

A DISSERTATION ON
**STABILIZATION OF AN EXPANSIVE SOIL USING MARBLE SLURRY DUST
AND LIME**

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the Award of the Degree of

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IN

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With Specialisation in

GEOTECHNICAL ENGINEERING

SUBMITTED BY

Rajat Verdhan Nagayach

(Roll No. 07/GTE/2k11)

Under The Guidance of

Dr. Ashok Kumar Gupta



Department of Civil

Delhi Technological University, Delhi

2013



DELHI TECHNOLOGICAL UNIVERSITY

CERTIFICATE

This is to certify that the major project report entitled “STABILIZATION OF AN EXPANSIVE SOIL USING MARBLE SLURRY DUST AND LIME” is a bona fide record of work carried out by Rajat Verdhan Nagayach (Roll No. 2K11/GTE/07) under my guidance and supervision, during the session 2013 in partial fulfillment of the requirement for the degree of Master of Technology (Geotechnical Engineering) from Delhi Technological University, Delhi.

The work embodied in this major project has not been submitted for the award of any other degree to the best of our knowledge.

Prof. A.K. Gupta

Department of Civil Engineering

Delhi Technological University

Delhi-110042

2013



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I thank all the people directly or indirectly involved in successful completion of this project.

Rajat Verdhan Nagayach

DECLARATION

I, hereby declare that the work being presented in this Project Report entitled **“Stabilization of an Expansive Soil using Marble Slurry Dust and Lime”** is the bonafide work carried out by me as a part of major project in partial fulfilment of the requirement for the award of the degree of Master of Technology in Geotechnical Engineering.

The matter presented in this report has not been submitted by me for the award of any other degree.

Rajat Verdhan Nagayach

M. Tech (Geotechnical Engineering)

Roll No. 2K11/GTE/07

Department of Civil Engineering

Delhi Technological University

Delhi-110042

ABSTRACT

Expansive soils occurring in vast regions of the world cause serious problems for civil engineering structures. These soils swell when given an access to water and shrink when they dry out. Several attempts have been 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 marble slurry dust, which is a by-product of marble industry, was used for stabilization of expansive soils in addition to lime.

The expansive soil was prepared in laboratory as a mixture of bentonite and local soil. Waste marble slurry dust and lime was added with predetermined percentage, varying from 0 to 30 percent to the expansive soil. The grain size distribution, consistency limits, mineral composition, percentage of swell and rate of swell were determined for the soil 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. It was found that curing of samples reduces the swell percentages and rate of swell.

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CHAPTER 1

INTRODUCTION

1.1 Expansive Soils

The soils which are expansive in nature are a problem worldwide and pose several challenges for civil engineers because of their tendency to heave during wet seasons and shrink in the dry seasons. (Mishra et al. 2008) The expansive soil deposits occur in vast areas throughout India and they do impose problems on engineering structures. These soils are thus considered as soils that are potential, natural hazards which may cause damage to structures extensively if not treated adequately (Al-Rawas, 2002). Sometimes the expansive soils cause damage to structures even more than any other natural hazard like floods or earthquakes structures, and that particularly to light buildings and pavements. (Jones and Holtz, 1973).

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

Expansive soil shrink and swell with varying water content and they lose strength upon ingress of water. This excessive heave associated with swelling of expansive soil can cause considerable distress to light weight engineering structures. Hence several attempts have been made earlier to control this behaviour of swelling-shrinking exhibited by these soils. There are several methods that have been used to minimize or eliminate the harmful effects of expansive/soft clayey soils on structures.

Several investigations have been done to evaluate the soil stabilization process using additives such as lime, fly ash, rice husk limestone dust etc. Numerous works have been done in this area by Rao (1994), Cokca E.(2001), Kumar *et al.* (2003), Kaniraj S.R. and Gayathri, V. (2004), Pandian N.S.(2004), Das, S.K & Yudhbir (2005), Kim *et al.*(2005), Kumar. A, Walia, S.B and Bajaj A. (2007), Ramesh *et al.*(2010) and Kumar *et al.*(2010). The stabilization of expansive soils with admixtures helps controlling the potential of soils for change in volume, and thus improves the strength of soils.



1.2 Production of Waste Marble Dust

Stone-masons often apply the term marble to any rock which can be easily polished. Limestones, schistes, travertines or even granites can be considered as marble in the business world. Waste marble dust produced from marble plants can be of either of these natural stones dust. This marble dust is an industrial by product and is a waste which if utilised positively is not only helpful in reducing environmental pollution but also caters to the problem of disposal of it. Marble or real marble is a metamorphic rock that consists predominantly of calcite and/or dolomite (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 marble dust was used which was dominantly calcite in nature.

The marble (natural stone) production amount was 11 million tons in India during the year of 2011 (Suresh et al). 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 (Suresh et al).

One of the major problems for the marble industry is the production of fine particles of size (<2 mm) while cutting of the marble. While a block of 1 m³ marble is cut into 2 cm thick slabs, the fine particle produced in the process is approximately 25 % in proportion (Suresh et al). Water is used as coolant while cutting of marble blocks, but the fine particles get easily dispersed after losing humidity, under atmospheric conditions, under the action of natural agents like wind and rain. Thus, inevitably fine particles can cause a lot more pollution than other forms of marble waste.

The waste thus produced in the marble industry needs to be disposed and dumped of in a way, which least effects the environment. Hence the large pieces of marble waste can be used as embankment or pavement material, while waste marble dust can be used as additives in some industries (paper, cement, ceramic etc.). But, only a small portion of the waste marble products is actually utilized economically and efficiently. Most of them are dumped on open lands. Increase of usage fields of waste marble products will eliminate the potentially harmful effects of them on environment and minimize the cost being incurred due to storage.





Image1.1 View from waste marble dust disposal sites in Bikaner(TOI,19-Nov-2012)

1.3 Lime

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 pre existing lime sediments (Dietrich and Skinner, 1979). Due to its color, durability and application possibilities, limestone has wide application field. The limestone is calcined, cooked and processed so as to produce quicklime. In this study, the lime used consists of mainly calcite.

Lime acts as an additive and its dominant construction related use is in soil stabilization for roads, building foundations, and earthen dams because of its chemical reactions and pozzolanic actions with the soil constituents. Lime is added to low quality soils in appropriate amounts so as to to exploit its properties in favour of engineering applications.

1.4 Objective

The set of challenges offered by the expansive soils as outlined above are a major concern for Geotechnical engineers world over. With the ever increasing demand of houses, dams and other infrastructure related construction, the issue of environmental degradation has also become a major concern. In such a scenario there is an urgent need to develop methods of construction in difficult soils that have minimum impact on the environment.

Hence the present work is carried out to study the suitability of waste marble slurry dust and lime as additives to stabilize an expansive soil for its swelling potential. The soil sampla was prepared in the lab using 75% local soil and 25% bentonite . Both lime and marble slurry dust can be used as stabilizers for expansive soils. But, the effectiveness of stabilizers depends on



their chemical content. By addition of stabilizers, the swelling percentage decreased considerably. The reduction was higher for the samples added with lime.

1.5 Outline of the thesis

The literature review and the previous work done, related to stabilization of expansive soil has been covered. Various researchers have tried to use different types of stabilizing materials like fly ash, limestone dust cement, rice husk etc. But not much work has been done using marble slurry dust and hence this area still remains unexplored.

The review also covers various topics which are directly related to the processes, factors and mechanisms of stabilization of swelling soils.

In Chapter 3 the experimental investigations which were accomplished during the course of the thesis have been explained

The Chapter 4 is a brief on results and analysis of the various experiments conducted.

Lastly the conclusions derived from this experiment oriented thesis and the further scope of work has been discussed in Chapters 5 and 6



CHAPTER 2

LITERATURE REVIEW

2.1 Properties of expansive soils

The expansive soils, which generally consists of the clay mineral montmorillonite, include shales, claystones and sedimentary and residual soils,. They exist in a moisture-deficient, unsaturated condition when in arid and semiarid climatic regions. The soil profile is subjected to seasonal ,environmental variations near the ground surface in maximum intensity. Hence the expansive nature of the soil is most observed near ground surface itself. (Terzaghi, Peck and Mesri,1996; Fredlund, 1993).

There are many correlations that are helpful to identify the soils which are potentially expansive. It may also be quiet possible to identify them with visual inspection. Visual indications include (Wayne et al,1984):

- 1) Deep and wide cracks called shrinkage cracks observed in the dry periods
- 2) Soil is very sticky and soft when wet while rock-hard when dry
- 3) The surrounding structures experience damages due to the expansion and contraction of soil.

2.2 Clay Mineralogy

The term clay is referred in context of both to a class of minerals and to a size of particle. When it is in context of the size term, it refers to all the constituents of the soil which are smaller than a particular size, usually 0.002 mm (2μ) in the terms of engineering classifications. On the other hand in terms of a mineral , it refers to a specific type of clay mineral which are peculiarly having (1) a net electrical charge, (2) high weathering resistance, (3) plasticity when mixed with water and 4) small particle size.

The basic and idealized crystalline structural unit of a clay mineral is composed of a silica block which is a tetrahedron block and an aluminium block which is an



octahedron block. Aluminium octahedron block may consist either of aluminium (Al^{3+}) or magnesium (Mg^{2+}). If only aluminium is present, then it is termed gibbsite and if only magnesium is present, then it is called brucite. Various different clay minerals are formed when these sheets stack on top of each other with different arrangements and ions bonding them together (Oweis and Khera, 1998). The representation of a silica tetrahedron along with a silica sheet, and an octahedron along with octahedron sheet are presented in Figure 2.1 and Figure 2.2, respectively. These figures also consist of schematic representations of silica and octahedron sheets.

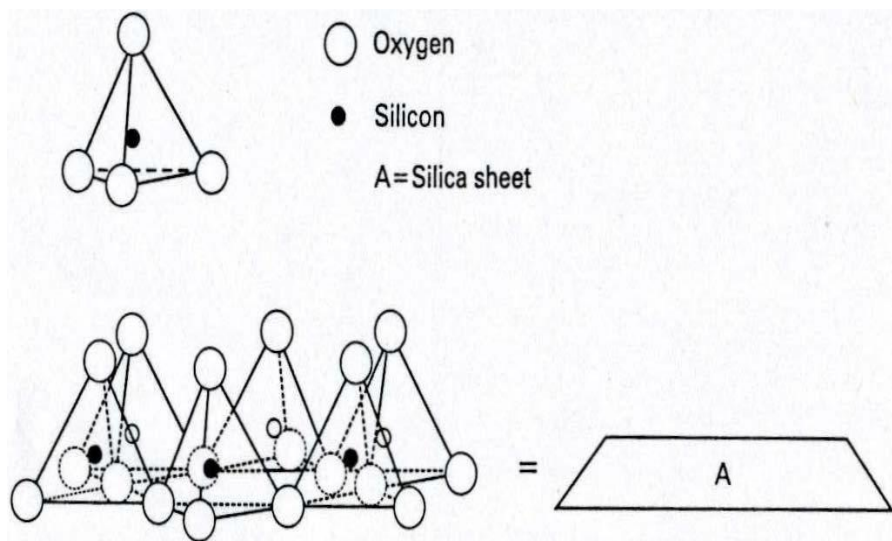


Figure 2.1 A silica tetrahedron and a silica sheet (Grim,1962)

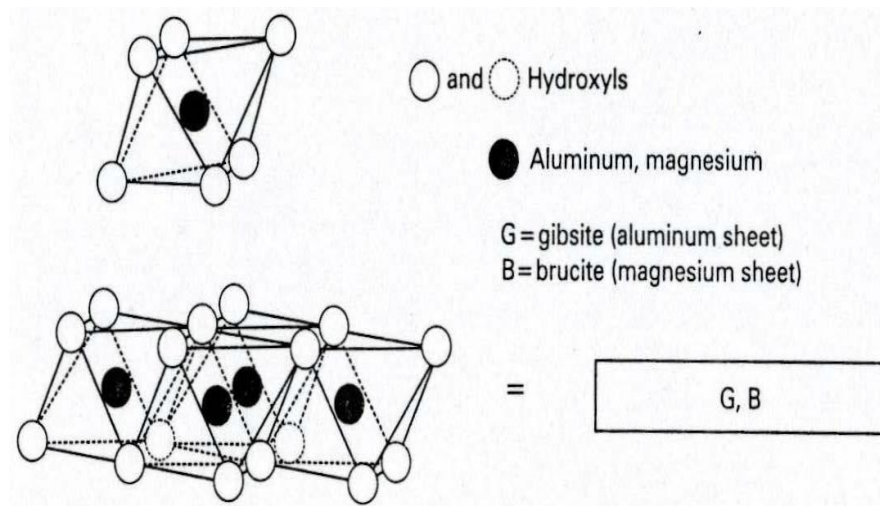


Figure 2.2 An alumina octaheron and an octaheron sheet (Grim,1962)

The three most important structural groups of clay minerals which are described for engineering purposes given:

Kaolinite group – are generally nonexpansive

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

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

2.2.1 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. (Craig, 1997).

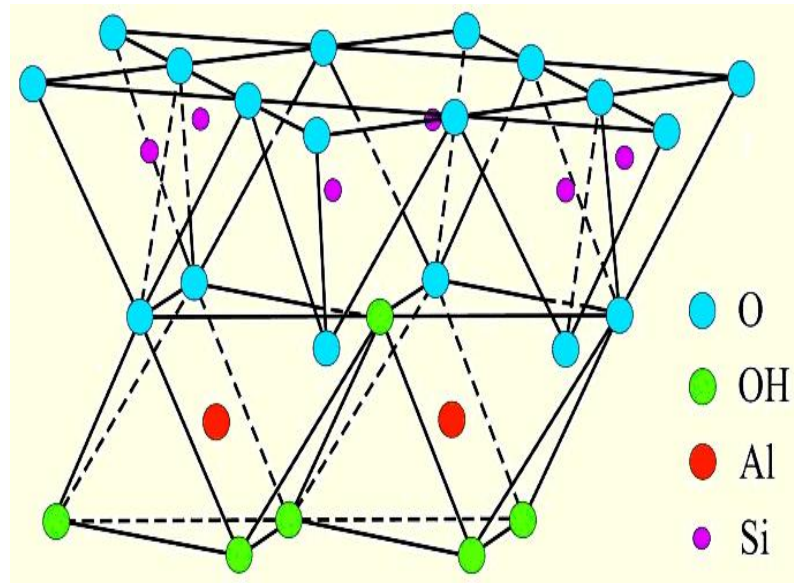


Figure 2.3 Diagrammatic sketch of the kaolinite clay mineral (USGS 2001, Grim 1962)

2.2.2 Illite group (Mica-like group)

Illite

Illite consists of a basic mineralogical structure having a sheet of alumina octahedron between sandwiched between two sheets of silica tetrahedrons. There is a partial substitution of aluminum by magnesium and iron in the octahedral sheet, while there is partial substitution of silicon by aluminium in the tetrahedral sheet. The combined layer of sheets are together linked by fairly weak bonds because of the non-exchangeable ions of potassium held between the two structural units (Craig, 1997).

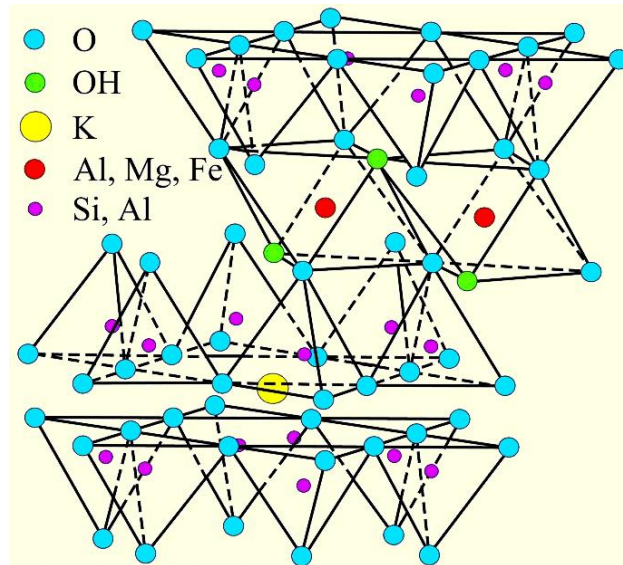


Figure 2.4 Diagrammatic sketch of the illite clay mineral (USGS, 2001,Grim, 1962)

2.2.3 Montmorillonite group (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 clay mineral are the same as that of illite clay mineral except there is no potassium ion present. The space between the combined sheets is occupied by water molecules and exchangeable cations. And hence in between these combined sheets there is a very weak bond due to these ions. Due to additional water being absorbed between the combined sheets considerable swelling of montmorillonite minerals can occur (Craig, 1997; Oweis and Khera, 1998).

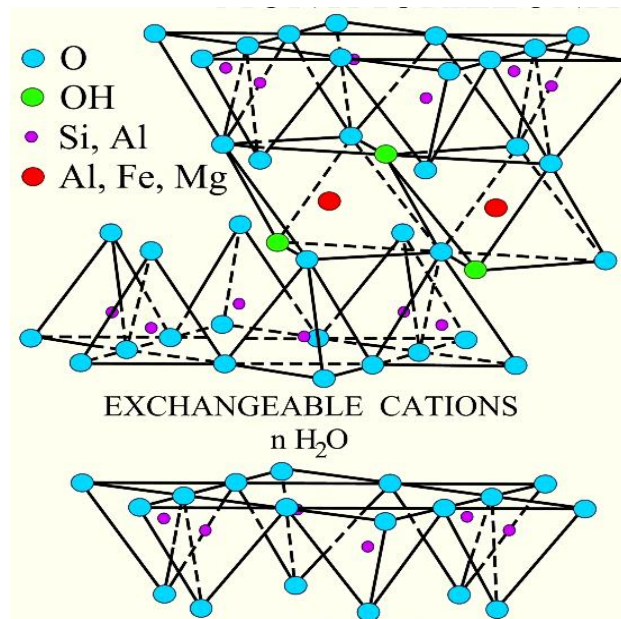


Figure 2.5 Diagrammatic sketch of the montmorillonite clay mineral(USGS, 2001 Grim, 1962)

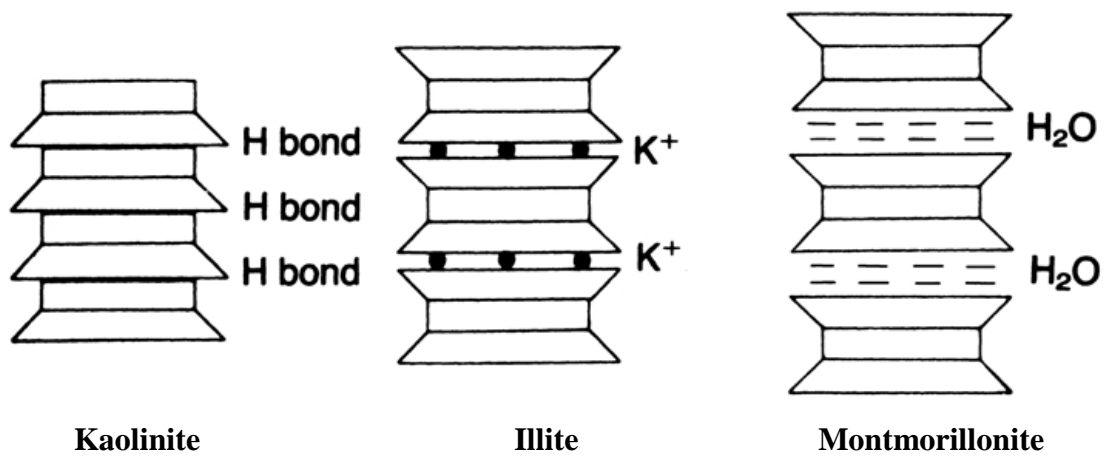


Figure 2.6 Schematic representations of clay minerals (Craig, 1997)

2.3 Mechanism of Swelling

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

2.3.1 Double Layer of Clay Minerals

The clay particle surface are negatively charged and the concentration of positive ions adjacent to the clay particle in solution forms what is referred to as the diffuse double layer or DDL (Bohn et al. 1985).The overlapping DDLs between different clay particles generates microscale “swelling pressures” which are small interparticle repulsive forces.

With the increase in the thickness of the DDL, interparticle interaction also increases among the DDLs and, thereby increasing the swelling potential (Mitchell, 1976). The DDL’s thickness majorly depends upon the concentration of cations in its vicinity, valence of cations, pH and temperature.

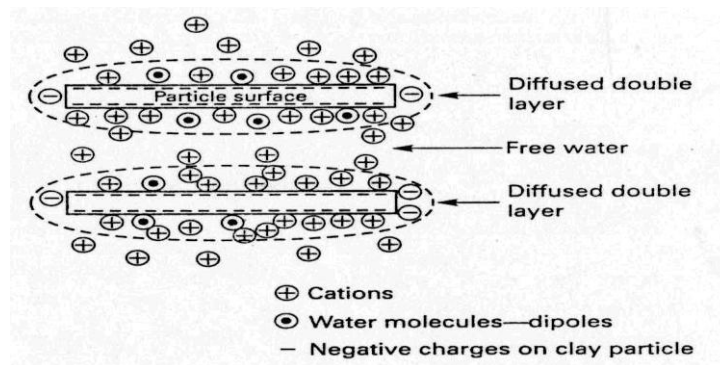


Figure 2.7 Double layer of clay minerals (Oweis and Khera, 1998).

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

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



c) Effect of temperature: Increasing the temperature causes the DDL thickness to increase, thus the change in temperature has effect on strength, compressibility and swelling of soils (Mitchell, 2005).

d) Effect of pH: Ions of hydroxyls (OH)⁻ are exposed on the edges of clay particles and on surfaces of it. The pH levels of the solution strongly influence the tendency for hydroxyl ions to dissociate in the water as higher the pH, the greater will be the tendency for H to go into the solution, and hence greater will be the effective negative charge of the particle. Alumina, that is exposed at the edges of clay particles, is by nature amphoteric i.e. (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, 2005).

2.3.2 Cation Exchange Capacity (CEC)

The net negative charge on the surface of soil particles in water is neutralised by the cations existing in the solution. These cations are readily exchangeable with other cations . The relative concentrations of cations in the water and the electrovalence of cations control the exchange reactions (Terzaghi et al, 1996). The quantity of exchangeable cations required to balance the net negative charge on the surface of the clay particles is the cation exchange capacity. CEC is expressed in milliequivalents per 100 grams of dry clay (Nelson et al 1992).

Table 2.1 CEC of principle clay minerals (Terzaghi, Peck and Mesri, 1996)

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



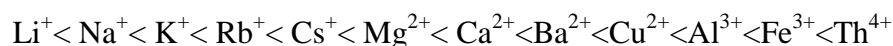
2.4 Chemical stabilization of soil

One of the most widespread and oldest method of ground improvement is to do soil stabilisation with the use of chemical admixtures. Chemical stabilization process is the mixing of soil with one of or a combination of admixtures in the form of powder, slurry, or liquid to achieve the general objectives of improving or controlling its volume stability, permeability, and durability ,stress-strain behaviour and strength characteristics. (Winterkorn and Pamukçu, 1990).The method of chemical stabilisation for soil improvement can be grouped into three chemical processes viz; cation exchange, flocculation - agglomeration, pozzolanic reactions.

2.4.1 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, 1991).

The clay particles, are negatively charged and thus adsorb cations of specific type and amount. The exchange of cations or ease of replacement depends on several factors, primarily the valence of the cation. Lower valence cations are easily replaced by higher valence cations. In case the ions are of the same valence, then the size of the hydrated ion becomes important; i.e. larger the size of the ion, the greater would be the replacement power. Keeping other conditions as constant, divalent cations are held more tightly than monovalent cations and trivalent cations are held more tightly than divalent. (Mitchell and Soga, 2005). A typical replaceability series is shown below



2.4.2 Flocculation and Agglomeration

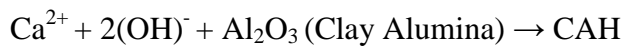
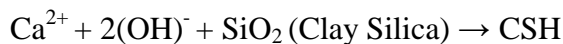
Cation exchange reactions result in the flocculation and agglomeration of the soil particles with consequent reduction in the amount of clay-size materials and hence the soil surface area, which inevitably accounts for the reduction in plasticity



(Terzaghi and Peck, 1967). A significant reduction in the swelling of the soil occurs due to change in texture.

2.4.3 Pozzolanic Reactions

The time dependent pozzolanic reactions do play a major role for stabilizing the soft soils as they act to improve 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).

2.5 Soil Stabilization using lime and marble dust

The chemical theory involved in the lime reaction is complex but the main reactions include flocculation, cation exchange, and pozzolanic reactions (cited in Nelson and Miller, 1992). The cation exchange and flocculation concepts, primarily are effects of stabilizer, are stated in Section 3.1.1 and 3.1.2, respectively. Also, in Section 1.2.3, the pozzolanic reactions for lime stabilized soils are presented. These three stabilization steps are valid for stabilization of expansive soils using waste marble dust and lime.

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



Many of the researchers (Oates, 1998; Almedia et al.2007) have reported that marble consists of very high lime (CaO) content up to 55 % by weight. Thus, stabilization characteristics of waste marble dust and lime is mainly due to their high lime (CaO) content.

2.6 Factors Influencing Swelling

The swelling potential of clayey soils may be affected by various factors like 1) the soil properties influencing the nature of the internal force field, 2)the environmental factors those may change the internal force system, 3) 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 density, initial water content, type and amount of compaction also influence the swell parameters and swell potential of soils. From the above mentioned factors, the clay content and the mineralogical composition are by far the most important factors influencing the extent of swell potential in clayey soils.It is a well known fact from earkier investigations that; the amount of swell or swell potential for a clayey soil varies with the type and amount of clay mineral present in the soil mass (Whitman and Lambe, 1969).

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

FACTOR	DESCRIPTION
Clay Mineralogy	Montmorillonites, some mixed layer minerals and vermiculates, are the typical clay minerals which are responsible to cause soil volume changes . Kaolinites and Illites are frequently inexpansive, but can seldom cause volumetric changes when the sizes of the particles are extremely fine.
Soil Water Chemistry	Swelling can be decreased by increase in the cation concentration and increase in the 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, those soils have greater potential for swelling and shrinkage, that exhibit plastic behaviour over wide ranges of moisture content and that have high liquid limits. Plasticity is an indicator of swell potential
Soil Structure and Fabric	Dispersed clays tend to be less expansive than flocculated clays. Cemented particles tend to reduce the swelling of the soils. Structure and fabric can be altered by compaction at high water content or by 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 indicate closer spacing between the particles, which may mean greater repulsive forces between particles and hence 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 be having higher affinity towards water, i.e. higher suction than the same soil which is having higher water content i.e. lower suction. Conversely, a wet soil profile will loose its moisture more readily on being exposed to drying influences, and will shrink more when compared to the same soil of a relatively dry initial profile. Hence while accounting for the expected range of final suction conditions, the initial soil suction must also be considered.
Moisture Variations	Heave is defined by the changes in moisture content near the upper part of the soil profile i.e.in the active zone. The widest variation in moisture and volume change will occur in those layers.
Climate	The moisture availability in the soil and the depth of fluctuations of moisture with season depends upon the variation and



	amount of precipitation and evapo transpiration in the region. The greatest seasonal heave are occurred in semiarid climates which have small spells of wet periods.
Groundwater	Fluctuating water tables contribute to moisture variations where water table is shallow and provide the source of moisture.
Drainage	Drainage features at the surface, like pond around a poorly graded foundation of a house, provides a source of water at the near surface. .Also the soil gets access to moisture at greater depths because of leaky plumbings.
Vegetation	Grasses, shrubs, and trees,through the process of transpiration deplete moisture from the soil, and thus make the soil to be wetted differentially in areas of varying vegetation
Permeability	Soils particularly having cracks and fissures in situ soil mass, have higher permeabilities, and thus allow quicker migration of water and promote quicker rates of swell.
Temperature	The moisture diffuses to cooler areas beneath pavements and buildings with the increasing temperatures.

Table 2.4 Stress Conditions Affecting Swell Potential (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
Stress History	A normally consolidated soil is less expansive than the same soil at the same void ratio, but overconsolidated soil. With the aging of compacted clays the swelling pressures can increase, but under light loading amount of swell has shown to be unaffected by aging. In laboratory samples repeated drying and wetting tend to reduce the swell, and after a certain number of wetting-drying cycles, the swell becomes unaffected.
In situ Conditions	It is of utmost importance to estimate the initial stress state in a soil ,so as to evaluate the probable affects of loading the soil mass and/or varying the moisture in the environment. One can determine the initial effective stresses, by making in-situ observations and



	measurements or through sampling and testing in the laboratory.
Loading	For a given density and moisture content, the change in volume of the soil is determined by the magnitude of surcharge loading. The interparticle repulsive forces can be balanced by an externally applied load and hence reduces swell.
Soil Profile	The swelling movements are considerably influenced by the location and thickness of potentially expansive layers in the soil profile. The greatest movements shall occur in those soil profiles which have expansive soils extending right from the surface till the depths below the active zone. Comparatively less movements shall occur where non expansive material overlies above expansive clay or is having overlying bedrock at a shallow depth.

2.7 Review of some previous works

Mishra et al (2008) examined the impact of the variation in compaction conditions on the swelling and shrinkage behavior of three types of soils. Two soils were natural soils, i.e black cotton soil and red soil and one was artificially mixed soil sample of commercially available bentonite with well-graded sand,. Four types of compaction conditions were selected and the compaction curves for the standard proctor conditions were drawn. The results of the experiments indicated that the shrinkage and swelling behavior of the tested soils is is influenced more by clay mineralogy rather than compaction conditions. Analysing the void ratio -moisture content relations during shrinkage indicated that the soil specimens generally shrunk in 3 linear stages, each distinct in nature. During the first shrinkage stage a small reduction in void ratio occurred on reduction in water content and this was termed as the initial shrinkage. The void ratio decreased rapidly with reduction in water content in the second stage and was called as primary shrinkage. In the third stage, reduction in water content was along with a marginal variation in void ratio and is called as residual shrinkage. Irrespective of the studied initial conditions of compaction, for all the specimens, the transition to the residual shrinkage from primary occurred within a range of water content (10–15%).



Tripathy, Rao, Freudland (2002) studied the response of compacted expansive soils under shrink-swell cycles. Cyclic shrink-swell tests were conducted in the laboratory on compacted samples of two different expansive soils at 6.25, 50.00, and 100.00 kPa of surcharge pressures. The water content and void ratio of the samples at different intermediate stages during shrinkage till the shrinkage completed and during swelling until it completed were determined to plot the moisture content versus void ratio paths with an increasing number of shrink-swell cycles. The results of the experiments indicated that the swell– shrink path were reversible once the soil reached to an equilibrium stage where the vertical displacements while swelling and shrinking were the same. This generally occurred after around 4 swell–shrink cycles. The swelling and shrinkage path showed an S-shaped curve (two curvilinear portions and a linear portion) for each sample when subjected to complete swelling – complete shrinkage cycles. But, the swell and shrink path occurred as a portion of the S-shaped curve, when the sample was subjected to full swell and partial shrinkage cycles. More than 50% of the total vertical displacements and more than 80% of the total change of volume occurred in the linear central portion of the S-shaped curve. The primary value of the swelling–shrinkage path was in providing information regarding the change in void ratio that will occur for a given change in moisture content for any possible swelling–shrinkage pattern. Further it was suggested with only a limited number of tests in the laboratory we can establish these swell–shrink paths.

Nelson, Chao, Overton (2007) defined a variable termed as the “Expansion Potential”, EP, in this paper to express the actual expansion potential of a soil as the risk of foundation or slab movement relates to the amount of heave that shall occur, The heave is more closely related with the product of the percentage swelling times the swelling pressure, and not just the percentage swelling. The expansion potential of a swelling soil is commonly expressed to be the amount of swelling which occurs while inundating the soil in the oedometer-swelling tests and it is expressed as very high, high, medium, or low based solely on the percentage of swell. He computed the values of expansion potential using a database of more than 1,100 collected results of consolidation-swell tests. It was shown that a very good correlation is observed between expansion potential and predicted heave. It is concluded that the use of



expansion potential (EP) will provide for a more accurate description for the potential of foundation or slab movements and their actual risk. No correlation is found between the EP and other parameters such as dry density ,plasticity or natural moisture content.

Savoohan (1970) on examining the geotechnical behaviour of expansive soils suggested that with time the soil surface rises during the swelling. The fluctuations in temperature gradient in both bottom and top layers govern the rate of this rise of soil surface. This expansion activity also occurs in the upper restricted zone of the soil irrespective of higher swelling potential. There will not be any further volume change if the water content of the clay remains unchanged. Even a slight variation in water content can prove enough to cause detrimental amount of swelling. A clay sample having low moisture content has a swelling capacity very high and hence it has a higher swelling pressure than the clayey soils which have higher water content. A soil layer which is close to the ground surface, will swell more with increase in intake of water, while on the other hand the same soil will be unable to swell if it is at a depth below the surface under an overburden load which tries to neutralize the effect of the swelling pressure by the dry soil.

Cokca (2001) studied the effect of flyash on expansive soils. The flyash often consists of hollow spheres of aluminium, silicon and iron oxides and carbon in unoxidized states. The flyash are of two major classes class F and class C. The class C is produced from burning of bituminous or anthracite coal and the class F is produced from burning of sub bituminous coal and lignite. Both the classes of fly ash are puzzolans, which are defined as aluminous and siliceous materials. Thus fly ash are able to provide an array of trivalent and divalent cations (Ca^{2+} , Al^{3+} , Fe^{3+} etc) under ionized conditions that are able promote the flocculation of dispersed clay particles. Thus the swelling soils can be potentially stabilized efficiently by cation exchange process using flyash or other similar materials.

He carried out investigations using tuncbilek flyash and soma flyash and added it to expansive soil at 0-25%. Soil samples mixed with flyash were put to



curing for 7 days and 28 days after which they were subjected to consolidation swell tests. His experimental findings confirmed that the activity, plasticity index and swelling potential of the soil specimens decreased with the increase in percentage of stabilizer and its curing time. The most optimum content of flyash responsible to decrease the swelling potential was observed to be 20%. The changes in the swelling potential and physical properties is upto some extent the result of additional silt sized particles and due to the chemical reactions that cause flocculation of clay particles immediately. The time dependent pozzolanic and self hardening properties of flyash also can be one cause of it. He thus concluded that both low calcium and high calcium class C fly ashes can act to become effective stabilizing agents for improvement of the expansive soils.

Phanikumar and Sharma (2004) carried out a study on the effect of fly ash on engineering properties of swelling soil through an experimental programme. The effect on parameters like swelling pressure free swell index (FSI), plasticity, swell potential, compaction, hydraulic conductivity strength of swelling soil was studied. The ash was mixed with expansive soil with flyash contents of 0, 5, 10, 15 and 20% on a dry weight basis and they concluded that increase in flyash content reduces plasticity characteristics and the FSI was reduced by about 50% by the addition of 20% fly ash. The hydraulic permeability of expansive soils mixed with flyash decreases with an increasing amount of flyash content, due to the increasing maximum dry unit weight with the increase in flyash content. When the flyash content is increased there is a decrease in the optimum moisture content and the maximum dry unit weight is increased. The effect of fly ash is akin to the increased compactive effort. Hence the swelling soil is rendered more stable.

Das (2005) reported the findings of experimental studies with regard to some common engineering properties e.g., specific gravity, grain size, unconfined compression strength and compaction characteristics, of both low and high calcium fly ashes, to evaluate their suitability as materials for reclamation fills and embankment materials. In addition, he also studied the chemistry, morphology and mineralogy of fly ashes using electron dispersive x-ray analyzer, scanning electron



microscope, infrared absorption spectroscopy and x-ray diffractometer. In high calcium fly ash, chemical and mineralogical differences were observed for particles, of size >75 micron and the particles of size <45 micron. He inferred that the mode and duration of curing significantly affects the strength and stress–strain behavior of fly ashes. The geotechnical properties of fly ash are governed by factors like iron content (Fe_2O_3) content, lime content (CaO), and loss on ignition (LOI). The distinct difference between pozzolanic reactivity and self-hardening and has been emphasized.

2.8 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 - 08 (Standard Test Methods for One- Dimensional Swell or Collapse 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 of soil column following absorption of water.

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

Primary swell, L = A short-term swell usually characterized as being completed at the intersection of the tangent of reverse curvature to the curve of a dimensional change v/s logarithm of time plot with the tangent to the straight line portion representing long-term or secondary swell.

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

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

Remoulded or compacted specimen= A test specimen compacted into a mould.



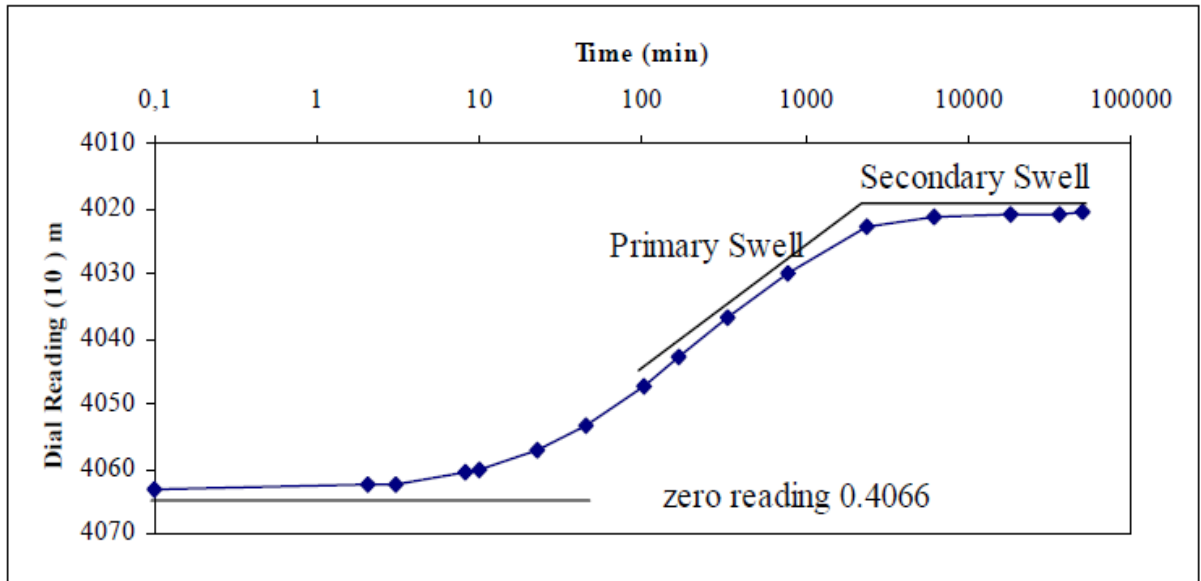


Figure 2.8 Time - Swell Curve (ASTM, 2008)

The following three alternative test methods require that soil specimens be restrained laterally and loaded vertically in a consolidometer, with access to free water.

2.8.1 Method A

This method is used to measure one-dimensional wetting-induced swelling or collapse (hydrocompression) strains for compacted or natural soils over a range of vertical stresses. Record the initial deformation readings, and apply the seating pressure on specimen. The specimen is inundated from top and bottom supply of water to swell vertically. While the specimen swells, its 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 subsequently at 1, 2, 4, 8, 24, 48, and 72 hours (Fig. 2.9). The resulting swell or collapse deformations are measured. The final water contents and dry densities are also measured. This method can be referred to as *wetting-after-loading tests on multiple specimens*. The data from these tests can be used to estimate one-dimensional ground surface heave or settlement.

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 removing the σ_1 , the sample is inundated, and the test continues as in stated above. Method A measures (a) the free swell, (b) percent heave for vertical confining pressures up to the swell pressure, and (c) the swell pressure.

2.8.2 Method B

This method can be used for measuring one-dimensional wetting-induced swell or collapse strain of a single “intact” specimen of natural soil, or a single “intact” specimen of compacted soil obtained from an existing fill or embankment. The specimen is loaded to a specific vertical stress, typically the in-situ vertical overburden stress or a particular design pressure, or 1 kPa (20 lbf/ft²) for measuring the free swell strain, and then inundated to measure the wetting induced strain under that particular stress. This method can be referred to as *single point wetting-after-loading test on a single specimen*.

2.8.3 Method C

This method is for measuring load induced strains after wetting-induced swell or collapse deformation has occurred. This method can be referred to as *loading-after-wetting test*. The results would apply to situations where new fill and/or additional structural loads are applied to the ground that has previously gone through wetting-induced heave or settlement. The first part of the test is the same as in Method A or B. After completion of the swell or collapse phase, increments of additional vertical loads are applied to the specimen in the same manner as in a consolidation test, Test Methods D2435, and the load-induced deformations are measured.



CHAPTER 3

EXPERIMENTAL INVESTIGATIONS

3.1 Purpose

The objective of this experimental study is to investigate the effects of the addition of waste marble slurry dust and lime 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 marble slurry dust and lime.

3.2. Material

In this study, locally available soil , commercially available bentonite, waste marble dust and lime were used.

Local soil: Soil was procured from local sources. It was sieved through # 40 sieve(425 μ) before usage.

Bentonite: Bentonite (Montmorillonite) was commercially available in the form of fine sized grains.

Waste marble slurry dust: Was procured from naraina industrial area.

Lime: Waste dolomitic marble dust was obtained from commercial sources.

3.3 Preparation of the test samples

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

The expansive soil sample was composed of 75% local soil passed through 425 μ sieve and 25% bentonite which did not require to be sieved. This mix of prepared expansive soil is termed as Sample A for brevity. Each sample was prepared by addition of lime or waste marble slurry dust to Sample A with different percentages to obtain a sample with pre determined percentage of stabilizer varying from 0 to 30 percent (by dry weight of the sample).



Firstly, the local soil, lime, waste marble slurry dust and bentonite, were oven-dried at 50° C for 24 hours . For each sample, the predetermined amount of material was mixed thoroughly with trowel and again passed through 425 μ sieve.

Table 3.1: Designation of soil mixes used in the study

Sample	Expansive Soil (%)	Lime (%)	Waste marble slurry dust (%)	Terminology assigned
Sample A	100	-		A
5% Lime	95	5		5L-95A
10% Lime	90	10		10L-90A
15% Lime	85	15		15L-85A
20% Lime	80	20		20L-80A
25% Lime	75	25		25L-75A
30% Lime	70	30		30L-70A
5% MSD	95	-	5	5M-95A
10% MSD	90	-	10	10M-90A
15% MSD	85	-	25	15M-85A
20% MSD	80	-	20	20M-80A
25% MSD	75	-	25	25M-75A
30% MSD	70	-	30	30M-70A

3.4 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.



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 lime were plotted. However, hydrometer analysis of pure waste marble slurry 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 marble dust added samples. The sample properties, swell potential and soil classification according to USCS are tabulated..

3.5 Experimental Program

Experimental study was conducted as under:

- 1) SEM and XRD analysis for stabilizers were performed at Physics lab DTU .
- 2) Hydrometer tests, Atterberg limit tests, and specific gravity tests were applied to the samples.
- 3) Free Swell Ratio Test was applied to the samples.
- 4) Free swell tests were applied to the samples without cure as per ASTM D4546.
- 5) Free swell tests were applied to the 7 days cured samples as per ASTM D4546.
- 6) Free swell tests were applied to the 28 days cured samples as per ASTM D4546

3.6 SEM Analysis

Scanning electron microscope (SEM) is a type of electron microscope that produces highly magnified images of a sample by scanning it with a focused beam of electrons. The electrons interact with atoms in the sample, producing various signals that can be detected and that contain information about the sample's surface topography and composition.



Image:3.1 SEM machine at DTU



3.7 XRD Analysis

X-ray diffraction technique is non-destructive analytical technique which reveals information about the crystal structure, chemical composition, and physical properties of materials and thin films. This technique is based on observing the scattered intensity of an X-ray beam hitting a sample as a function of incident and scattered angle, polarization, and wavelength or energy.



Image:3.2 XRD machine at DTU



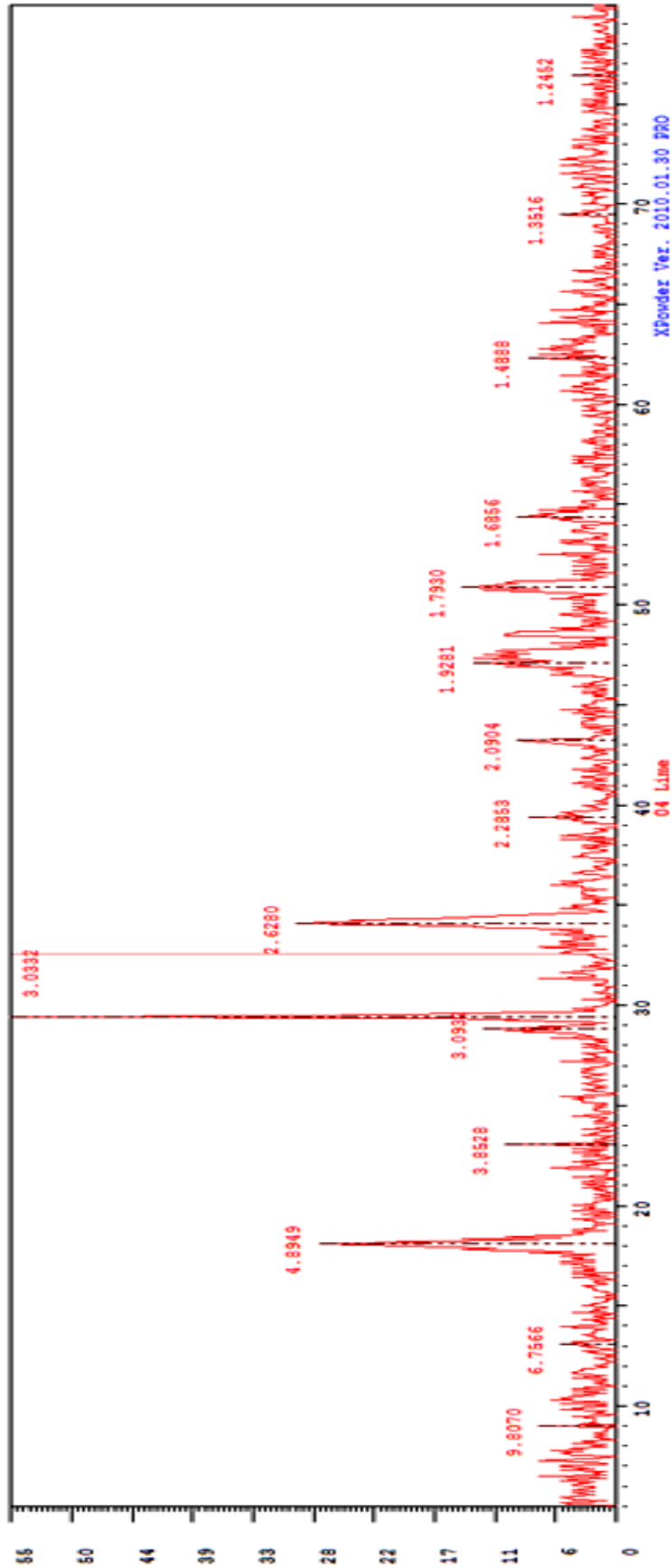


Figure 3.1: X-ray diffraction analysis plot of lime(DTU)(Gupta et al 2013)

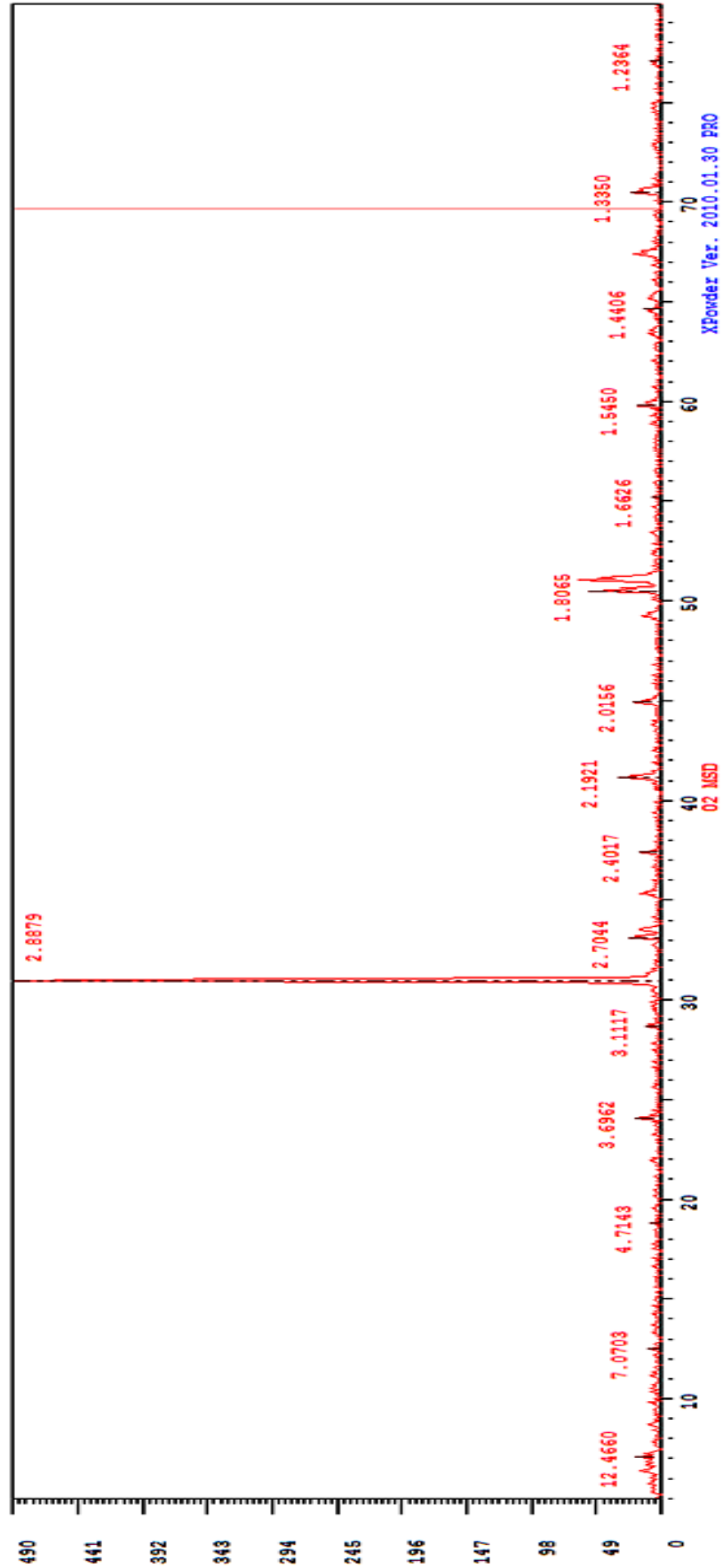


Figure 3.2: X-ray diffraction analysis of MSD(DTU)(Gupta et al 2013)

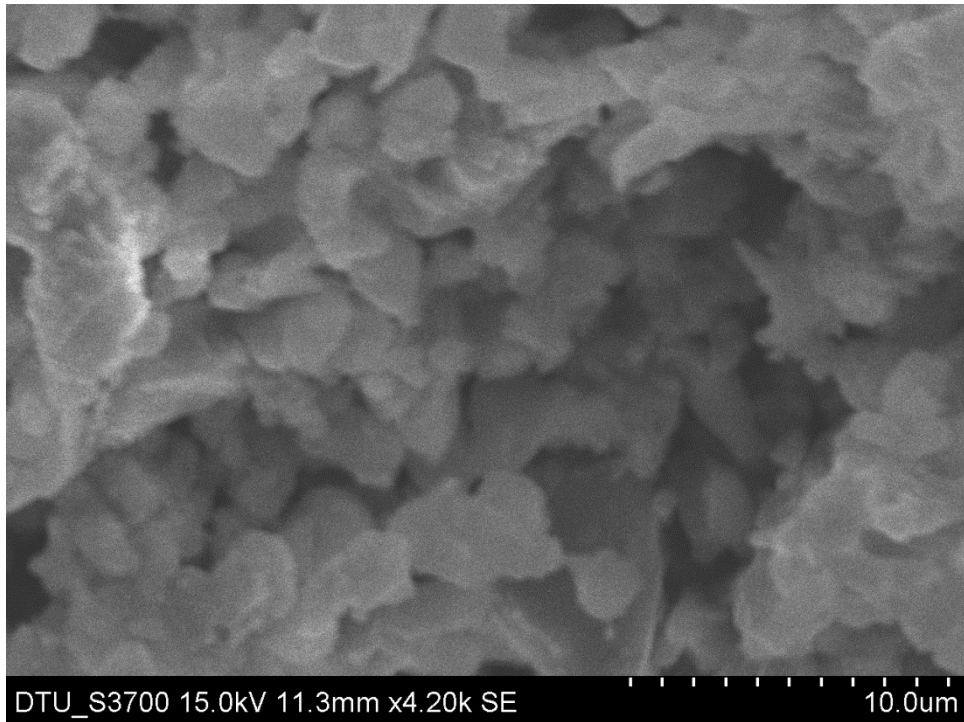


Figure 3.3: SEM image of lime stabilizer (DTU)

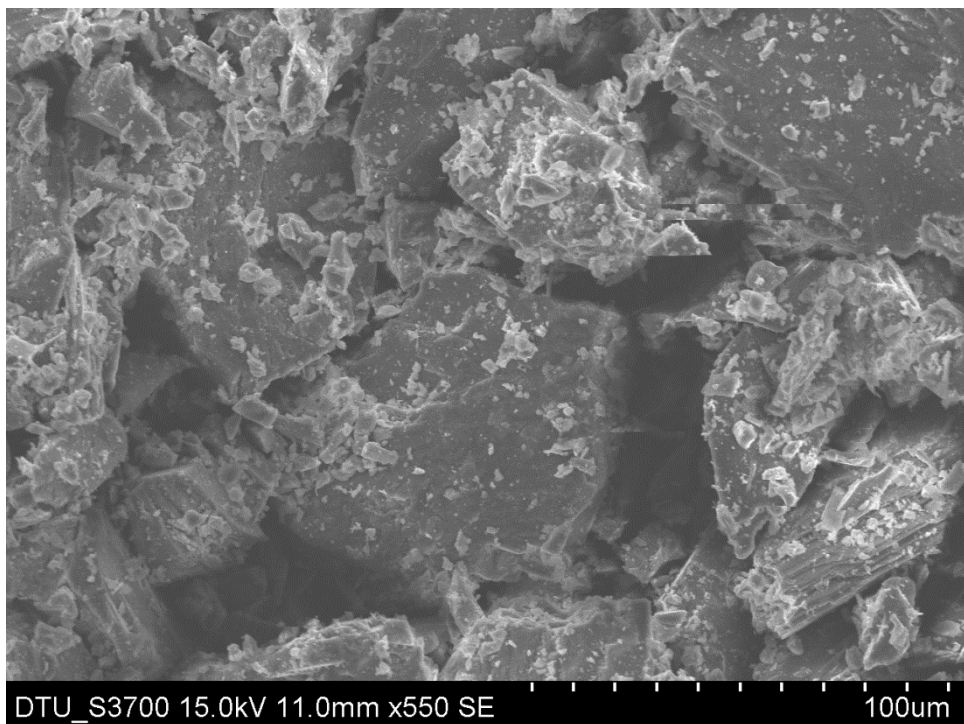


Figure 3.4: SEM image of MSD stabilizer (DTU)

3.8 Specific Gravity IS: 2720 (Part-4) 1980

It is defined as the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at the same temperature. The specific gravity of the soil mixes was determined using density bottle.(IS: 2720 (Part-III, section-2) 1980)



Image 3.3: Specific gravity using density bottle

3.9 Grain size Analysis IS : 2720 (Part 4)-1985

Grain size analysis was performed on soil sample by sieve analysis and hydrometer as per (IS : 2720 (Part 4)-1985)

The plate aperture size is used in the process of sieving to determine the grain size of soil. The weight of soil retained in each stack is used to get the percentage finer "by weight" and obtain the grain size distribution.

The hydrometer test is performed to determine the distribution of particle size finer than 75μ sieve, by sedimentation analysis, using density hydrometer. Its readings are used in conjunction with those of sieve analysis to plot a grain size distribution curve for all ranges of soil particle size.



Image 3.4: Apparatus for grain size distribution analysis.

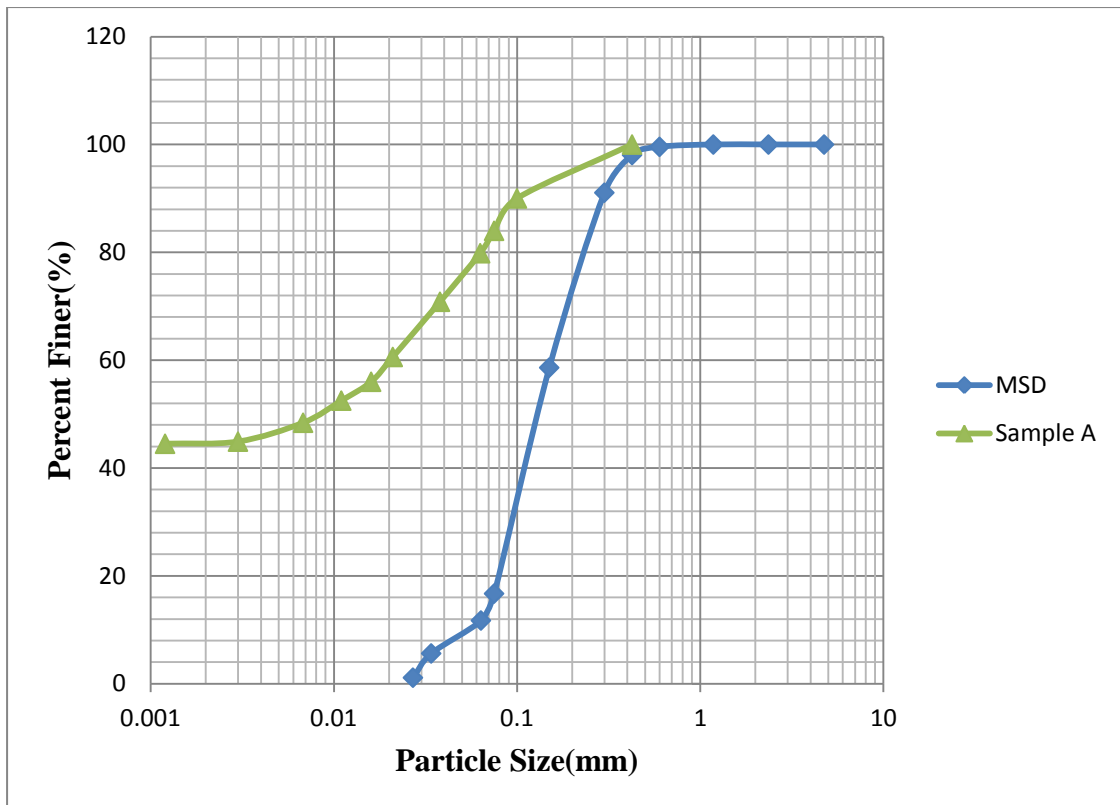


Figure 3.5: Particle size distribution curves for Sample A and MSD

3.10 Liquid limit *IS: 2720 (Part 5) 1985*

Liquid Limit is defined as the water content at which the soil changes from liquid state to plastic state.

The test was conducted in conformity with *IS: 2720 (Part 5) 1985*.



Image 3.5: Casagrande's apparatus to determine liquid limit

3.11 Plastic limit IS: 2720 (Part-5) 1985.

Plastic limit is defined as minimum water content at which soil remains in plastic state. The test was conducted in conformity to IS: 2720 (Part 5) 1985.



Image 3.6: Determination of plastic limit

3.12 Free Swell Ratio

To have information on the swelling property of a soil specimen, Sridharan and Prakash (2004) to characterize the soil expansivity, proposed the free swell ratio method. Free swell ratio (FSR) may be defined as the ratio of equilibrium sediment volume of 10-g oven dried soil passing a 425- μ m sieve in distilled water (V_d) to that of carbon tetra chloride or kerosene (V_k) (Sridharan and Prakash, 2004).

In this study, the recommended procedure of Sridharan and Prakash for FSR was followed and samples were prepared as stated in section 3.3 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.

Classification of soils based on free swell ratio is listed in Table 3.4.

$$FSR = \frac{V_d}{V_k} \times 100$$



Table 3.2: Classification of soils based on free swell ratio (Prakash and Sridharan,2004)

Free Swell Ratio	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



Image 3.7: Free swell Ratio test

Table 3.3 Properties of the swelling soil stabilized with lime

Sample Name	G _s	LL(%)	PL(%)	PI	USCS	FSR
A	2.42	98.9	23.6	75.3	CH	1.88
5L-95A	2.44	81.9	23.9	58.0	CH	1.80
10L-90A	2.46	79.4	24.8	54.6	CH	1.76
15L-85A	2.46	75.2	25.2	50.0	CH	1.65
20L-80A	2.48	73.7	27.3	46.4	CH	1.31
25L-75A	2.49	68.8	28.1	40.7	CH	1.14
30L-70A	2.50	65.9	28.7	37.2	CH	1.02

Table 3.4 Properties of the swelling soil stabilized with MSD

Sample Name	G _s	LL(%)	PL(%)	PI	USCS	FSR
A	2.42	98.9	23.6	75.3	CH	1.88
5M-95A	2.45	80.0	24.4	55.7	CH	1.85
10M-90A	2.47	78.3	24.8	53.6	CH	1.80
15M-85A	2.49	75.1	25.9	49.2	CH	1.71
20M-80A	2.51	72.6	26.5	46.1	CH	1.39
25M-75A	2.52	69.6	27.8	41.8	CH	1.20
30M-70A	2.54	68.9	29.1	39.8	CH	1.13



A: Expansive Soil Sample (75% Local soil passed through 425 μ sieve + 25% Bentonite)

LL: Liquid Limit, PL: Plastic Limit, PI: Plasticity Index,

M: Waste marble slurry dust(MSD)

Naming is explained with the following two examples:

5M-95A: 5% Waste marble slurry dust+ 95% Sample A

10M-90A: 10% Waste marble slurry dust+ 90% Sample A

3.13 Free Swell Test

3.13.1 Compaction of Samples for Free Swell Test

In this study, “Free swell method” was applied using oedometer, 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 3.3 and compacted using static compaction technique into the consolidation ring obtaining a compacted density of density of 1.64 g/cm³. This process was finished until the entire calculated amount of sample was compressed.

For experiments on cured samples, the sample was took and left to cure at room temperature in the dessicator for 7 and 28 days. After, 7 or 28 days passed, sample was put to the required oedometer testing.

3.13.2 Procedure for Free Swell Test

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 a seating load of 5 kPa was applied through the lever arm. Now the dial gauge measuring the vertical deflection was set to zero. The sample was inundated by pouring water through stand pipes 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, 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

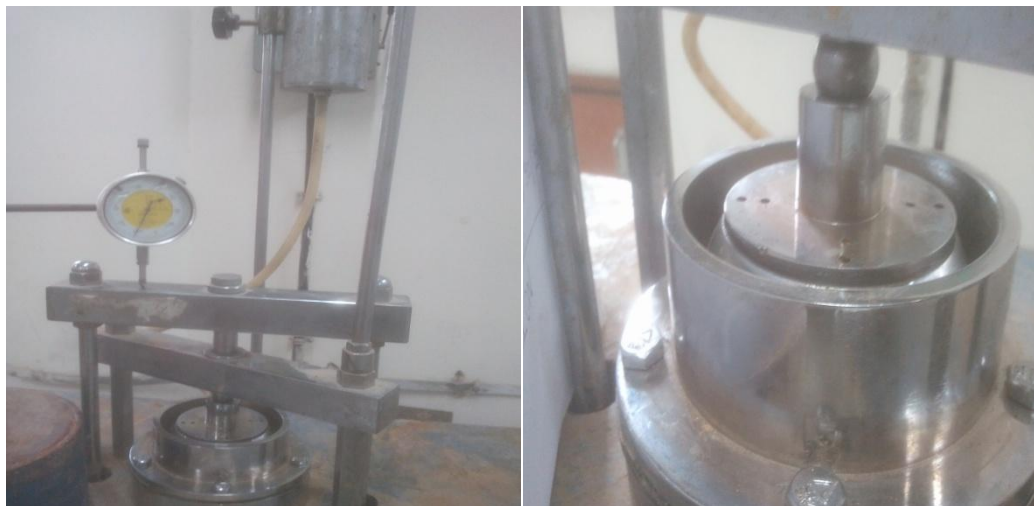


Image 3.8: The Oedometer free swell test

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 marble slurry dust and lime added samples without cure, 7 days cured and 28 days cured are plotted. Effect of addition of the waste marble slurry dust and lime on free swell for samples without cure, 7 days cured and 28 days cured are also presented later under the chapter of results and discussions.

There is no readily available method to measure the rate of swell. Hence, for the purpose of evaluating the rate of swell of experiments, time elapsed for fifty percent swell, (t_{50}) is considered. Effect of adding the waste marble slurry dust and lime on rate of swell for samples without cure, 7 days cured and 28 days cured are plotted .



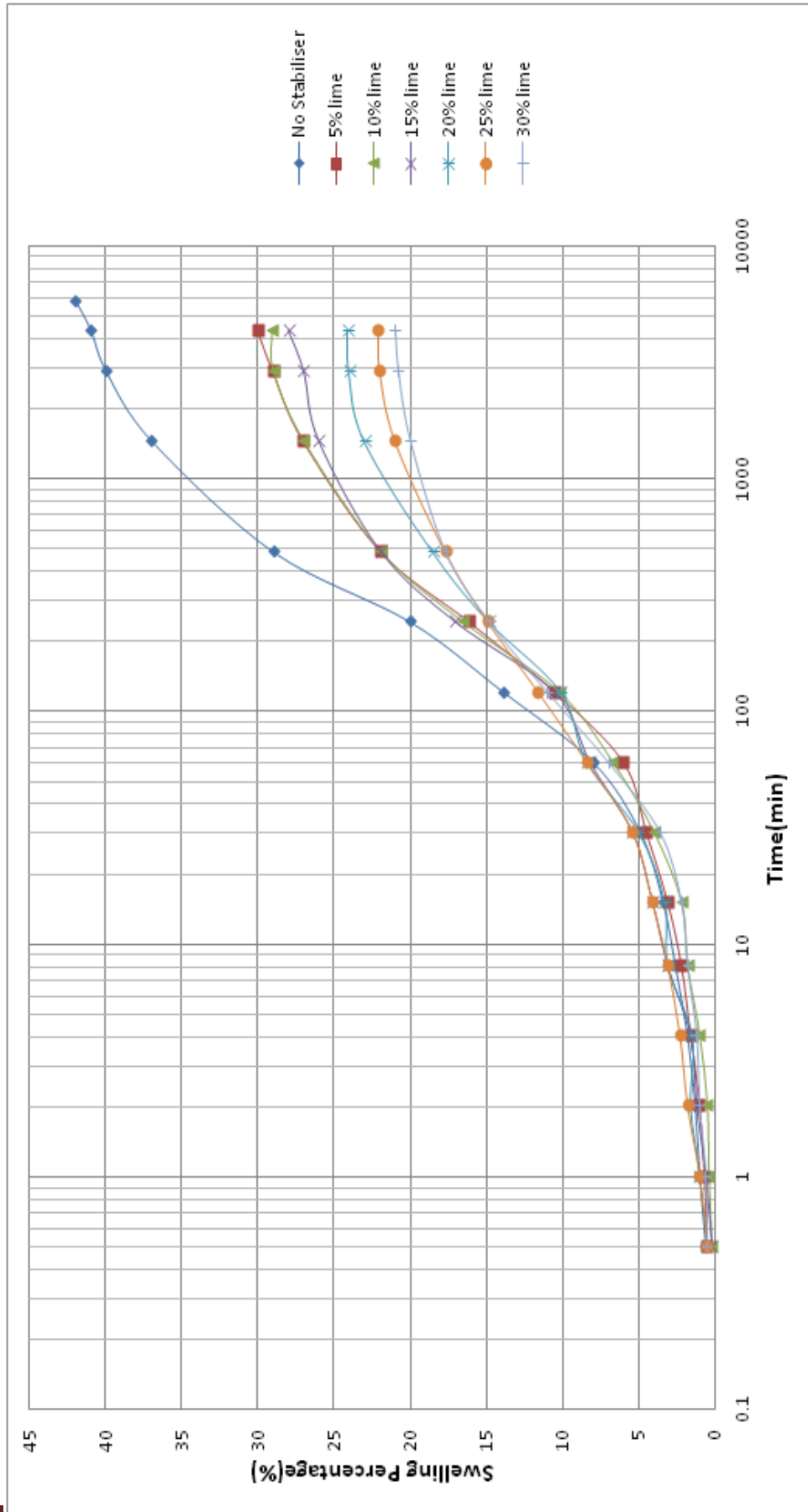


Figure 3.6: Swelling Percentage vs. Time Graph for Sample A added with different amounts of lime, without curing.



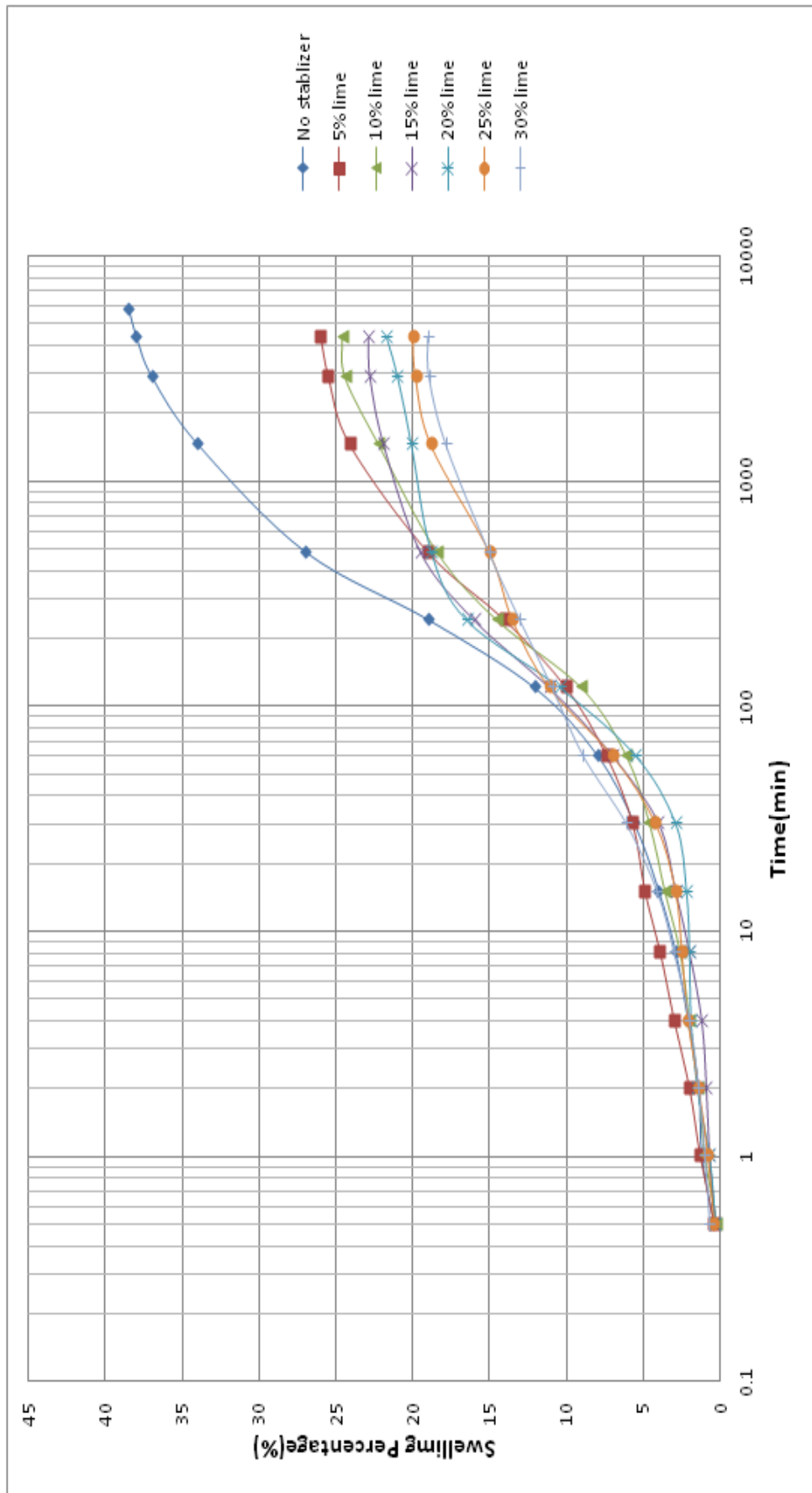


Figure 3.7: Swelling Percentage vs. Time Graph for Sample A added with different amounts of lime, with 7 days curing.



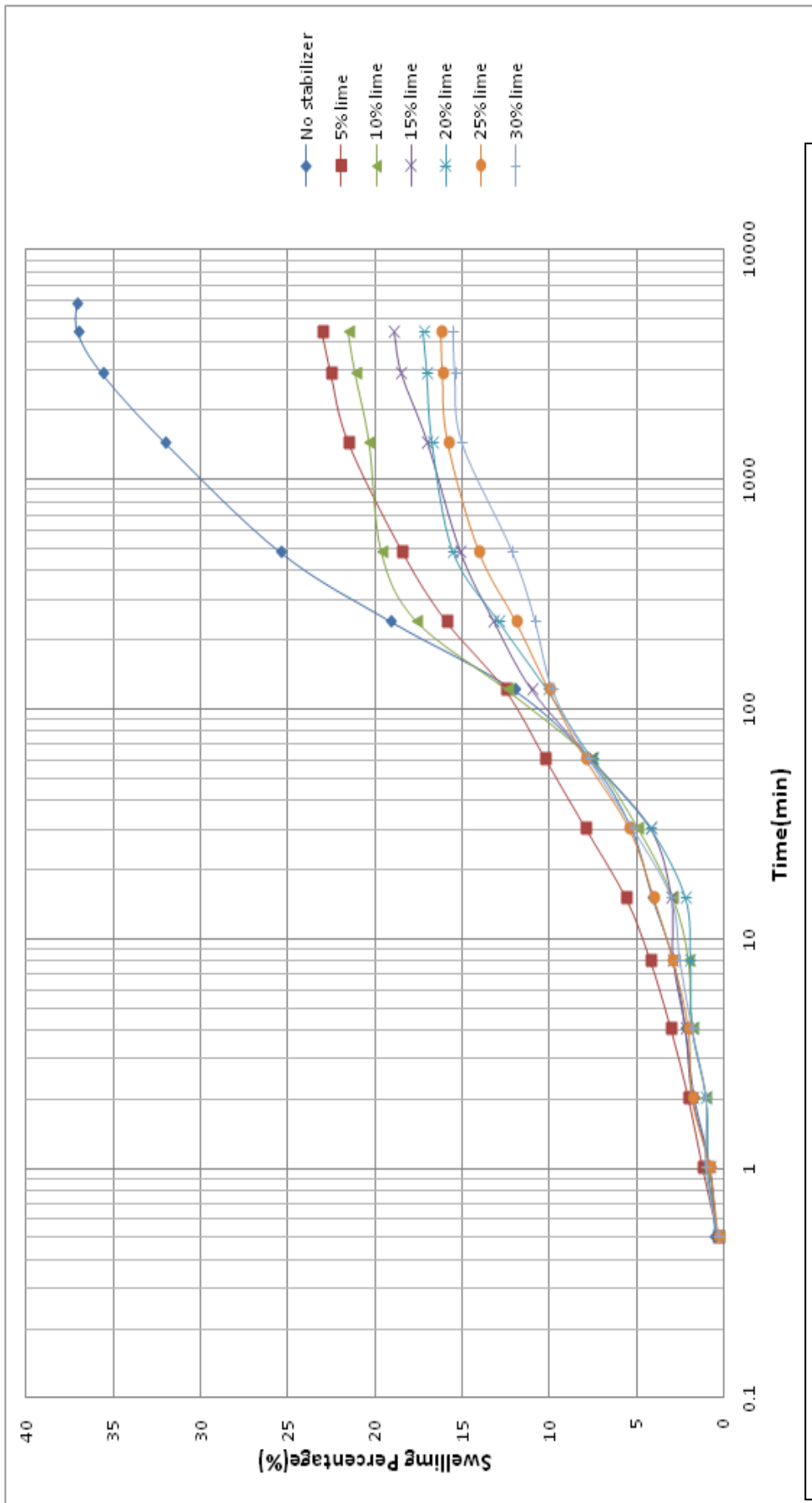


Figure 3.8: Swelling Percentage vs. Time Graph for Sample A added with different amounts of lime, with 28 days curing.



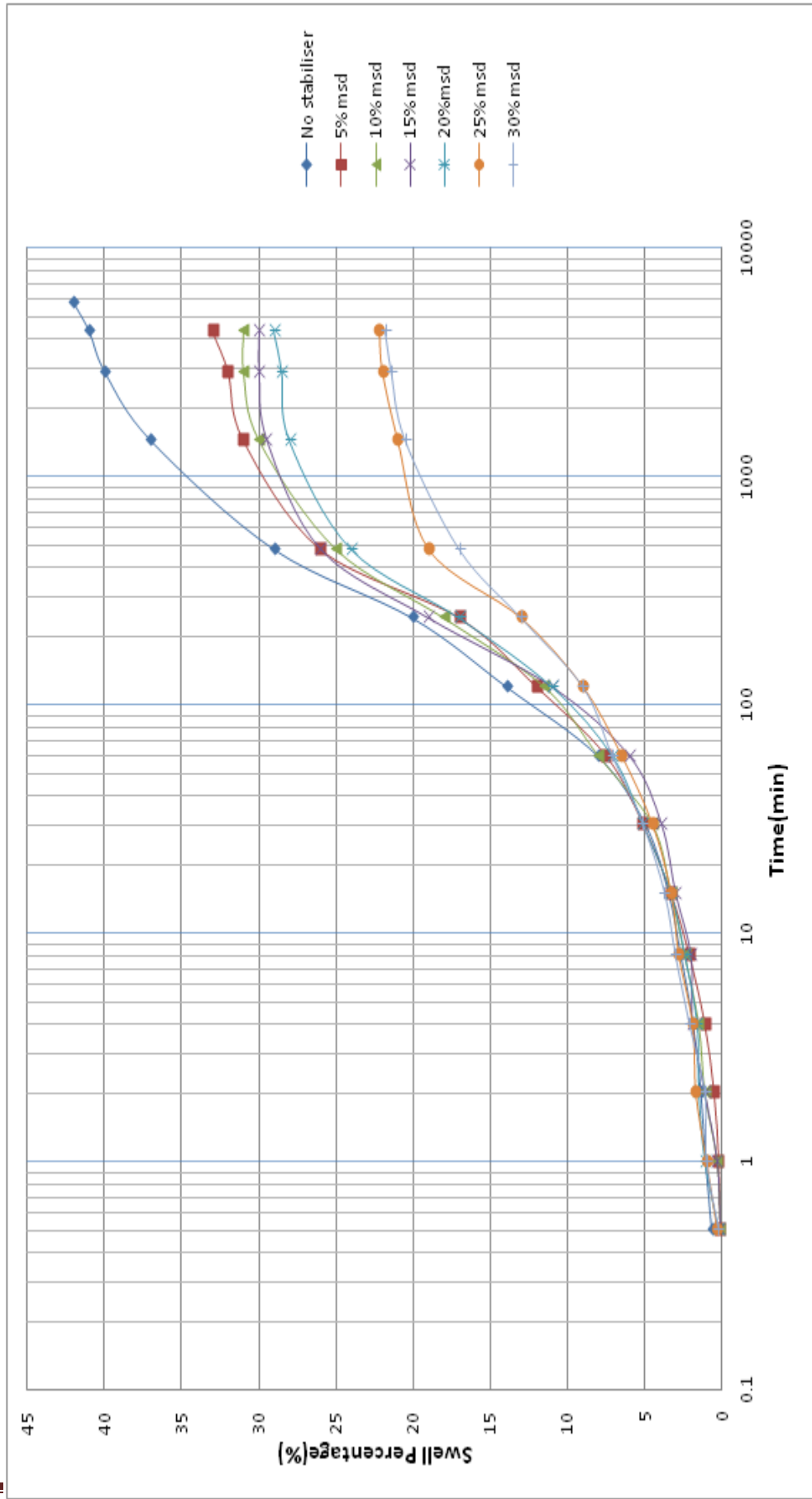


Figure 3.9: Swelling Percentage vs. Time Graph for Sample A added with different amounts of MSD, without curing.



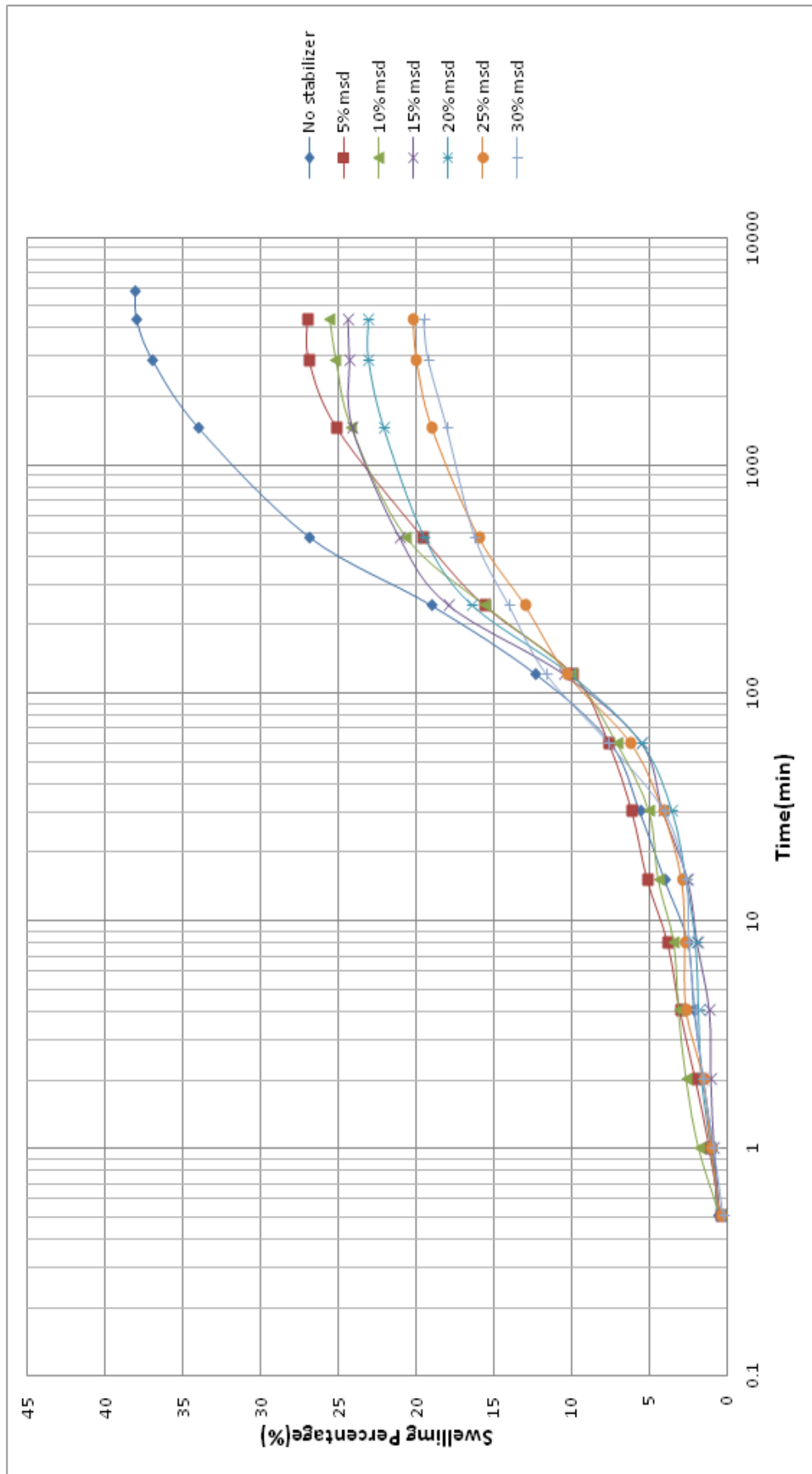


Figure 3.10: Swelling Percentage vs. Time Graph for Sample A added with different amounts of MSD, with 7 days curing.



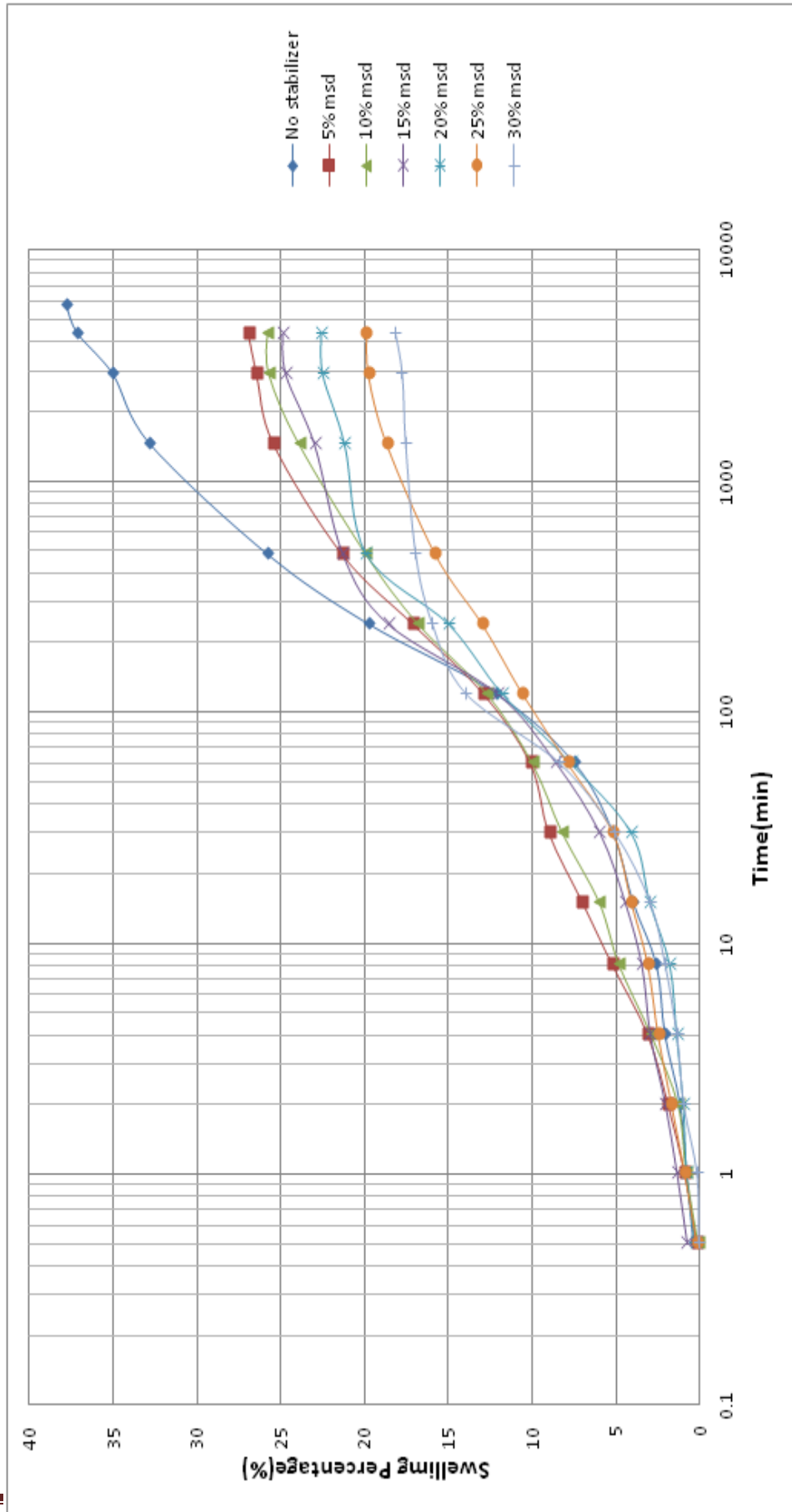


Figure 3.11: Swelling Percentage vs. Time Graph for Sample A added with different amounts of MSD, with 28 days curing.

3.14 Calculations for prediction of swell percentage

A typical log time vs. percent swell relationship generally follows a standard “S” shape curve. This “S” shape curve can be divided into 3 phases; Initial, primary and secondary swelling. Initial swelling observed is generally less than 10% of total swelling. This can be attributed essentially because of the swelling of the clay particles of bentonite 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 completes, slow and continued swelling, i.e. 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 (m) of the straight-line part of $\frac{\text{time}}{\text{free swell}}$ v/s. **time** graph can be used to predict maximum swell.

Dakshnamurthy (1978) proposed a relationship for prediction of free swell and showed that the swelling-time relationship for expansive soils can be represented by a rectangular hyperbola :

$$\% \text{ swell} = \frac{t}{m \times t + c} \quad (1)$$

where,

m is the slope of straight-line part of $\frac{\text{time}}{\text{free swell}}$ v/s. vs. time graphs,

c is the constant and

t is the time at which swelling is required

The validity of this relationship can be demonstrated if a transformed plot of $\frac{\text{time}}{\text{free swell}}$ v/s. **time** graph results in a straight line. The maximum value of swell can be predicted by taking the limit of equation (1) as $t \rightarrow \infty$ as for maximum swell percent, time goes to infinity.

Hence,

$$\% \text{ max swell} = \lim_{t \rightarrow \infty} \frac{t}{m \times t + c} \quad (2)$$



The $\frac{\text{time}}{\text{free swell}}$ v/s. time relationships of the waste marble slurry dust and lime added samples without cure, 7 days cured and 28 days cured are presented in figures 3.12 to 3.17. Predicted and measured free swell percentages and difference between them are presented later in the chapter of results and discussions.



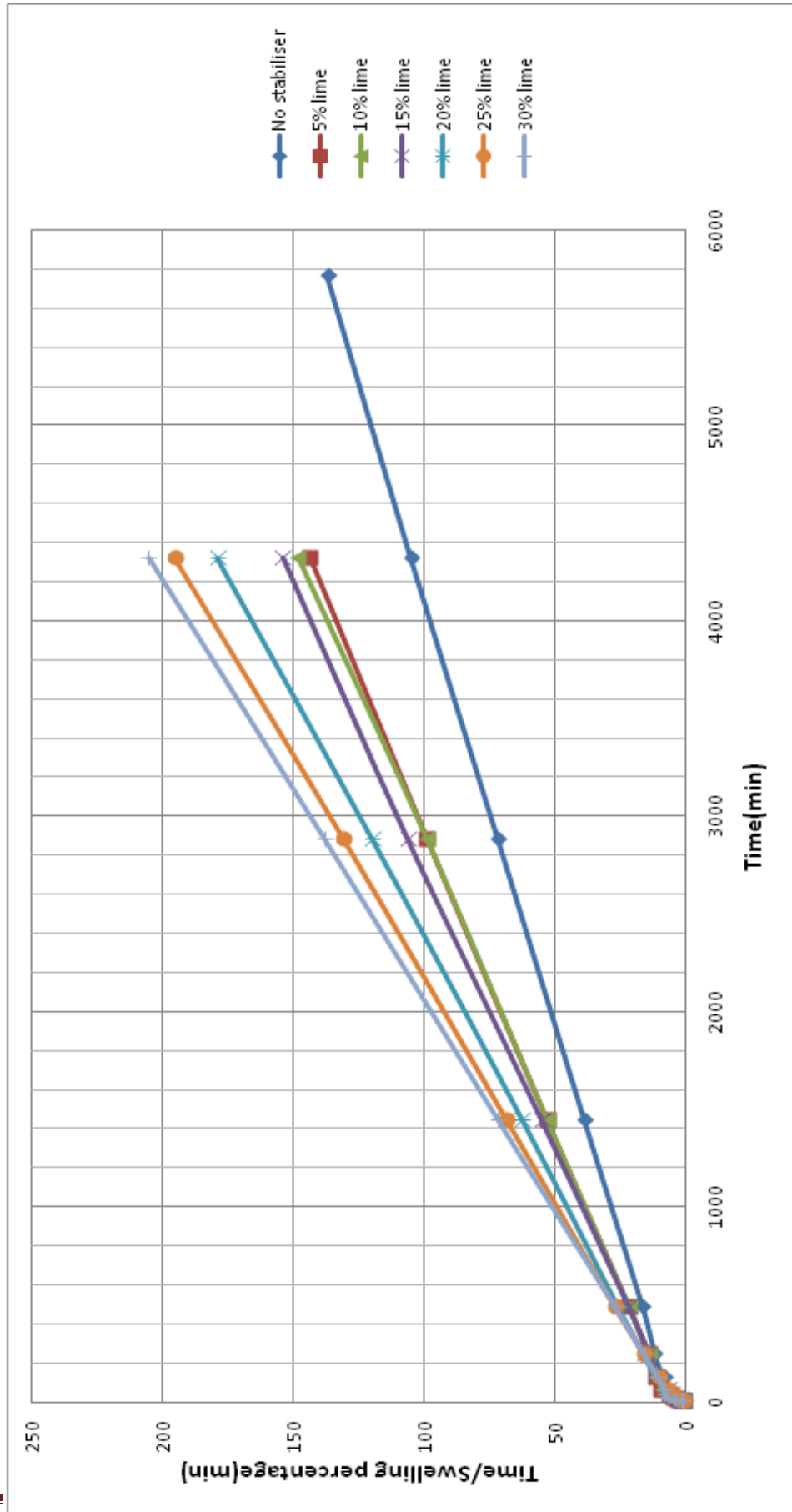


Figure 3.12: Time/Swelling Percentage vs. Time Graph for Sample A added with different amounts of lime, without curing.



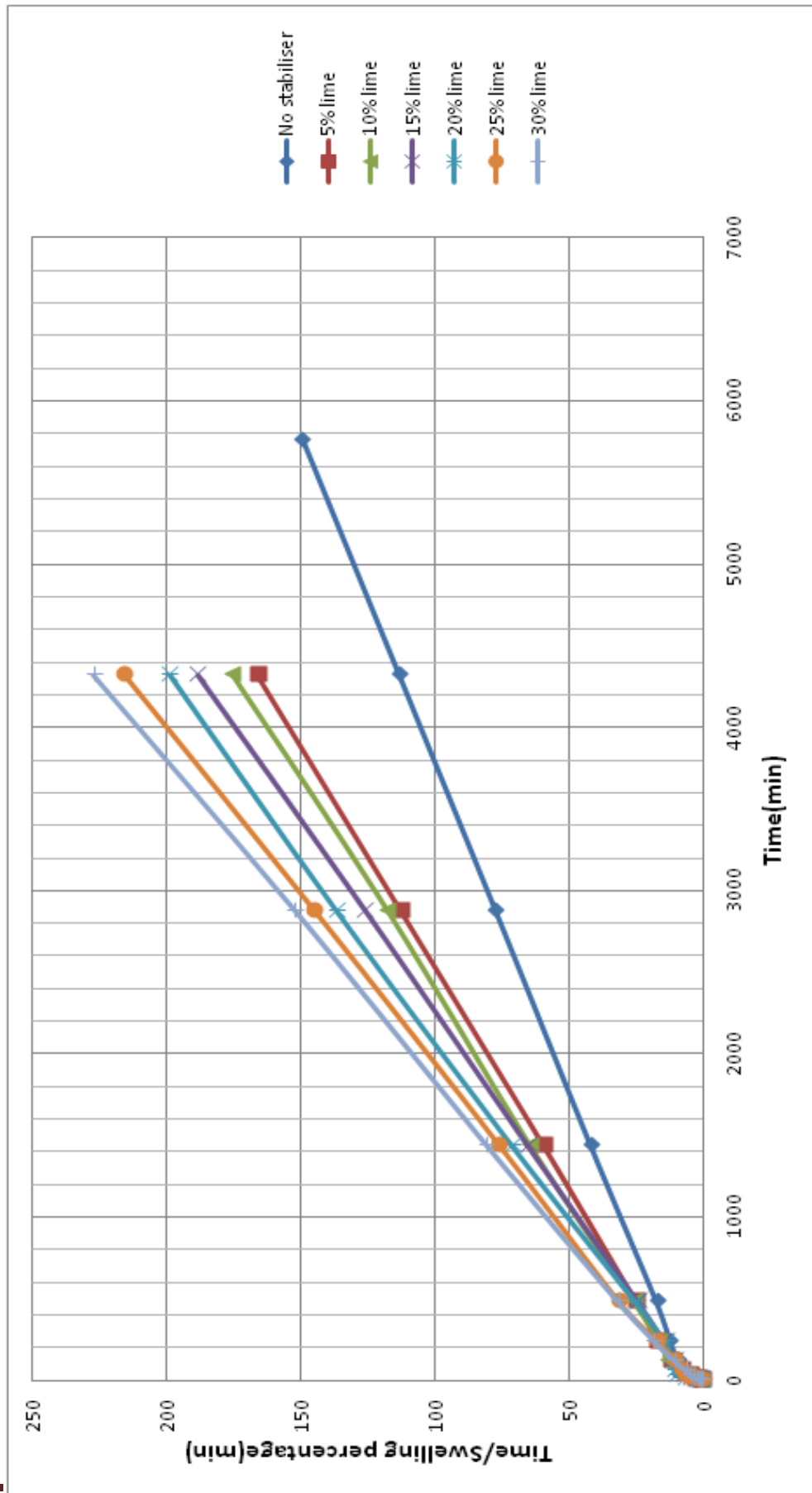


Figure 3.13: Time/Swelling Percentage vs. Time Graph for Sample A added with different amounts of lime, with 7 days curing.



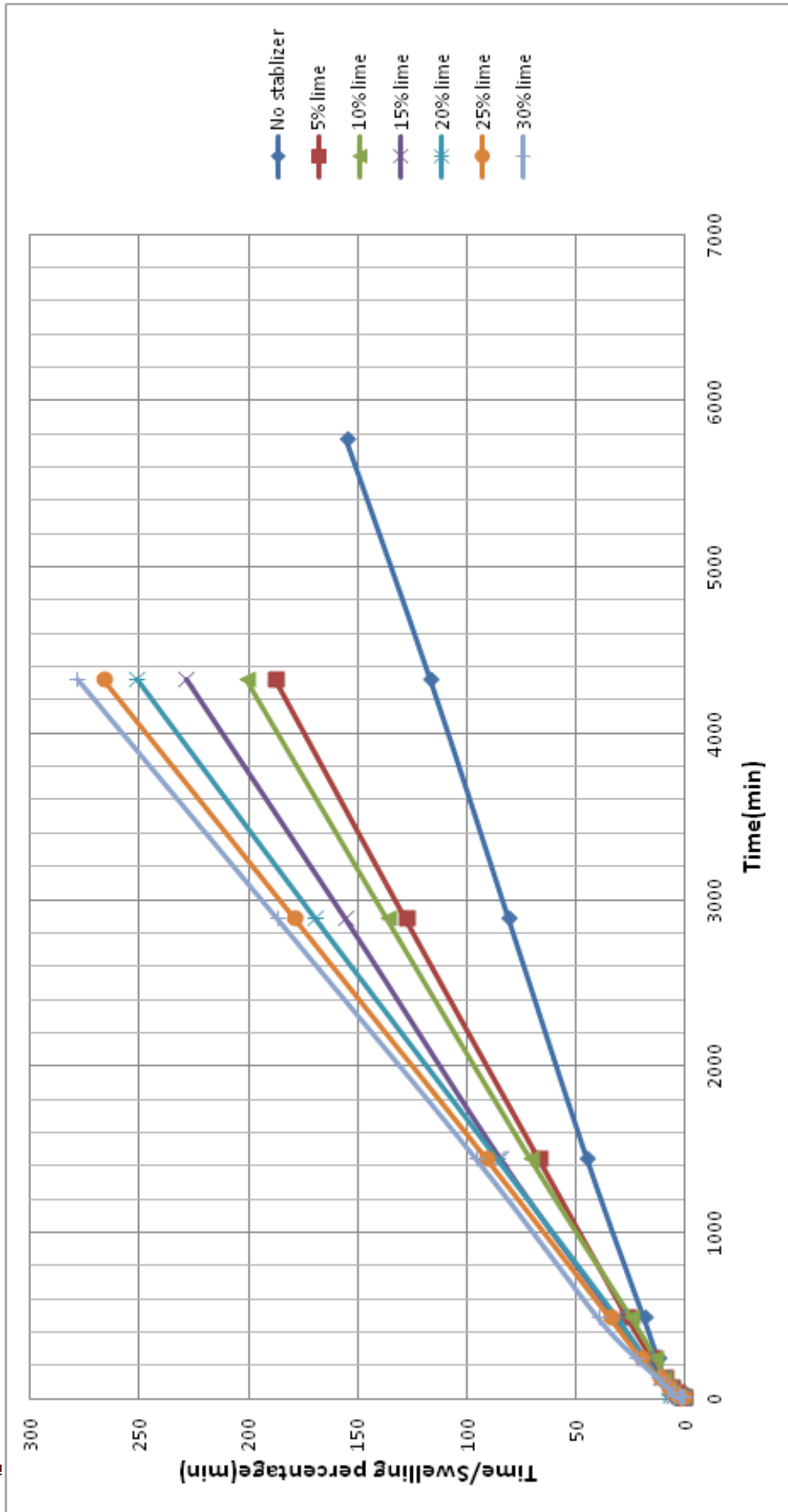


Figure 3.14: Time/Swelling Percentage vs. Time Graph for Sample A added with different amounts of lime, with 28 days curing.



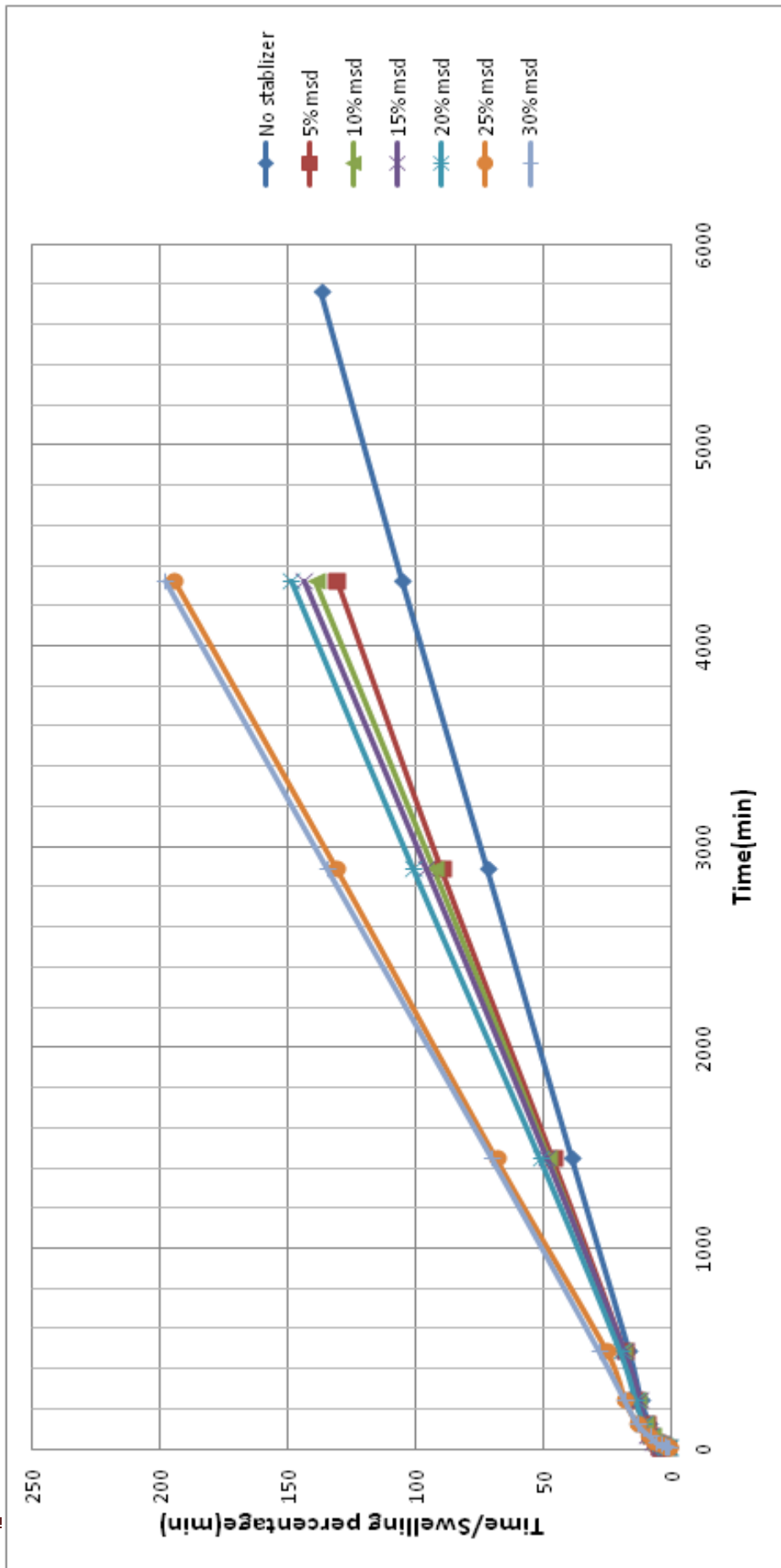


Figure 3.15: Time/Swelling Percentage vs. Time Graph for Sample A added with different amounts of MSD without curing.



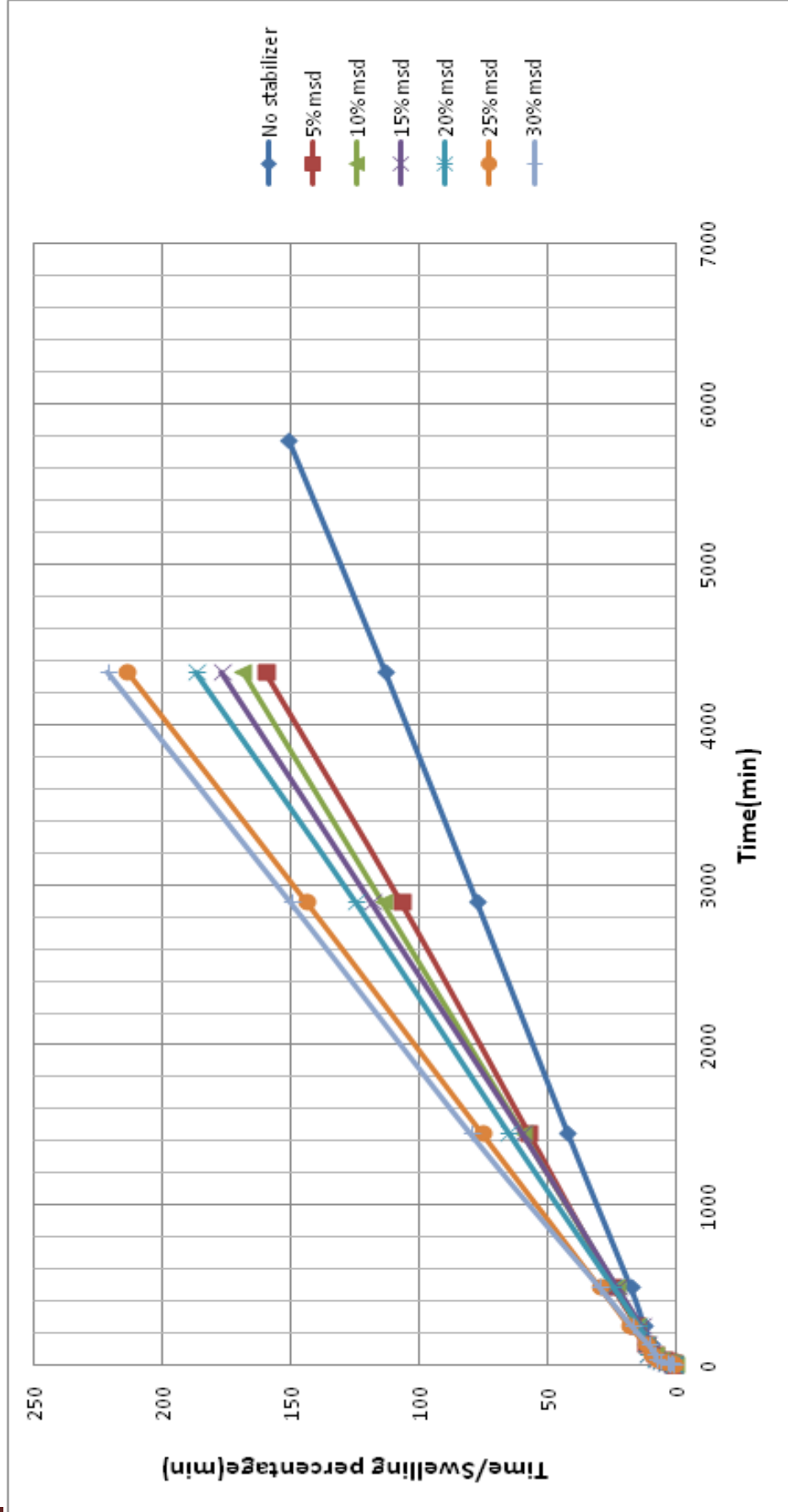


Figure 3.16: Time/Swelling Percentage vs. Time Graph for Sample A added with different amounts of m, with 7 days curing.



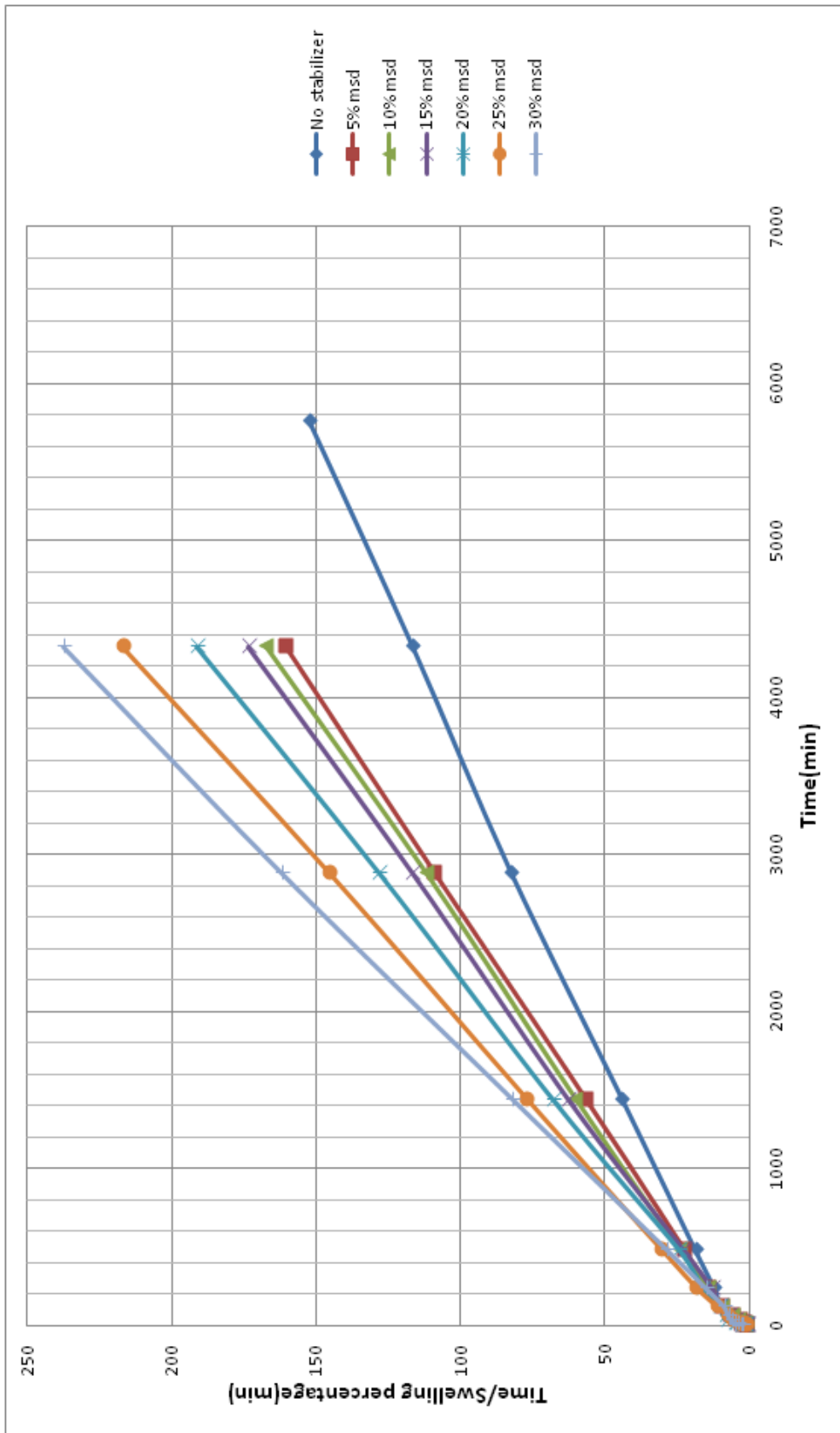


Figure 3.17: Time/Swelling Percentage vs. Time Graph for Sample A added with different amounts of m, with 7 days curing.

CHAPTER 4

DISCUSSIONS OF TEST RESULTS

4.1 SEM Analysis

XRD: The X-Ray Diffraction results shows the presence of calcite and silica in marble slurry dust and presence of calcite in lime when matched with JCPDS (*Joint Committee on Powder Diffraction Standards*)

SEM: Marble Slurry Dust particles can be seen with sharp edges due to cutting with abrasive cutters. The lime particles were also found to be angular in shape.

4.2 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 1.88. 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 1.02 for lime and 1.13 for waste marble slurry dust, with increasing stabilizer percent. A dramatic decrease of Free Swell Ratio occurred after 20 % addition of stabilizers.

4.3 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 lime added samples and that of 30.3 % for waste marble slurry dust added samples.

4.4 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 lime added samples and that of 23.3 % for waste marble slurry dust added samples .



4.5 Effect of Stabilizers on the Plasticity Index of Expansive Soil

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

4.6 Effect of Stabilizers on the Specific Gravity of Expansive Soil

Since the specific gravity of the samples were higher than that of Sample A ($G=2.42$), specific gravity of Sample A increases with addition of stabilizers. Specific gravity of waste marble slurry dust ($G_{s\text{ MSD}}=2.80$) was higher than specific gravity of lime ($G_{s\text{ L}}=2.68$). Thus, the specific gravities of waste marble slurry dust added samples were higher than the lime added samples.

4.7 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 the lime added samples, the reduction of swelling percentage is 26.8 % for only 5 % addition of stabilizer, and maximum reduction is 48.7 % for 30 % addition. For waste marble slurry dust added samples, the reduction of swelling percentage is 19.5 % for 5 % addition of stabilizer, and maximum reduction is 46.8 % for 30 % addition .Lime is more effective than waste waste marble slurry dust on swell percentage because of comparatively higher lime content than the latter. The reduction in swell percentage is due to chemical reactions between lime and expansive soil.

4.8 Effect of Stabilizers on the Rate of Swell of Expansive Soil

The time necessary for 50 % swell of the total swell, decreased with addition of stabilizers, and thus, rate of swell of samples decreased. For lime added samples, the reduction of rate of swell was higher than waste marble slurry dust added samples.



The maximum decrease in rate of swell is 53.8 % for the lime added samples and 26 % for waste marble slurry dust added samples. The reduction was nearly same for 25 % lime and 30 % lime samples.

Table 4.1 Percentage changes (decrease) in Swelling Percentage (%) and t_{50} (min) for lime stabilised samples without cure, 7 days cured and 28 days cured

Sample	Samples without cure		Samples with 7 days cure		Samples with 28 days cure	
	(%)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
5L-95A	26.8	12.1	31.5	16.3	37.8	51.2
10L-90A	29.0	23.9	35.2	25	41.8	54.7
15L-85A	31.7	27.9	39.7	47.9	48.9	57.1
20L-80A	41.2	48.2	42.8	54.8	53.5	59.5
25L-75A	46.0	50.1	47.3	58.3	56.2	61.9
30L-70A	48.7	53.8	50	60.4	58.1	76.2



Table 4.2 Percentage changes (decrease) in Swelling Percentage (%) and t_{50} (min) for MSD stabilised samples without cure, 7 days cured and 28 days cured

Sample	Samples without cure		Samples with 7 days cure		Samples with 28 days cure	
	(%)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
5M-95A	19.5	14.7	28.9	17.5	27.4	28.5
10M-90A	24.3	20.2	32.6	20.8	30.4	34.6
15M-85A	26.8	22.5	35.7	31.2	32.8	38.1
20M-80A	29.2	24.7	39.1	41.6	39.8	47.6
25M-75A	45.8	25.1	46.1	52.1	46.9	57.4
30M-70A	46.8	26	48.6	64.6	50.9	70.3

4.9 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 compare with all samples. Especially, lime added samples show higher reduction in swelling percentage after cured.

The maximum decrease in swell percentage is 58.1 % for lime added samples and 50.9 % for waste waste marble slurry dust added samples after 28 days curing. Even,



5 % addition of lime resulted in 37.8 % reduction in swelling percentage after 28 days curing . Except 5 %, 10 % and 15 % MSD samples, gradual decrease of swelling percentage can be summarized as;

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

4.10 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 compare all the samples.

The 28 days cured samples peculiarly had much tendency to decrease in rate of swell.

The maximum decrease in t_{50} is 76.2 % for lime added samples and 70.3 % for waste marble slurry dust added samples after curing .Even 5 % addition of lime resulted in 51.2 % reduction in t_{50} after 28 days curing . Gradual decrease of rate of swell can be summarized as;

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

4.11 Discussion on Prediction of Swell Percentage

The mean of differences between predicted and measured swell percentages of samples are .93 %, 0.53 % and 0.84%, 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, as proposed gives consistent results for this study.





Sample	Samples without cure			Samples with 7 days cure			Samples with 28 days cure		
	Measured swell (%)	Predicted Swell (%)	Difference	Measured swell (%)	Predicted Swell (%)	Difference	Measured swell (%)	Predicted Swell (%)	Difference
Sample A	41	43.15	2.15	38	40.11	2.11	37.1	38.4	1.3
5M-95A	33	35.19	2.19	27	27.2	0.2	26.9	27.96	1.06
10M-90A	31	31	0	25.6	26.44	0.84	25.8	26	0.2
15M-85A	30	30	0	24.4	24.6	0.2	24.9	25.33	0.43
20M-80A	29	30.05	1.05	23.1	23.1	0	22.6	22.8	0.2
25M-75A	22.2	22.61	0.41	20.2	20.61	0.41	19.9	20.1	0.2
30M-70A	21.8	22.64	0.84	19.5	20.12	0.62	18.2	19.05	0.85

Table 4.3: Predicted and measured Swell values for the test samples with MSD as stabilizer



Sample	Samples without cure			Samples with 7 days cure			Samples with 28 days cure		
	Measured swell (%)	Predicted Swell (%)	Difference	Measured swell (%)	Predicted Swell (%)	Difference	Measured swell (%)	Predicted Swell (%)	Difference
Sample A	41	43.15	2.15	38	40.11	2.11	37	40.15	3.15
5L-95A	30	32.22	2.22	26	27.06	1.06	23	24.07	1.07
10L-90A	29.1	29.3	0.2	24.6	25	0.4	21.5	22.34	0.84
15L-85A	28	30.24	2.24	22.9	23.1	0.2	18.9	19.75	0.85
20L-80A	24.1	24.3	0.2	21.7	21.4	-0.3	17.2	17.61	0.41
25L-75A	22.1	22.3	0.2	20	20.41	0.41	16.2	16.4	0.2
30L-70A	21	21.41	0.41	19	19.2	0.2	15.5	15.7	0.2

Table 4.4 : Predicted and measured swell values for the test samples with lime as stabilizer

CHAPTER 5

CONCLUSION

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

1. Addition of waste marble slurry dust and lime decreased liquid limit, plasticity index and, increased plastic limit of Sample A.
2. Free swell ratio of Sample A decreased with addition of stabilizers. As the percentage of stabilizer increased, free swell ratio decreased.
3. By addition of stabilizers, the swelling percentage decreased considerably. The reduction was higher for the samples added with lime.
4. By addition of stabilizers, the t_{50} values were decreased. In other words, samples having more stabilizers reached the 50 % of total swell quicker.
5. Swelling percentage and rate of swell of samples decreased by curing. Curing was more effective for the lime added samples. Also, curing was more effective for rate of swell of samples than swelling percentage.
6. The predicted swell and measured swell percentages of samples were close to each other.
7. 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.



General Discussion

The addition of waste marble slurry dust and lime 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) reduce the plasticity index (PI) of soil, and also the swelling potential.

In one process, a base exchange occurs with the calcium ions of waste marble slurry dust and lime 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 marble slurry dust and lime resulted in the formation of aggregations which reduced the swelling potential of the soil.

Future Scope

The swell potential tests were performed on samples treated with waste marble slurry dust and lime 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 for each sample.

Also the variations in shrinkage limits, shrinkage index and behaviour of particle distribution for each sample mix will give more accurate findings in relation to the swelling properties of swell.

Further the effect of other waste materials like fly ash, rice husk can be determined and a proper comparison can be done with regard to the most economical and environment friendly stabilizer.



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