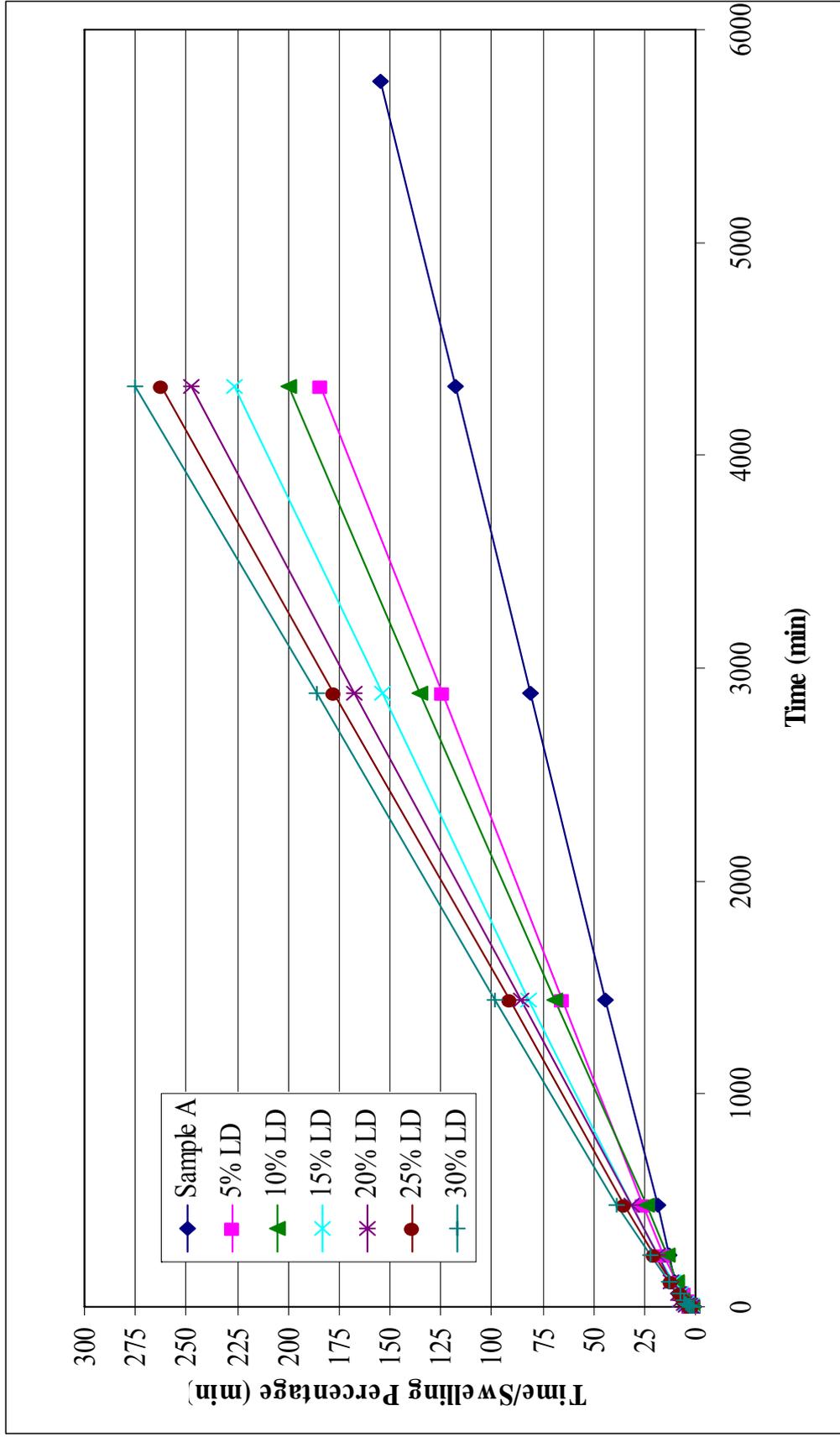


Figure B.3 Time / Swelling Percentage vs. Time Graph for Waste Limestone Dust Added and 28 Days Cured Samples

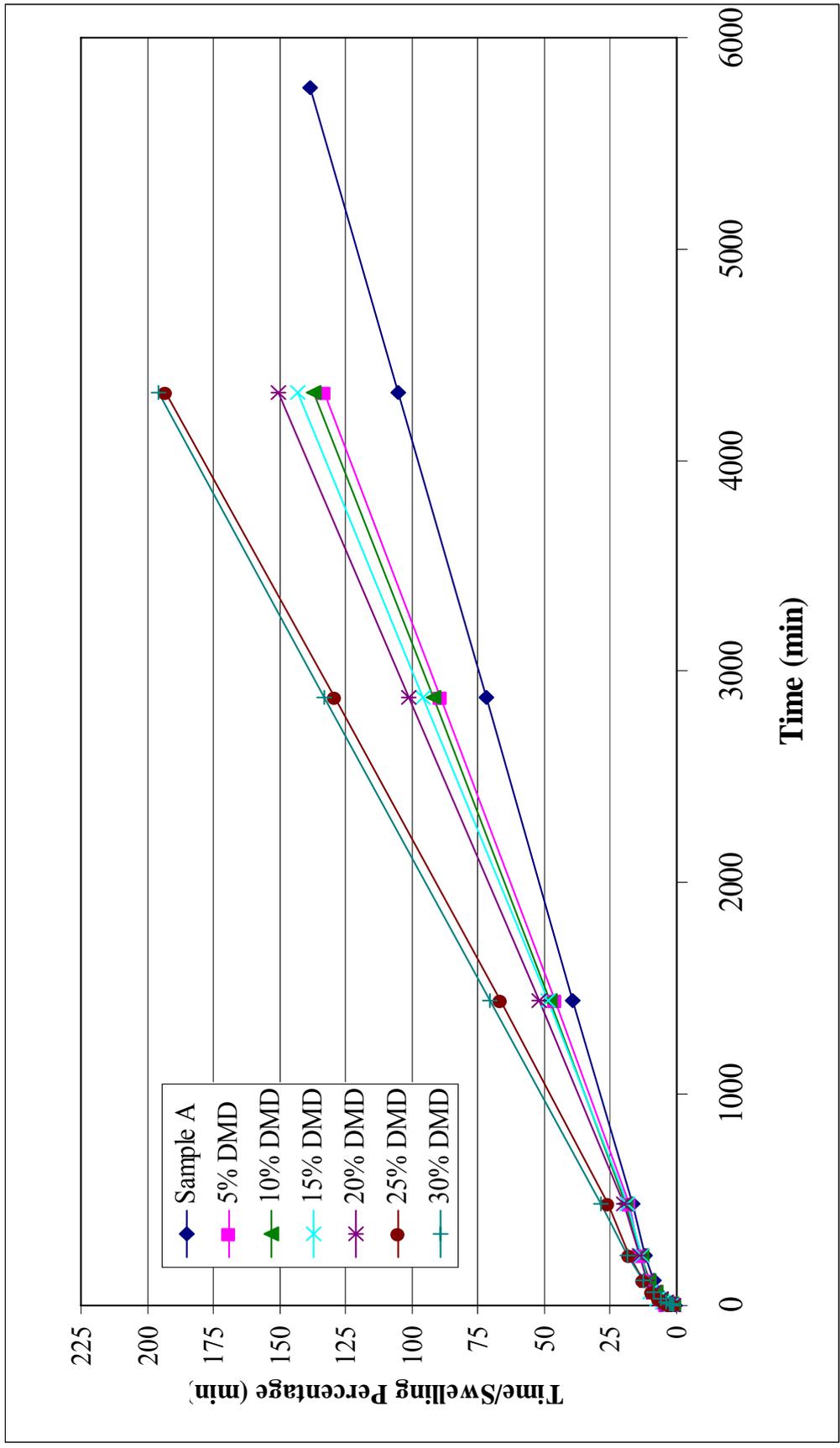


Figure B.4 Time / Swelling Percentage vs. Time Graph for Waste Dolomitic Marble Dust Added Samples

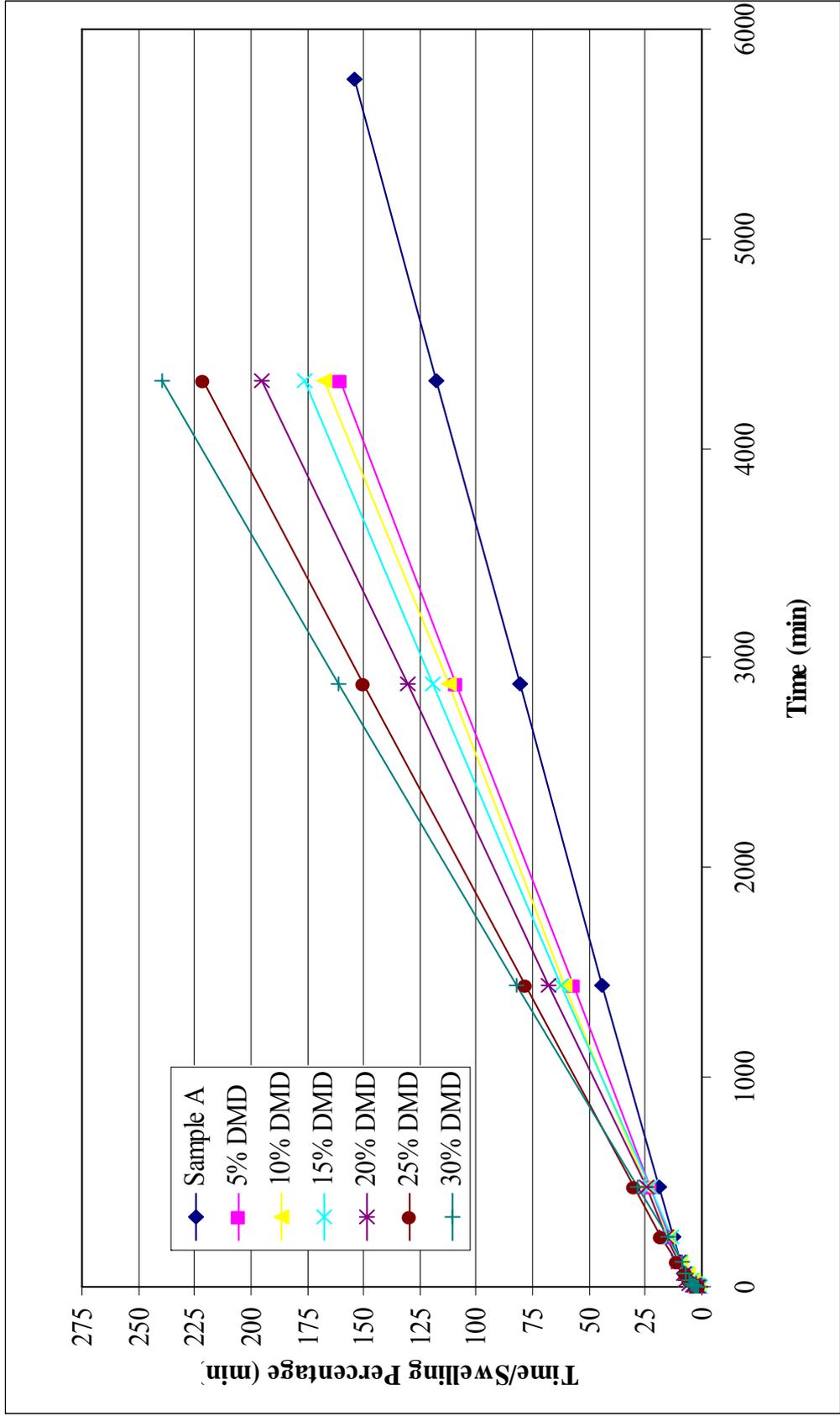


Figure B.6 Time / Swelling Percentage vs. Time Graph for Waste Dolomitic Marble Dust Added and 28 Days Cured Samples

APPENDIX C

MEASURED AND CALCULATED GRAIN SIZE DISTRIBUTION CURVES FOR WASTE LIMESTONE DUST ADDED SAMPLES

Grain Size Distribution of Sample A, Limestone Dust, Calculated and Measured Grain Size Distribution Curve of 5% LD, 15% LD and 25% LD Samples are presented in Figure C.1. Also, Grain Size Distribution of Sample A, Limestone Dust, Calculated and Measured Grain Size Distribution Curve of 10% LD, 20% LD and 30% LD are presented in Figure C.2.

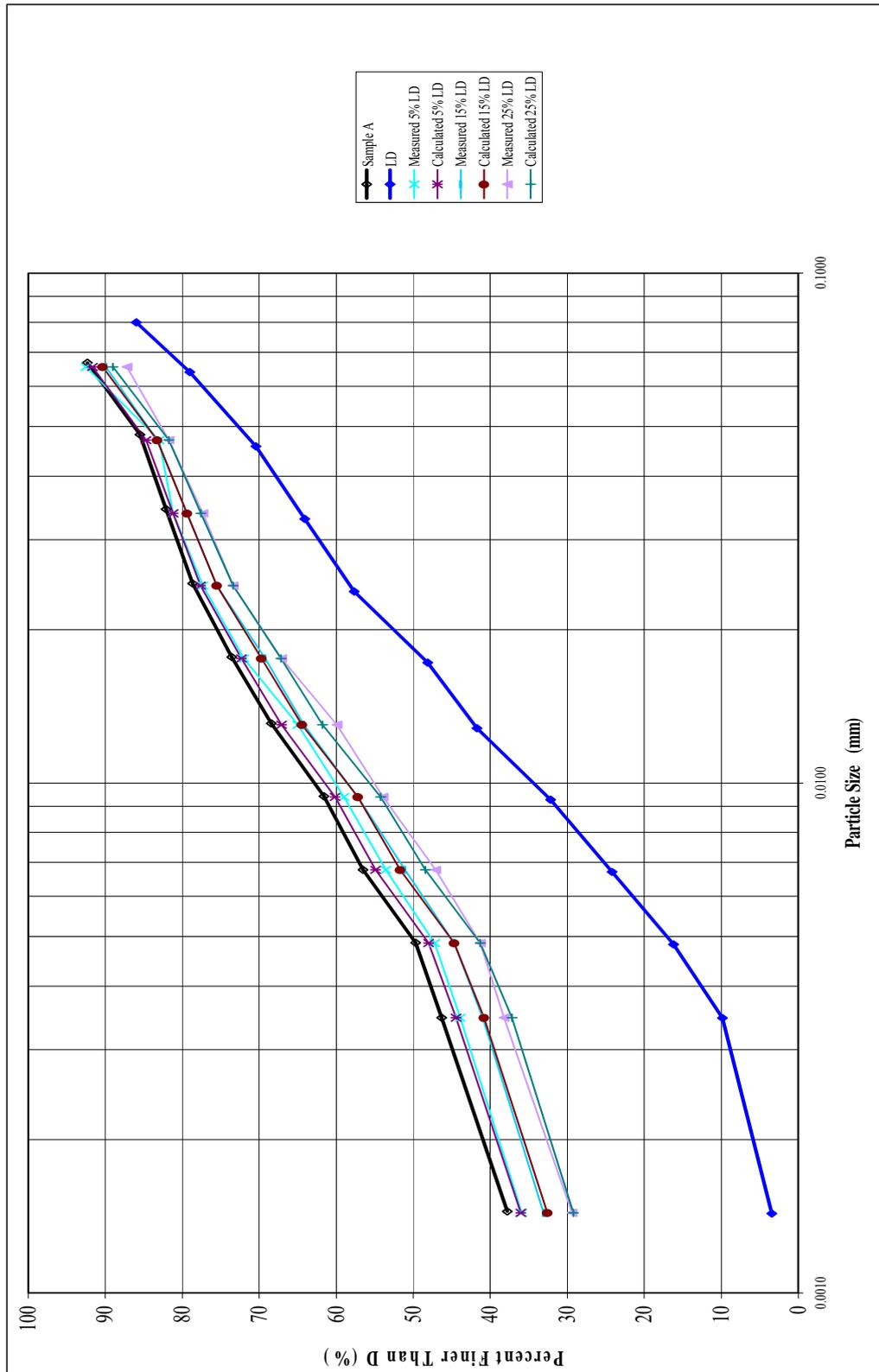


Figure C.1 Grain Size Distribution of Sample A, Limestone Dust, Calculated and Measured Grain Size Distribution Curve of 5% LD, 15% LD and 25% LD

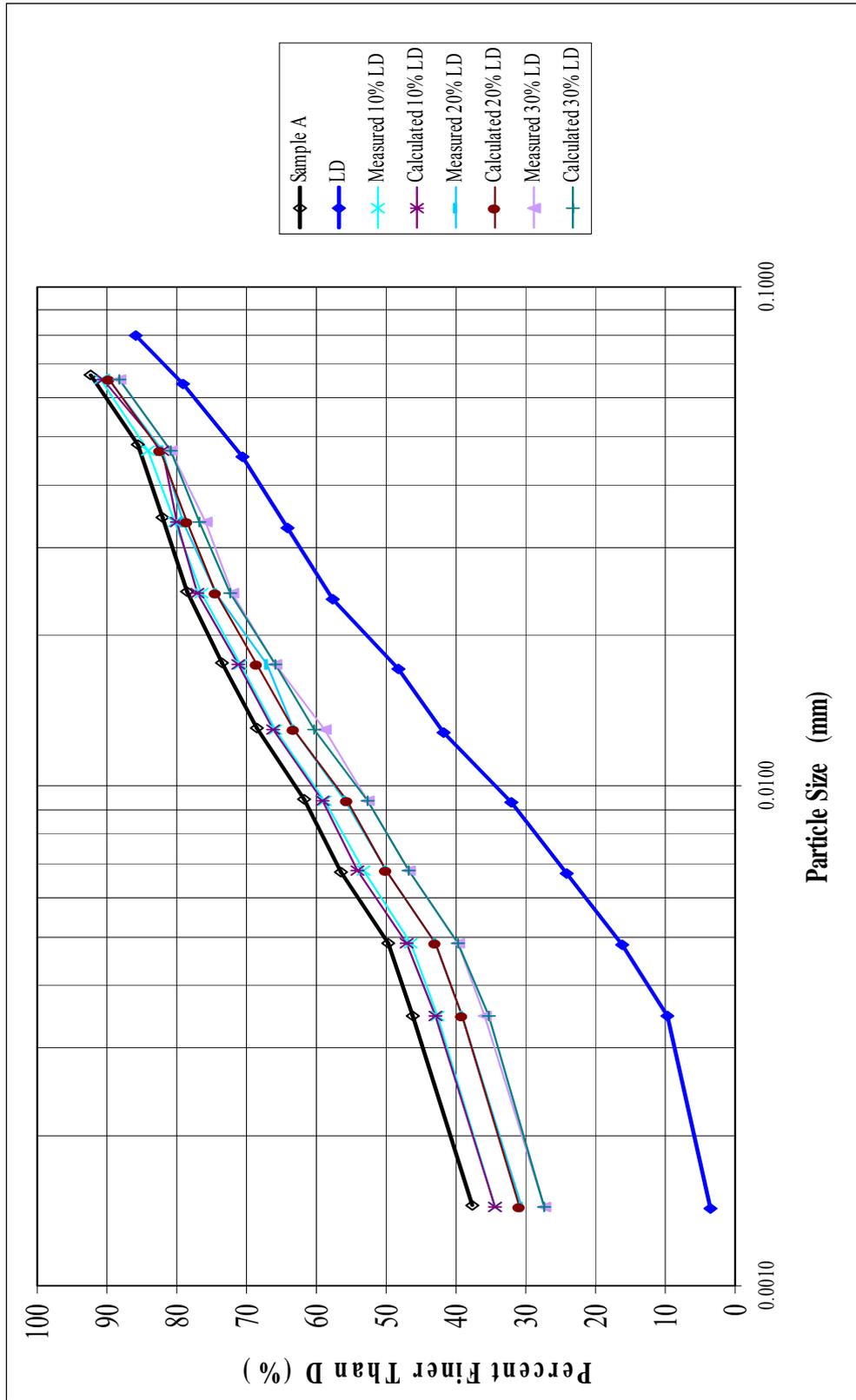


Figure C.2 Grain Size Distribution of Sample A, Limestone Dust, Calculated and Measured Grain Size Distribution Curve of 10 % LD, 20 % LD and 30 % LD