EFFECT OF QUARRY DUST ENHANCEMENT ON ENGINEERING BEHAVIOR OF EXPANSIVE CLAYS

A THESIS SUBMITTED TO THE GRADUATE SCHOOL OF APPLIED SCIENCES

OF

NEAR EAST UNIVERSITY

By

SALAH YASEEN MOHAMMED SALEM AL-DUBAI

In Partial Fulfilment of the Requirements for the Degree of Master of Science

in

Civil Engineering

NICOSIA, 2018

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Approval of Director of Graduate School of
Applied Sciences

Prof. Dr. Nadire ÇAVUŞ

We certify that, this thesis is satisfactory for the award of the degree of Master of Science in Civil Engineering

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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To my family…

ABSTRACT

The volumetric change of expansive soil is considered as a challenge for geotechnical engineering. Such problems have cost governments millions of dollars; therefore, many methods were produced in order to stabilize such soils. The stabilization method has to be safe for environmental point of view.

In this study Quarry Dust (QD) by proportion of 10, 20, and 30% was used, in order to study the enhancement of quarry dust as stabilization material for four types of clays characterized as potential expansive soil. The soils were collected from three different locations in Northern Cyprus. The obtained soils were examined according to the American Society for Testing and Materials ASTM. The tests were carried out in this study are specific gravity test, hydrometer test, proctor compaction test, one-dimensional swell, one-dimensional consolidation, and unconfined compressive strength.

Atterberg limits showed an overall decrease with addition of 10, 20, and 30% QD. Also, there was enhancement on the swell behavior with the increases of the QD as well as the compressive strength. Moreover, the water absorption during the swell reduced with the increase of QD proportion. The compressibility was also decreased at all proportion additions.

Keywords: Quarry dust; soil stabilization; expansive clays; volumetric change; compressive strength

ÖZET

Jeoteknik mühendislik bakımından şişen zemine bağlı hacimsel değişiklikler büyük bir zorluk olarak görülmektedir. Bu tarz sorunlar hükümetlere milyarlarca dolara mal olduğundan bu tip zeminleri iyileştirmek için birçok yöntem geliştirilmiştir. Zemin iyileştirme yöntemlerinin çevresel yönden de güvenilir olması şarttır.

Bu çalışma sırasında potansiyel şişen zemin olarak sınıflandırılan dört tip kil toprağına yüzde 10, 20 ve 30 oranlarında taş ocağı tozu eklenerek, bu malzemeyi zemin iyileştirme materyali olarak kullanılma olasılığı araştırılmıştır. Bu toprak türleri kuzey Kıbrıs'in farklı bölgelerinden toplanmıştır. Toplanan toprak türleri Amerikan topluluğu deney standart (ASTM) yöntemine göre değerlendirilmiştir. Bu çalışma sırasında uygulanan testler arasında özgül ağırlık deneyleri, hidrometre analizi, standart proktor sıkıştırma deneyi, tek eksenli şişme, tek eksenli konsolidasyon ve serbest basınç mukavemet deneyi bulunur.

Yüzde 10, 20 ve 30 oranlarında yapılan taş ocağı tozu eklenmesinin ardından Atterberg limitlerinde düşme saptanmıştır. Aynı zamanda taş ocağı tozu ve basınç dayanımında artış ile birlikte şişme davranımında da yükselme saptanmıştır. Diğer yandan şişme süresince taş ocağı tozu oranının yükselmesi ardından su emilimi azalmıştır. Kompresibilite oranında, genel olarak her bir yüzdeliğin eklenmesine bağlı düşme görülmüştür.

Anahtar Kelimeler: Taş ocağı tozu; zemin iyileştirilmesi; şişen kil; hacimsel değişiklik; basınç dayanımı

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LIST OF ABBREVIATIONS

QD:	Quarry dust
LL:	Liquid limit
PL:	Plastic limit
PI:	Plasticity index
Gs:	Specific gravity
ASTM:	American Society for Testing and Materials
OMC:	Optimum moisture content
MDD:	Maximum dry density
γ_d:	Dry density
Ks:	Coefficient of hydraulic conductivity
ΔH/H0:	Swell potential
UCS:	Unconfined compressive strength
w_a:	Water absorption

CHAPTER 1

INTRODUCTION

Some clayey soil have seasonal ability for volumetric change due to their capability of absorbing water (Nelson & Miller, 1992). However, this volume change induces a ground movement which causes damage to buildings. Low-rise buildings are more exposed to such problem since they don't have adequate weight to resist. Moreover, the effect of such phenomena is clearly noticeable in arid and semi-arid zones due to the differences in the amount and the period of precipitation and evaporation (Jefferson, 2001).

The problematic phenomena of expansive soils may cause cracks in buildings and roads, due to swelling-shrinking behavior beneath pavements and foundations, which is considered as a challenge for geotechnical engineering as well as economical problem for governments due to more cost incurred. It was reported that the cost of expansive soil damages in US has achieved the annual average cost of damages by hurricanes, earthquakes, floods, and tornadoes combined (Wyoming Office of Homeland Security, 2014).

Many methods had been established such as mechanical, chemical methods and soil stabilization by additives, in order to improve the engineering properties of those soils. In some cases, traditional earth material is more desirable due to its low cost, also industrial by-products can attribute as superior additives.

The main purpose of this study is to stabilize the obtained expansive clays using Quarry Dust. Physical and mechanical properties are to be implemented for carrying out the study which includes swell, consolidation, hydraulic properties, and unconfined compressive strength. The Quarry Dust is waste material found at some mining sites accumulated in open areas, thus their presences might be risky to the environment causing asbestos health problems when inhaled and destroys crops around the mining sites. Using it on expansive clays of Cyprus could be a suitable solution to stabilize the soil while solving the storing problem of quarry dust in the quarrying sites.

1.1 Thesis Objective

The main objective of this thesis is to study the effect of quarry dust and engage it as a stabilizer material for expansive soil by monitoring the effects of it on four types of clayey soil, characterized as expansive. However, comparing the mechanical and physical properties of soils after mixing with different proportions and assess the impact at each proportion.

1.2 Thesis Outline

This thesis consists of five chapters; the first chapter presents the introduction and the objective of the thesis. The second chapter presents the literature review and previous studies on the soil stabilization by addition of different materials. The third chapter presents the material that was used in this investigation and the methodology that was implemented on the soil quarry dust mixture in order to obtain the properties. The fourth chapter presents the results and discussions of this study. The fifth chapter contains conclusions and recommendations for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Expansive soil is an unstable type of soil which presents a change in volume (swelling and shrinkage) during wet and dry seasons according to the change in soil moisture content. This change of volume happens due to different criteria such as the type of soil and the mineralogy that it consists, however, the cyclic volume change for such soil causes serious damages on structures, highways, and pavements (Wyoming Office of Homeland Security, 2014).

Expansive soils cost governments billions of dollars around the world. In Australia, the expansive soil was found in Adelaide city and the damages that happened affected about 600,000 people. In Canada due to the wide variety of climate and geological structures a lot of foundation problems exist, furthermore, in western Canada, expansive soils have severely affected infrastructures. In India, the expansive soil called "Black cotton" soil covers approximately 200,000 square miles of lands and it is recognized by its high hardness when dry, and having high swell potential while wet. In United States of America expansive soil is found in many states and it is classified according to the severity. Colorado, Texas and Wyoming are severe; California, Utah, Nebraska, and South Dakota are moderate, and in Oregon, Montana, Arizona, Oklahoma, Kansas, Alabama, and Mississippi is mild. Furthermore, it was reported that the estimation of damages due to the volume change of these soils cost 2,255 million of dollars annually which means that damages caused by expansive soil achieved the combined average annual damages from earthquakes, floods, hurricanes, and tornados. Therefore, the expansive soil is considered as a worldwide problem (Chen, 1975).

Clay soil is the most important phase of expansive soil since it contains Montmorillonite which has the capability of absorbing water and presenting volumetric change (Das, 2008). Expansive soil with constant water content will not usually cause a problem, however, as liquid limit increases, the plasticity index increases, according to the American Society of

Testing Materials "ASTM". Test method D 4829 presents the “expansion index” to quantify the results, the range of expansive soil and its swell potential. Expansion index limits ranging from 0 to 20 is regarded as very low, from 21 to 50 is low, from 51 to 90 is medium, from 91 to 130 is high, and plasticity index over 130 presents very high expansion potential.

2.2 Mineralogy

Understanding the behavior of expansive soil primarily depends on understanding the mineralogical composition of clays due to their responsibility for the changes in volume. Clay minerals consist of basic structural units of silica tetrahedron and alumina octahedron. The tetrahedron unit represents four oxygen atoms surrounding a silica atom when tetrahedrons merge together they form silica sheets. In octahedron case six of hydroxyls surround an alumina unit, these combination merges to form gibbsite sheet (octahedral sheet). Sometimes aluminum are replaced by magnesium to form brucite sheet (Das, 2008).

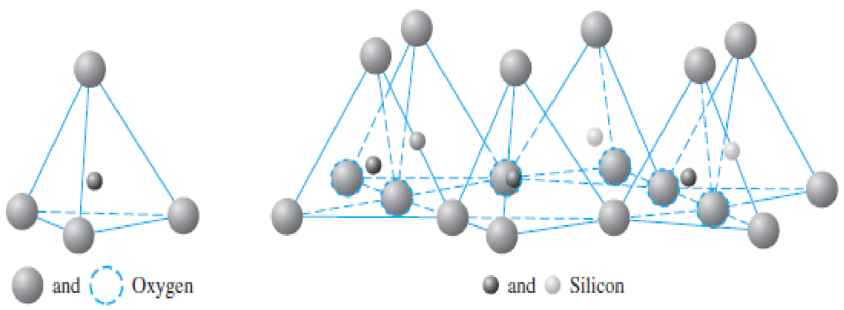

Figure 2.1: Silica tetrahedron and silica sheet (Das, 2009)

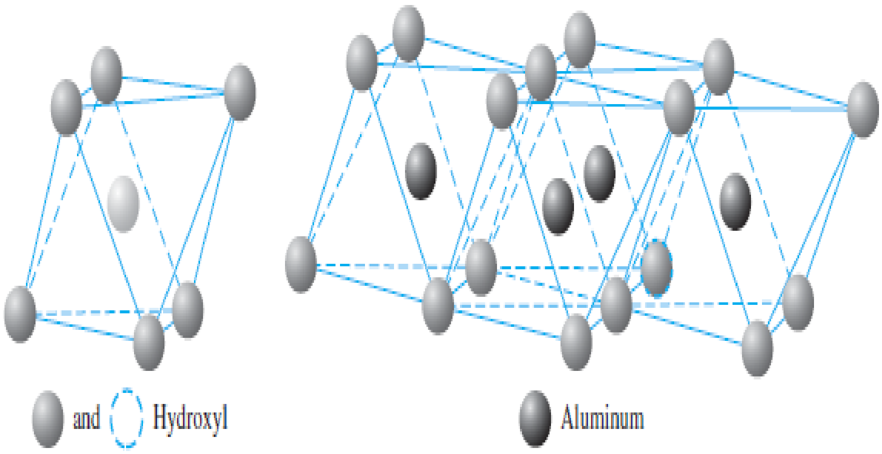

Figure 2.2: Alumina octahedron and alumina sheet (Das, 2009)

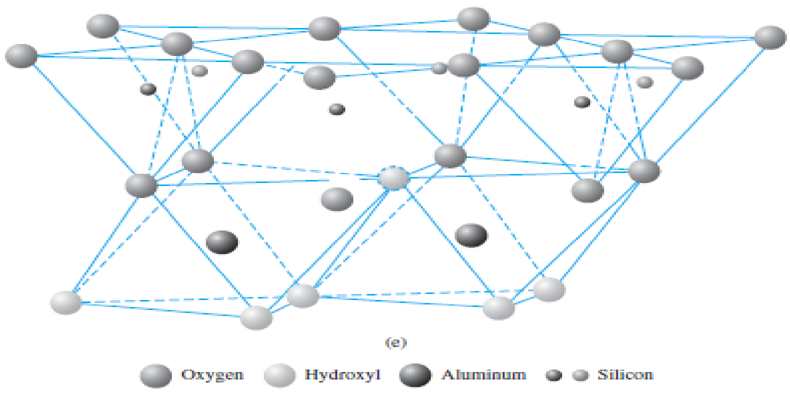

Figure 2.3: Elemental silica-gibbsite sheet (Das, 2009)

The most three clay minerals, Kaolinite, Illite, and Montmorillonite, are an accumulation of some sheets. Kaolinite consists of two layers of elemental silica gibbsite sheets or silica brucite sheet. These layers are connected to each other by hydrogen bonders, and they occur as platelets. Illite consists of three sheets bonded to each other, one of these sheets is gibbsite sheet which is located at the middle, and the others two are silica sheets, located at the top and bottom. The bonder between these layers is potassium ions. The montmorillonite structure also consists of two silica sheets, one at the top and the other at the bottom, and one gibbsite sheet in the middle. This structure is similar to that in illite with one difference which is the bonder between the layers. Spaces between the layers in montmorillonite are full of water (Das, 2009).

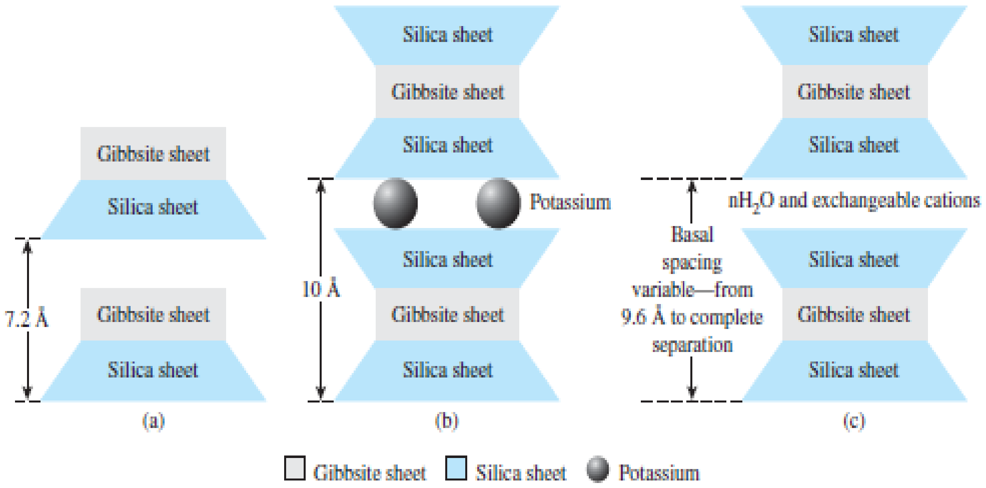

Figure 2.4: (a) kaolinite structure, (b) illite structure, (c)montmorillonite structure (Das, 2009)

2.3 Mechanism of Expansive Clay

The phenomenon of swelling is presented in two phases, one is called intercrystalline swelling and the other as intracrystalline swelling.

-Intercrystalline swelling is found in any type of clay deposit despite its mineralogical compassion. The particles of clay in its relatively dry status, are held together by retained water due to tension effect of capillary forces. The capillary force reduces when the clay is wet, which leads to the expansion of clay.

- Intracrystalline swelling happens in smectite types, particularly montmorillonite. The crystals of montmorillonite are made by individual molecular layers; however, these crystals are weakly bonded and in wetting condition water enters between the crystals, moreover between the layers which encompass the crystals. The swelling in calcium montmorillonite is much less comparing with swelling in sodium montmorillonite (Jefferson, 2001).

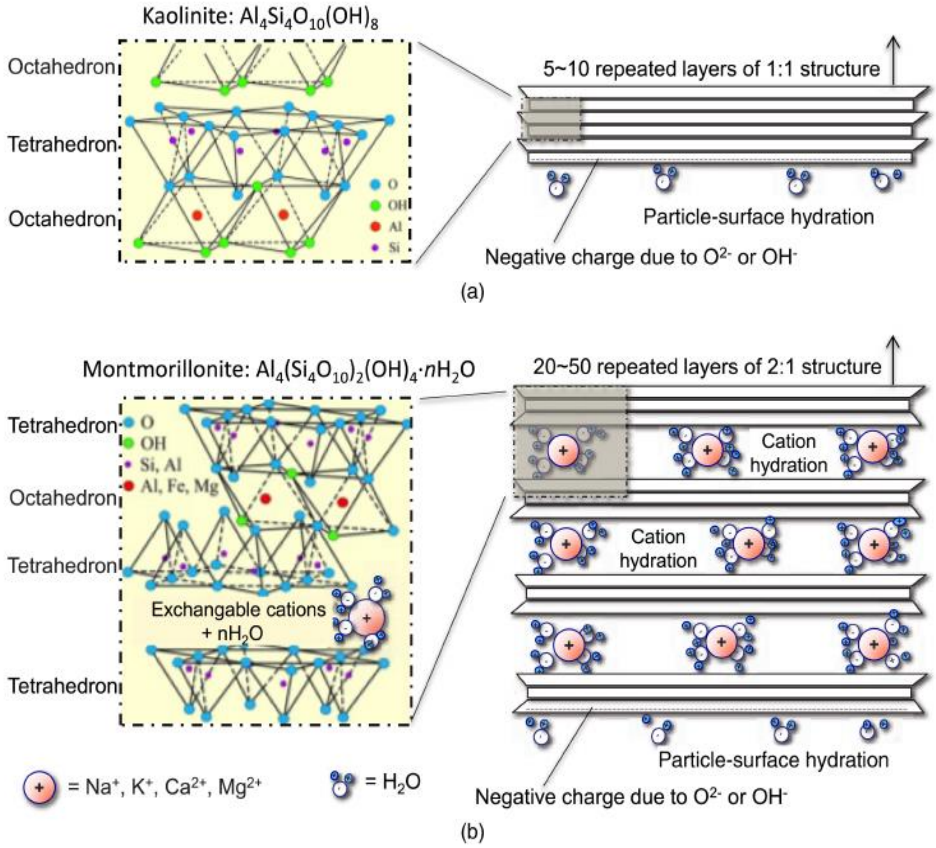

Figure 2.5: Mechanism of expansive soil (Lu & Khorshidi, 2015)

2.4 Expansive Clay Types

There are many types of expansive clays all over the world. Some of these type names and their properties are as defined below.

1- ***Black Cotton, BC:*** This soil is a clay type that could be found in many parts of India, it is mostly concentrated in the middle and western parts and it covers approximately 20% of the total area of India. This type of soil is very rich in montmorillonite which leads to its high capability to expand. Its property of high expand causes severe problems, in some cases the cracks that are caused by this soil reach to 12" deep. BC soil is recognized by its high hardness in dry condition and losing this hardness while wet and by exhibiting high swell potential (Oza & Gundaliya, 2013).

2- ***Red soil or red earth soil:*** This type of soil contains kaolinite. It was called red because of its red color due to the high presence of iron. This kind of soil is acidic in nature and is not able to retain moisture(Manjunath, Kuma, & Kumar, 2012).

3- ***Bentonite clay:*** This type of clay is used to produce low permeability barriers due to its capability of high expansion. Bentonite clay is rich in montmorillonite (Mollins, 1996).

2.4.1 Expansive clays in Cyprus

The clays of Cyprus are formed as the result of the alternation of sedimentary cycles of the Troodos ophiolite and the pelagic sediments. A large part of Cyprus is covered by expansive clay. The expansive clays found in Cyprus are Bentonitic clays, clay of Mamonia complex, Kythera group clays, Nicosia formation clays, and alluvium clays (Cyprus Geological Survey Department Offices, 2016).

For alluvium clays it was found that they have a liquid limit up to 48% with low to intermediate swelling potential, while for Nicosia formation the swelling potential is considered high to extremely high due to liquid limit of 47 to 73%, Kythrea group, however, have a liquid limit of 47 to 73% with an intermediate to high swelling potential, in Mamonia complex and Bentonitic, liquid limits are varying from 33 to 167% and 55 to 210% respectively, having intermediate to extremely high, and high to extremely high swelling potentials respectively (Cyprus Geological Survey Department Offices, 2016). However, the liquid limit for bentonitic soil taken from west of TRNC was found to be 119% (Iravanian, 2008).

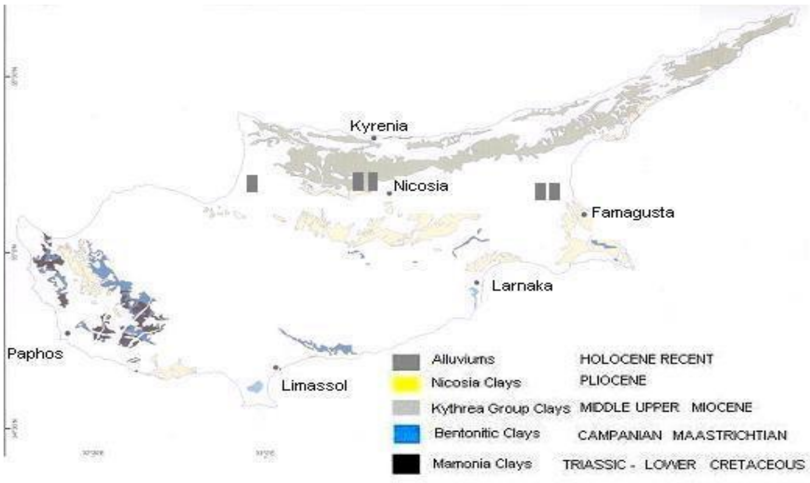

Figure 2.6: Expansive clays of Cyprus (CGS, 2016)

2.5 Factors Influencing Expansive Soils

The factors that affect swelling could be intrinsic factors as clay content, gradation, and pore water chemistry, clay mineralogy or it could be environmental factors as density, stress history, soil structures, temperature, and water content, moreover there are procedural factors like specimen shape and size (Elsharief, Zumrawi, & Salam, 2014). Some of these factors are summarized with the description in Table 2.1 and 2.2 (Nelson & Miller, 1992).

Table 2.1: Factors related to soil properties affecting expansive behavior

Factor	Description
Clay mineralogy	Clay minerals which are the principle of the volumetric change are montmorillonite and vermiculites, other minerals which are illite and kaolinite are rarely expansive especially when the particle size is fine.
Soil water chemistry	The increasing of cation concentration decreases the swelling, these cations repress the swelling.
Soil Suction	Soil suction is a distinct functional stress variable, it performs in the unsaturated soil due to the negative pore pressure. Soil suction is correlating by gravity, saturation, shape and pore size, surface tension, and chemical and electrical characteristics of water and soil particles.
Soil structure and fabric	The expansion potential of dispersed clays is less than that in flocculated clays, connected particles reduce swelling. compaction at high water content tends more dispersed fabric.
Dry density	Higher density affects positively on the spacing between the particles. This density causes generation of repulsive forces between particles and induces larger swell potential.
Plasticity	Soils have a high range of plastic behavior, means high liquid limit that leads to increase in plasticity, which also is considered as an indicator of swell potential.

Table 2.2: Environmental factors affecting expansive soils

Factor	Description
Initial water condition	Expansive soil in its dry status will have a higher suction, furthermore, soil with higher water content have lower suction.
Temperature	Increasing the environment temperature induce water to diffuse beneath buildings and pavements.
Climate	Due to the amount and alternation between precipitation and evaporation effect on the moist seasonal period.
Drainage water sources	Shallow drainage features consider as a source of water at the surface which can feed the soil and pass at greater depth in case of leaky plumbing.
Groundwater	Groundwater tables is considered as a rich moisture source.
Permeability	Soils have high permeability provide a high pass for water and fast swell rate.
Vegetation	The depletion function of plants for moisture in soil induce the soil to be wet
Stress condition	Over-consolidated soils have higher expansion capability than those that are normally consolidated. Swell under light loads had shown unaffectable behavior by aging.

2.6 Soil Stabilization

Soil stabilization term applies to any procedure that can enhance an improvement of soil properties to become reliable to be used in engineering sector as a structural material. The feature of stabilization soil started as a science since 1939. The main aim of soil stabilization is to improve its durability and bearing capacity subjected to many conditions. The stabilization of soil has economic trait. This economical trait makes it possible to be used as construction material. In case of unstable soil, excavation and replacing the soil with another material as crushed rocks, or gravel is considered as uneconomic procedure. Soil stabilization has been used in many applications such as airports runway, roads, buildings, and dams. Soil and clay had been used in early ages as a construction material (Gillott, 1968).

2.6.1 Mechanical stabilization

The process of this stabilization is by mixing the soil with another type to change the gradation. Achieving the desired density, it can be done by using compaction effort. Also, excavating the soil and replacing it with another one that does not have expansive characteristics are obtainable.

2.6.2 Chemical stabilization

In order to improve the properties of expansive soil, a combination of chemical stabilizers such as cement, fly ash, and lime with chloride or individually can be used. About replacing soil particles to meet more stable soil structure, there are two main methods. Firstly, increasing the particle size by cementation to produce an increment in shear strength, reduction in plasticity index, and reduction in expansion potential. Secondly, improve the compaction and physical properties of the soil by using absorption and chemical binding of moisture (Onyelowe & Chibuzor, 2012).

2.7 Types of Additives Used in Soil Stabilization

There are many additives that have been used to improve the engineering properties of expansive soil. These additives can be classified as waste materials such as dust, agricultural wastes, synthetic wastes, and organic wastes to enhance the economic cost.

Table 2.3: Type of additive used for soil stabilization

Industrial solid wastes	Agricultural solid wastes	Domestic solid wastes	Mineral solid wastes
• Fly Ash. • Cement Kiln Ash • Silica Fume • Copper Slag • Granulated Blast Furnace Slag • Phosphogypsum • Ceramic Dust • Brick Dust • Red Mud • Polyvinyl Waste	• Rice Husk Ash • Bagasse Ash • Olive Cake Residue • Wheat Husk • Groundnut Shell Ash	• Incinerator Ash • Waste Tire • Egg Shell Powder • Grain Storage Dust • Glass Cullet	• Quarry Dust • Marble Dust • Baryte Powder • Pyroclastic Dust • Lime Stone Dust • Granite Dust • Mine Tailings

2.8 Quarry Dust

Defined as " the inherent fraction of aggregates passing 0.063mm (63microns)", is described as the production process of quarry dust as the secondary result from processes of blasting, processing, handling, and transportation of aggregates in quarries, and it is figured that the majority if quarry dust is produced during crushing, milling, and screening processes of the quarried rocks. It is illustrated that the annual quarry dust production in the United Kingdom is about 52.6 million tonnes, divided as 20% from limestone, igneous, and metamorphic rock, 10% from sand and gravel, and 25% from sandstones (Mitchell, 2009).

2.8.1 Environmental and health problems due to quarrying activities

Dust is not the only problem produced by quarrying process and dredging operations that affect the environment and human health but there are many other factors that have negative effects as well. Those factors with descriptions are defined in Table 2.4. below (European Bank for reconstruction and development, 2014).

Table 2.4: Quarrying factors affect the environment and human health

Factor	Description
Solid Waste	Solid waste can be produced at any point of quarrying operation. Waste materials should be removed in order to expose the minerals needed. Industrial wastes are included such as workshop scrap, domestic and non-process related to the site.
Habitat Loss and Biodiversity	Quarrying operation can lead to a direct influence on the surrounded habitats: impact can be as alternation the whole area or degradation of the habitats around.
Dust	A large amount of quarry dust is produced by the quarrying operation. This impact of dust effects the communities, workers in quarry and environment. Asthma considered as a common widespread disease in quarries.
Hazardous Materials	The equipment use during quarrying and dredging are heavy and diesel powered. Unwell stored (tanks) for diesel with lubricant, paints, hydraulic oils can affect the environment due to any leaking or explosion.
Community Replacement and Resettlement	Legal titles are lacked in rural communities on their lands, though, their lands had been occupied by them for many years, generation after generation. When a quarry lease is given to contractors, those communities are sometimes forced to evacuate without consulting them or offering them equivalent lands. Environmental damages cause by quarrying such as water pollution and crops contamination will lead to revenue loss for those who will remain in their lands.
Contractors and Migrant Workers	Migrant Workers hired by contractors-sub contractors or agents for quarrying work are vulnerable to discriminatory exploitations.

Table 2.4: Continued

Factor	Description
Noise and Vibrations	Noise and vibrations are caused by mechanical equipment which affects the structures and community surrounding the quarry site.
Community Welfare and Health	An influx increase of migrants is led by quarrying operations, therefore the risk of diseases might increase unless well controlled.
Working Conditions and Labor	Emergency services are usually far from quarry sites, therefore increment in health issues. Foreign workers that are attracted to work in quarry can be exposed to exploitation to work overtime.
Risks of Collision	Accidents occur due to big vehicles collision within the perimeters of the quarry site and high ways.
Explosion and Fire Risk	Risks are very common in many quarry sites due to unorganized storage and misused explosive.
Visual Impacts	Tourism is affected by quarry sites when it is in areas nearby.

2.9 Experimental studies

Shukla (2016) had stabilized black cotton soil by using micro-fine slag. 3, 6, 9, and 12 percent micro-fine slag were used and mixed with soil. The tests carried out were Atterberg limits, free swelling, the California bearing ratio, compaction parameters, and unconfined compressive strength. The micro-fine slag had decreased the plasticity index, liquid limit, and the optimum moisture content. On the other hand, plastic limit, unconfined compressive strength and California bearing ratio of the soil were significantly increased by using 6-7% of micro-fine slag by the weight of the soil and the swell potential was decreased from medium to very low.

Sabat (2012b) had investigated the effect of Polypropylene Fiber on Engineering Properties of Rice Husk Ash, Lime Stabilized Expansive Soil. Tests quarried out are Maximum Dry Density "MDD", Optimum Moisture Content "OMC", Unconfined Compressive Strength "UCS", and California Bearing Ratio "CBR". For UCS it was found that by adding a 10 % of rice husk ash and a 4 % of lime will positively affect the UCS without any addition of lime beyond 4 % because in this case the UCS will be negatively affected due to reacting with silica and alumina. For MDD and OMC polypropylene were added until reaching 2% causing a decrease in MDD due to lower density of polypropylene comparing with soil, while OMC was increased. There was an increase in UCS value without curing time. For soaked CBR it was observed that an increment occurred with/without curing. Also, with addition of fibers soaked CBR was increased.

Lavanya & Jyothi (2017) had stabilized black cotton soil by using fly ash. The percentages that were used are 10, 20, 30, 40, and 50%. Tests that were used in this investigation are specific gravity, Proctor compaction, Atterberg limit, and unconfined compressive strength. Specific gravity for natural soil was decreased when fly ash was added. According to the proctor compaction the Maximum Dry Density "MDD" and the Optimum Moisture Content "OMC" for the soil at its natural status was 1.768g/cc and 22.546% respectively, moreover when fly ash was added to the soil MDD faced increment with addition of all percentage except addition 50% of fly ash the MDD decreased, also for OMC there was variation in its value. Liquid limit, plastic limit with addition of fly ash were increased gradually. On the other hand, plasticity index was decreased. For Unconfined Compressive Strength "UCS" the samples had suspended to 3, 7, 28 days of curing time, further the strength value was increased respectively.

Leite et al., (2016) had the stabilization of expansive soil by using lime. Tests which were used in this investigation are sieve analysis and hydrometer test, Atterberg limits, compaction, free swell, and swell pressure test. Three mixtures were prepared by using 3, 6, and 9% lime addition. For Atterberg limits there was a slight decrement when lime was added also significant decrement was showed in plasticity index with lime addition especially when 3% was added. However, higher addition of lime had no more effect on plastic limit and plasticity index. Moreover, this reduction in plasticity had improved the workability of the soil. Maximum dry unite was decreased in all mixtures and increment occurred in the optimum moisture content. The swell behavior of soil was decreased by lime addition according to free swell test. The preformation of swell pressure test had been done on all samples at curing time of 7 days. The swell pressure had shown significant decrement with lime addition.

Seco et al., (2011) had investigated the stabilization of expansive soil by using of by-products and waste material. Tests were used in this investigation are Standard Proctor Compaction "SPC", Free Swelling "FS", and Unconfined Compressive Strength "UCS". Twelve mixtures were performed using different types of additives which are lime, commercial by-product called PC-7, Natural Gypsum "NG", Rice Husk Fly Ash "RHFA", Cereal Fly Ash "CFA", Coal Bottom Ash "CBA", Steel Flay Ash "SFA", and Aluminate Filler "AF". Twelve mixtures were performed using these additives with different percentages. For maximum dry density and optimum water content showed a variation in their values at all mixtures. Furthermore, the results showed a reduction in swelling behavior for all mixtures, but a mixture with 2% lime + 1%PC-7 showed a significant effect in swelling. All the samples were performed for cure time of 7, 14, 28 days for UCS, the compressive strength was increased for all stabilizers.

Koyuncu et al., (2015) had studied the effect of using ceramic waste to stabilize expansive soil. Crushed Ceramic Dust Waste "CCDW" was added by 40% that affected positively the swelling pressure by 86% and swelling percent by 56%, and by adding Ceramic Tile Dust Waste "CTDW" in 10%, 20%, 30%, and 40% swelling percent and swelling pressure were also positively affected.

Subash et al., (2016) had studied the stabilization of black cotton soil using glass and plastic granules. The tests which were carried out in this investigation are Modified Proctor Compaction "MPC", Unconfined Compressive Strength "UCS", and California Bearing Ratio "CBR". The percentage of glass and plastic that were used are 2%, 4%, 6%, and 8%. In modified Proctor compaction, the maximum dry density and optimum water content were constantly increasing by addition of 6% glass and plastic and then started reducing after using more than 6%. For UCS the strength increased at the optimum glass and plastic percentage which is 6% when the percentage increased beyond 6% the strength started decreasing. CBR was increased using the optimum percentage of glass and plastic then started reducing beyond 6%.

Kulkarni & Patil (2014) had an experimental study of stabilization of black cotton soil by using slag and glass fiber. The tests which were used are Proctor Compaction test, free differential swell and California Bearing Ratio “CBR”. Three preparation mixes were made by using different percentages. The optimum moisture content was decreased, furthermore, the maximum dry density was increased. The CBR values was also increased.

Barman (2017) had studied the effect of “Tire Buffing” and cement on strength behavior of soil-fly ash mixes. The soil that was used is mixed with fly ash of 20, 35, and 50% after curing period of 0, 3, 7, 14, and 28 days. the soil was well compacted at suitable optimum water content and unconfined compression and triaxial compression tests were used. The tire buffing was added to the soil with different percentages of 0, 5, and 10%. When buffing tire was added to the soil-fly ash mixes was found that strength is lower than strength value in soil-fly ash. On the other hand, when fly ash was added to the soil the unconfined compression strength was decreased, furthermore, there was increasing on the peak strength obtained from a triaxial compression test. With addition of 2% of cement to soil-fly ash-tire buffing, the peak strength was increased comparing to soil-fly ash mixes. Also, was observed that tire buffing reduced the stiffness of the soil-fly ash mixes. Mixes contained 35 to 50% of fly ash with 5% of tire buffing and 2% of cement were recommended to be potentially used in the construction of road and dams.

Igwe (2017) had investigated on clay stabilization by using granite and dolerite dusts. The tests which were carried out are standard compaction, California Bearing Ratio "CBR", and Atterberg limit tests. When granite and dolerite dust were added with 20, and 10 % respectively to the soil, a reduction was observed in plasticity by 6.7% and 6.8% respectively. For compaction, the maximum dry density was decreased when 10% of both dusts were added but it was increased with addition 15% of dust. With addition of 15% granite dust, the maximum dry density was pointed 1935 kg/m^3 comparing with addition of 20% dolerite which was recorded 1880 kg/m^3. The lowest optimum moist content was pointed as 14% with addition of 15% of both dusts. For "CBR" with addition of 20% of both dusts enhanced increasing in its value but granite showed better improvement.

Sabat (2012a) had studied on some geotechnical properties lime stabilized expensive soil-quarry dust mixes. The tests which were carried out are Atterberg limits, Unconfined Compressive Strength "UCS", California Bearing Ratio "CBR", modified Proctor test, consolidated undried triaxial compression test, and swelling pressure test. The materials that were used are lime and quarry dust. The quarry dust was used at optimum percentage from 2-7% by soil dry weight. The soil was prepared as a mixture consisting of expansive soil, quarry dust, and lime. When quarry dust was added by 40% the liquid limit, plastic limit and plasticity index were decreased, furthermore, shrinkage limit was increased. However, when lime was added by 7% the optimum moist content was increased, and the maximum dry density was decreased. In addition, reducing the addition of lime to 5% increased the maximum dry unit weight for the mix more than its value at original state which means increasing in strength. The shear strength was procured from consolidated undrained triaxial compression test, the cohesion of the soil was decreased by addition 40% of quarry dust. Furthermore, per contra, the internal friction was increased because quarry dust has very low cohesion value and very high angle of internal friction than soil. When lime of 5% was added to the soil the cohesion increased, but further addition of lime caused a reduction in cohesion, with curing time to 28 days the cohesion again started increasing, as well as the angle of internal friction.

Jayapal (2014) had studied the stabilization of Koratture clay with quarry dust for using in flexible pavement. The tests which were carried out in this study are sieve analysis, Atterberg limit, Modified Proctor Compaction "MPC", California Bearing Ratio, and Differential Free Swell "DFS". The proportion of quarry dust that was used are 0, 10, 20, 30, 40, 50, and 60% of soil weight. For Atterberg limit results showed high decrement in liquid limit and plasticity index up to 50%, at 0% addition of quarry dust liquid limit pointed 55.3% while with addition of 60% of quarry dust pointed 24.2%, on the other hand, plasticity index with 0% addition was pointed 38.9% but with addition 60% quarry dust showed significant decrement of 2.7%. For "MPC" the maximum dry density was increased from 1.87 to 2g/cc at addition of 0 to 60% respectively, furthermore, the optimum moisture content was decreased from 12.7% to 8.75% at addition of 0% to 60% respectively. For "CBR" and "DFS" at 0, 10, 20, 30, 40, 50 and 60 % of addition quarry dust the "CBR" value increased respectively from 4.9% at 0% addition to 22.84% at 60% addition, on the other hand "DFS" decreased from 58.70 at 0% to 17.75% at 60% portion additives.

Jayapal et al., (2014) had studied the stabilization of expansive soil by using three mixtures consisting of quarry dust, fly ash, and lime. The tests that were used are Atterberg limits, Modified Proctor Compaction "MPC", California Bearing Ratio "CBR", and Deferential Free Swelling "DFS". When quarry dust of 10, 20, and 30% were added there was a reduction in plasticity index, on the other hand when fly ash added with the same percentage plasticity index was reduced too. For modified Proctor compaction by adding quarry dust at 10, 20, and 30% the Maximum Dry Density "MDD" was increased and the Optimum Water Content "OWC was decreased. For CBR and DFS when the soil was in its natural status CBR value was 5.29% and DFS was 93.3%, furthermore, when quarry dust was added the value of CBR was increased respectively according to the percentages used and reached 18.55% at 30% addition, on the other hand, DFS were decreased respectively according to the percentage used and reached 35.6% at 30% addition

Amu et al., (2005) had stabilized expansive soil by using eggshell Powder "ESP". Tests were carried out in this investigation are proctor compaction test, California Bearing Ratio Test "CBR" (non-soaked), Unconfined Compression Test, and Undrained Triaxial Test. The Maximum Dry Density reduced from 1508.0 kg m^{-3} with addition just lime to 1473 kg m^{-3} with addition of lime and ESP by 3% and 4% respectively, on the other hand, the Optimum Moisture content increased to 23.8%. For CBR test the value of non-soaked CBR was decreased with adding lime and ESP from 4.8% to 4.1% but with using lime individually CBR increased to 45.5%. For unconfined compression test, shear strength of clay was increased with addition of lime and showed decrement by using lime with ESP. Undrained triaxial shear strength test the value of cohesion was increased by adding lime, furthermore angle of internal friction was reduced. On the other hand, with using lime + ESP the value of cohesion was reduced. Due to these results was observed that lime shows high improvement on stabilization soil than ESP.

Shaka & Shaka (2016) had studied the stabilization of cotton and red soil by using enzyme. Tests were used in this investigation are Atterberg limits, Proctor compaction, Free Swell Index "FSI", and California Bearing Ratio "CBR". Four samples were papered in different dosages of enzyme of 200ml/0.5m^3, 200ml/0.75m^3, and 200ml/1m^3 was used and kept for curing period of 7, 14, and 21 days. The plasticity index for the four samples were 19.57, 37.9, 48.61, and 21.15, on the other hand, the FSI was 75, 81, 90.5, and 25. When stabilizer was added the soil the percentage of 200ml/0.75m^3 showed a significant affect at curing period of 21 days than others. Soaked CBR value was increased in all samples when enzyme was added but at 200ml/0.75m^3 it showed better results.

Mishra & Chandra Babu (2017) had improved the geotechnical properties of red soil by using plastic wares. The percentages of wastes used are 0, 0.25, 0.5, 0.75, 1, and 1.25 % by the soil weight. Test methods that were carried out in this investigation were compaction test, California Bearing Ratio "CBR", and direct shear. In compaction test the optimum water content got increased and reached 16.364 at 0.75% addition of wastes after that started decreasing, furthermore, the maximum dry density increased till 1.99g/cc at 1% addition and started decreasing. CBR was reached to its maximum value with addition of 0.75%, with further increasing of waste there was a decrement in CBR value. For direct shear test, the cohesion and angle of internal friction was increased at addition of 0.25% and kept increasing till reach its maximum value at 1.0% addition. With further increasing with waste material the cohesion and internal friction showed a decrement in its values.

Basha et al., (2005) had investigate the effect of husk ash and cement on the residual soil. The tests carried out in this investigation are compaction, strength, and x-ray diffraction. With addition of 6-8% cement and 10-15% husk ash the maximum dry density was decreased and there was increasing in the optimum moisture. Furthermore, the plasticity of swell was reduced.

Benny et al., (2017) had studied the affection of glass powder on clay soil properties. Tests were carried out in this investigation are Atterberg limits, hydrometer, Standard Proctor Compaction "SPC", Direct Shear Test "DST", Unconfined Compression Test, and California Bearing Ratio. The percentages of class powder that were used are 0, 2, 4, 6, 8, and 10% by the soil weight. For SPC the maximum dry density increased respectively and reached its high value at 6% addition after that started reducing, on the other hand, the optimum water content decreased respectively and reached 28% at 6% addition. The value of CBR reach to 12.8% at addition of 6% of glass powder and started decreasing with further addition. For direct shear box method, the angle of internal friction was increased respectively and scored 37 at 8% addition and kept settle with further addition, furthermore, the cohesion also increased. The value of UCS was increased with the further addition of glass powder and scored 0.071 N/mm^2 at 10% addition.

Ahmed & El Naggar (2016) had studied the effect of recycled bassanite on bentonite clay. The tests that were carried on in this investigation are Atterberg limits, free swell test, Proctor compaction, cation exchange, scan electron microscope SEM, x-ray diffraction, x-ray fluorescences, and unconfined compression. the admixture that was used is lime and cement mixed with bassanite in percentage of 2:1 and then was mixed with natural bentonite at different proportions. It was observed that the mixture of lime-cement with bassanite was more effective on the swelling stabilization more than the bassanite alone. By increasing the admixture, the plasticity, the swell potential, the percentage of sodium ions, the montmorillonite intensity, and the cation exchange capacity of the bentonite was reduced. On the other hand, unit weight and percentage of calcium ions, and the compressive strength were increased for all admixtures used.

CHAPTER 3

MATERIALS AND METHODOLOGY

3.1 Introduction

The purpose of this investigation is to study the effect of quarry dust on the behavior of the expansive soil. The carried out investigation was on four samples collected from different places around TRNC in order to study the effect of quarry dust in soils that have different expansive potential rates. The test methods used were Atterberg limits, Proctor compaction, one-dimensional swell test, and one-dimensional consolidation. The investigation was held in two stages, the first stage is carried out on samples without any quarry dust addition and the second set of tests is with addition of quarry dust with different proportions, the comparison between the results will provide clear vision for the effect of the quarry dust at each proportion.

3.2 Materials

3.2.1 Location of sample 1

The first sample is named T1 and was collected from the road cut south of Taskent village almost 1 km from road cut near to Martyrs remembrance place. The soil was collected in disturbed condition. From the first impression, the soil surface was showing cracks due to the exposure to the atmosphere. T1 was taken from depth about 2.5 meters from the surface after removing the exposed surface soil of 300 mm. The soil that was obtained had a dark brown color. About 20 kg of soil was taken in plastic bags.

3.2.2 Location of sample 2

The second sample was named T2 and collected from the clay pit, north of Haspolat village and located in the northern part of the pit. The soil had a light gray color and it was made of organic mudstone. About 20 kg of soil was collected in plastic bags.

3.2.3 Location of sample 3

Sample 3 was collected from the same pit that sample 2 was taken from. Sample 3 is on the northern flank of the pit. The pit location is behind Cyprus international university. The soil color is dark gray. About 20 kg of soil were taken in plastic bags.

3.2.4 Location of sample 4

The fourth sample is located in the south of Yigitler village. This soil considered as bentonitic and has some gypsum and chalk. About 15-20 kg was taken. The color of this soil is light brown.

Figure 3.1: Location of the studied area

3.2.5 Quarry dust

Quarry dust is a waste material from the quarrying industry as the result of crashing activities and is mainly composed of calcium carbonate $CaCO_3$. However, it is defined as "the inherent fraction of aggregates passing 0.063mm (63microns)", and the production process of quarry dust is described as the secondary result from blasting, processing, handling, and transportation of aggregates in quarries, and it is figured that the majority if quarry dust is produced during crushing, milling, and screening of the quarried rocks (Mitchell, 2009).

It was concluded from our visit to the environmental office of TRNC that there is no monitoring of the amount of quarry dust in the atmosphere, also there is no official reports from the hospitals about the diseases caused by air pollution that affect the people or workers inside the quarry site, however it was concluded after meeting the occupational health and safety expert, Mr. Halil Erdim that respiratory diseases are shown up in late stages almost when the workers get retired (H. Erdim, personal communication, April 17, 2018).

It was concluded from the meeting with the previous chairman of quarry sites association and the owner of RBM company LTD, that there are 14 quarry sites for aggregate production in TRNC and about half of them are near residential areas. Daily production of crushed limestone in TRNC is about 20,000 tons per day, furthermore quarry dust is used only as filler material for asphalt production, therefore there were no measurements to get the amount of quarry dust from crushers. Quarry dust is considered as degradation problem on the environment of the area.

3.3 Methodology

To determine the engineering properties of the soil, some test methods were carried out according to American Society for Testing and Materials (ASTM) as shown in Figure 3.1. The aim of using these methods is not just studying soils properties but also to be able to stabilize it with using quarry dust and studying the effect of quarry dust on these properties.

Table 3.1: Tests used and codes

Test Name	Code Used
Specific Gravity	ASTM D854
Hydrometer	ASTM D422
Atterberg limits	ASTM D4318
Proctor compaction test	ASTM D698
One dimensional consolidation	ASTM D2435
One dimensional swell	ASTM D4546
Unconfined Compressive strength	ASTM D2166

3.3.1 Specific gravity ASTM D854

This test method is applied to the soil fraction that passes from sieve number 4 (4.75mm). The specific gravity test method by means of water pycnometer is not appropriate for those soils containing impurities or high organic material.

This test method is used to determine phase relationship of solids such as the degree of saturation and void ratio. However, this test method is used to determine the specific gravity of soil by using the density of water. Soils containing a water-soluble substance such as chloride, sodium, and soils containing substance has a specific gravity less than one should have special treatment or a good definition of their characteristics. The recommended soil mass according to the pycnometer volume has given in Table 3.2 (American society for testing and materials, 2014b).

Table 3.2: Recommended mass according to pycnometer volume (ASTM)

Soil type	Specimen mass by using 250 ml pycnometer	Specimen mass by using 500ml pycnometer
SP, SP-SM	60 ± 10	100 ± 10
SP-SC, SM, SC	45 ± 10	75 ± 10
SILT OR CLAY	35 ± 5	50 ± 10

About 150 grams of each soil was prepared, which is approximately equal to 600 g from the four samples. The samples were passed from sieve no 4 and also passed from sieve no 200, for fine fraction of soil and it was dried at 60c̊ oven for a minimum of 24 hours in order to drying the initial water content. The pycnometers used for the test were 500 ml for all samples. They were washed and cleaned to make sure there is no impurities remained on it, and they were dried afterward. The weight of all empty pycnometers was taken individually with marks names. About 50g of soil was poured into the pycnometer by using funnels and the weight of the pycnometer with the soil was recorded for all specimens. Distilled water was used in this investigation in order to prevent any effect of impurities in tap water because it may consist of some salt that may affect the properties of soil. Distilled water was poured into the pycnometer till the half of it with a smooth shake to ensure that the slurry is formed.

After insuring that there is no dry part left in the soil all the pycnometers were individually placed into the vacuum for approximately five minutes to extract the air out of the suspension as it shown in Figure 3.1. After getting the pycnometers out of the vacuum distilled water was added again into the pycnometers till the top and weight of it was measured. Finally, the weight of the pycnometers full with distilled water was measured and the specific gravity was calculated using Equation 3.1.

Figure 3.2: Vacuum pump and pycnometer

$$Gs = \frac{M_s}{(M_{p.w.t}(M_{pws,t} - M_s)} \tag{3.1}$$

where

G_s= the specific gravity

M_s= the mass of the soil solids (dry)

M_{pwt} = the mass of the pycnometer with the distilled water

M_{pwst} = the mass of the pycnometer with dry soil with distilled water

Table 3.3: Specific gravity for the collected soil and quarry dust

Soil type	Specific gravity
T1	2.56
T2	2.55
T3	2.55
T4	2.38
Quarry dust	2.72

3.3.2 Hydrometer test ASTM D422

This test method is to determine the distribution of the particle size in the soil. The distribution of the soil that has particles larger than 75µm can be found by using sieves, while those have particles less than 75µm their distribution can be found by using sedimentation process by using hydrometer test method (American society for testing and materials, 2007).

Sodium hexametaphosphate solution should be used in distilled water or demineralized water to prevent any effect of impurities in tap water because it may consist of some salt that may affect the properties of soil. The percentage of hexametaphosphate that should be added is 40g/liter of solution. The water that is used in the test should be brought to the temperature of the room that this test will be carried out in.

Soils were prepared from the four samples that were collected and about 50g from each sample was passed from sieve #200 and the soil considered as fine. The soils were dried in an oven at 60C° for 24 hours to extract the initial water content. Sodium hexametaphosphate was used and about 40g/liter was prepared. The solution with soil was poured into the beaker and mixed together until the slurry was formed. The slurry was poured from the beaker into a cylinder tube with 1000ml volume ensuring that there are no stuck particles remain on the beaker. Distilled water was poured into the mixture until 1000ml and then it was subjected to a shake upside down for approximately one minute after covering the open end by hand palm for avoiding any leaking. The hydrometer that was used in this investigation is H151 and was placed into the tube cylinder, after taking the reading the hydrometer was placed in a tube full of distilled water with a good spin to remove the stuck particles to be used in the second sample, Figure 3.3 is shown the hydrometer test and the graphs that were extracted as the result for all samples. The percentage of sand, silt, and clay in the samples are shown in Table 3.4.

The percentage of the soil in the suspension and the diameter of the particles were determined by Equations 3.2 and 3.3.

(a) soil T1 and T4 (b) soil T2 and T3

Figure 3.3: Hydrometer test for the obtained soils

$$P = [\left(\frac{100000}{W}\right) \times \frac{G}{G-G_1}](R - G_1) \quad (3.2)$$

where

P is the percentage of the soil in suspension.

W is the weight of the dry soil.

R is hydrometer readings with the correction applied.

G is specific gravity of the soil used.

G_1 is specific gravity of the distilled water used.

$$D = K\sqrt{L/T} \quad (3.3)$$

where

K= is the constant that depends on the temperature of the suspension corresponding to the specific gravity of the soils used.

T= time recorded when readings were taken.

L= is the distance between the surface of the suspension and the level at which the specific gravity of the suspension was taken (effective length).

The effective length L and the value of the constant K were taken according to the tables 2 and 3 from ASTM standard D 422-36 (2007).

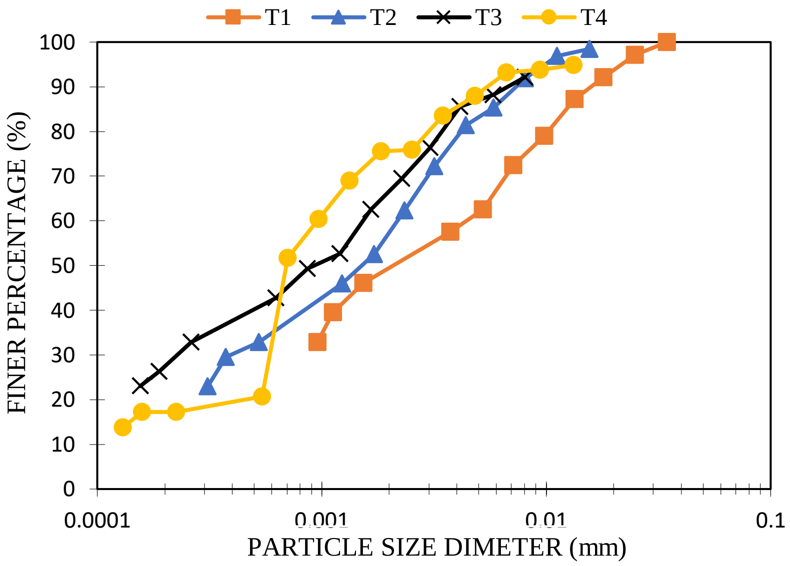

Figure 3.4: Particle size dimeter versus finer percentage for hydrometer test

Table 3.4: Percentage of sand, silt, and clay in soil

Soil	Silt (%)	Clay (%)
T1	52	48
T2	40	60
T3	32.5	67.5
T4	25	75

3.3.3 Atterberg limits ASTM D4318

This test method is done to determine the liquid limit, plastic limit, and the plasticity index of the soil. There are two methods mentioned within ASTM code which is a multipoint test and one-point test (American society for testing and materials, 2010). In this investigation, a multipoint test was used because this method is generally used when precision is required.

Three specimens were prepared, and about 450 grams of each soil and 10%, 20%, and 30% of quarry dust by the weight of the soil were prepared and each proportion was mixed with 150 grams of soil. All specimens were kept in the vacuum for 24 hours after mixing it with water. About 20 grams was extracted from each specimen to be used for plastic limit and the remaining soil was used for the liquid limit. While doing the liquid limit test the number of drops was recorded at approximately 13 mm groove closing. Small specimens were taken from the closing groove and placed in empty cans and kept in the oven for 24 hours at 110± 5 C° to calculate the water content.

The type of soil was determined by using the plasticity chart as shown in Figure 3.5.

Water content % = (mass of water/ mass of dry soil)

Plastic limit (PL) = 100× (mass of water/mass of dry soil)

Plasticity index (PI) = LL-PL

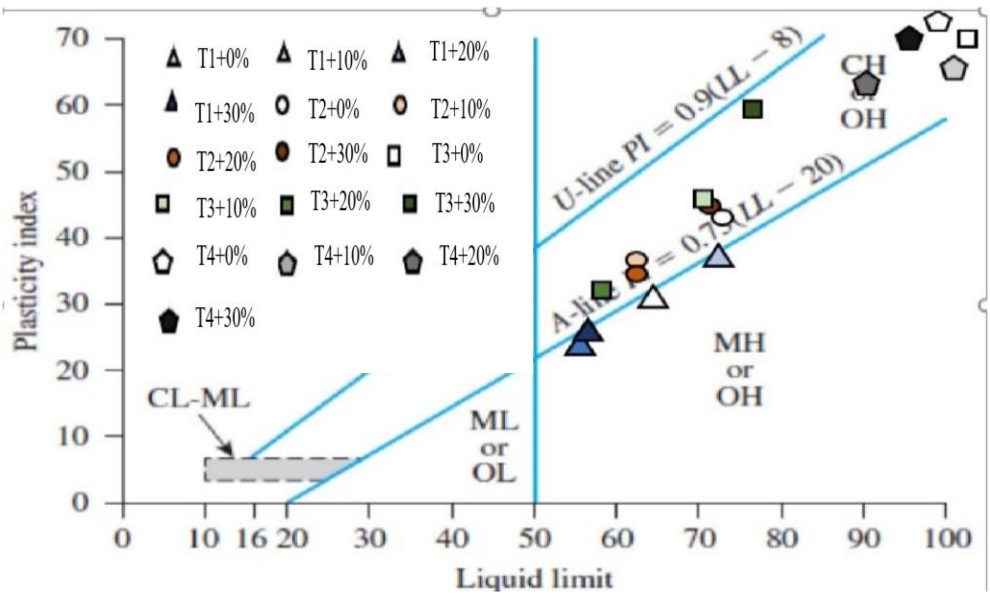

Figure 3.5: Plasticity chart

The expansion index for the obtained soil was determined by using Equation (3.4) (Abbas & Rashid, 2017), in order to determine the expansion potential of the soils according to ASTM D 4829 Table 3.5 and 3.6.

$$EI = 1.8 \times Pl \qquad (3.4)$$

Table 3.5: The expansion potential related to the expansion index according to ASTM

Expansion Index, EI	Expansion potential
0-20	Very low
21-50	Low
51-90	Medium
91-130	High
>130	Very high

Table 3.6: The expansion potential for the obtained soil related to their plasticity index and expansion index

Samples	Plasticity index	Expansion index EI	Expansion potential
T1	31.3	56.34	Medium
T2	43.7	78.66	Medium
T3	82.8	149.04	Very high
T4	92.1	165.78	Very high

3.3.4 Proctor compaction test ASTM D698

This test method is used to determine the relationship between the water content and the dry unit weight of soil by defining the optimum water content and maximum dry density. These determinations are significantly valuable to improve the engineering properties of the soil to be used in road pavements, embankments, and foundation (American society for testing and materials, 2012).

The equipment that was used is mold with a diameter of 152.4 mm and rammer dropped from a height of 30 cm to produce a compaction effort. This test method is applied for soil that is not subjected to previous compaction efforts and was retained on the ¾-in sieve.

In this study about 6-kilograms from each soil were passed from sieve # 4, also were divided into three parts each part about 2-kilograms and was mixed with a deferent proportion of 10%, 20%, and 30% of quarry dust. The initial water content that was added to the soil considered as 10% by the weight of the soil, after the mixing was done all specimens were kept in plastic bags for 24 hours as a curing time to avoid any dry particles. All specimens were put in the mold by three layers each layer subjected to compaction effort of 25 blows by free fall rammer from distance of 12-in. The weight of mold base with soil was recorded; furthermore the weight of specimens that were taken from the compacted soil was recorded twice before and after placing in the oven for 24 hours at 110± 5 C° to measure the optimum water content and the maximum dry density. The equation 3.5 and 3.6 are used to calculate the dry density for the obtained soils.

$$\rho_m = \frac{M_t - M_{md}}{V} \tag{3.5}$$

where

ρ_m= the total density.

M_t= the mass of mold, base plate, and moist soil.

M_{md}= is the mass of mold and base plate.

V= volume of mold.

$$\rho_d = \frac{\rho_m}{1+\frac{w}{100}} \tag{3.6}$$

where

ρd is the dry density of the soil,

ρm is the total density, and w is the water content.

$\gamma d\ (kN/m^3) = 0.00980665 \times \rho d\ (kg/m^3)$

3.3.5 One dimensional swell ASTM D4546

This test method is applied for two purposes, one is wetting-induced swell and the other is the hydrocompression of prepared specimens simulating field conditions of the compacted fills. This test method can be used to define the magnitude of swell pressure or free swell strain (American society for testing and materials, 2014a).

The weight of soil needed was calculated and the proportion of quarry dust of 10, 20, and 30% was added and mixed with the soil at the optimum water content and maximum dry density that were found by the Proctor compaction test. All the specimens were put in the rings of the equipment with a height of 20 mm and diameter of 49.7 mm. The weight of the specimens and the rings were measured.

All the specimens were put in the rings of the equipment and filter paper was used to prevent any wash for small grains. The rings were placed in the odometer. Distilled water was poured in the odometer and the readings were taken in 0.25min, 0.5min, 1min, 2min, 4min, 8min, 15min, 30min, 1hrs, 2hrs, 4hrs, and 24hrs. The free swell was determined by using Equation 3.7.

$$\text{Free Swell} = \frac{\Delta H}{H} \tag{3.7}$$

where

ΔH= Change in initial height (H) of the specimen

H= Initial height of the specimen

3.3.6 One dimensional consolidation ASTM D2435

This test method is applied to determine the magnitude and the rate of consolidation for soil. There are to methods according to ASTM standard. The first method is conducted with a constant load increment for 24 hours. The deformation readings are required to be recorded at a minimum of two increments loads, on the other hand, the time deformation readings in the second method are required to be recorded in all load increments (American society for testing and materials, 2011). Moreover, this test method is providing a compression curve with clear data to account for secondary compression.

The data that is provided from these test methods are used to estimate the rate and magnitude of settlement of structures. The estimation results are considered as important key for designing the engineered structures and the evaluation of their performance. The result of this test method can be negatively affected by the wrong preparation of the specimens due to that reduction of the potential disturbance is required.

Figure 3.6: One-dimensional consolidation equipment

In this investigation, all the specimens that were used for one-dimensional swell were used at the maximum swell and the load increments were applied. The loads that were applied are accumulative as shown in Figure 3.6, started from 1 kg and till 64 kg loading. The time-deformation readings were recorded at each load increment for 0.25, 0.5, 1, 2, 4, 8, 15, 30, 60, 120, and 240 minutes, after that at 24 hours individually according to the second method. The rebound was done on two stages, the first one 32kg was lifted and the readings were recorded after 24 hours after that the second stage started with lifting the remained loads and the readings recorded after 24 hours too. After the rebound completed the specimens were taken out from the odometer and the weight of them was recorded before and after placing in the oven for 24 hours at 110± 5 C^{o} to measure the final water content. In order to determine the void ratio, the Equations 3.8 to 3.12 were used.

$$H_s = \frac{M_s}{G_s \times A \times \rho_w} \tag{3.8}$$

$$H_v = H - H_s \tag{3.9}$$

$$\mathrm{e_o} = \frac{\mathrm{V_v}}{\mathrm{V_s}} = \frac{\mathrm{H_v}}{\mathrm{H_s}} \tag{3.10}$$

$$\Delta e_1 = \frac{\Delta H_1}{H_s} \tag{3.11}$$

$$e_1 = e_o - \Delta e_1 \tag{3.12}$$

where

H_s= height of solid

H_v= height of voids

ΔH_1= change in height

M_s= mass of solid

G_s= specific gravity of the soil

A= area of the specimen

ρ_w= density of water

e_0= the initial void ratio

e_1= the void ratio

3.3.7 Unconfined compressive strength ASTM D2166

This test method was used to determine the unconfined compressive strength for clay soil and it provides an approximate value of the strength of cohesive soil (American society for testing and materials, 2013). The test method was done according to the ASTM code and just applicable for cohesive soil which will not expel or bleed water.

This test method was done on four types of soil, having cohesive characteristics with and without any addition of quarry dust. About 12 specimens' cylinders were prepared from each soil. The specimens were having 7.6 cm length and 3.8 cm diameter. The specimens were placed individually in the unconfined strength device. The upper platen placed to touch the specimen and the deformation indicator was adjusted on zero. Load, deformation, and time values were recorded up to 15 points to define the stress-strain curve. The weight of the specimens was taken before and after the test and then was put in the oven to determine the water content. Two methods are applicable in this case either trimming or the whole sample. The axial strain was calculated by recording the change of length over the initial length 3.13.

$$\varepsilon = \frac{\Delta L}{L_o} \tag{3.13}$$

where

ε = the axial strain.

ΔL = the change in length (deformation).

L = the initial length of the specimen before the test.

The average cross-sectional area was calculated by Equation 3.14.

$$A = \frac{A_o}{(1-\left(\frac{\varepsilon_1}{100}\right))} \tag{3.14}$$

where

A = corresponding average cross-section area.

A_o = the initial average cross-section area of the specimen.

ε_1 = axial strain for the given load by percent.

The compressive strength was calculated by Equation 3.15.

$$\sigma_c = \frac{P}{A} \tag{3.15}$$

where

σ_c = the compressive strength.

P = applied load.

CHAPTER 4

RESULT AND DISCUSSION

Study of cohesive soil has a major problem, which is volumetric instability. Many methods of volumetric stabilizing for such types of soils were adopted. In this study quarry dust as an additive for volumetric stabilizing was studied, different adding proportions of 10, 20, and 30% by weight of soil were adopted. This study was applied on four different types of soils with an expansive potential. Results of tests applied on soil types with different percentages of quarry dust will be shown and discussed.

4.1 Specific Gravity

Test was applied on all soil samples according to ASTM D854. For all of the soil samples with/without adding proportions of quarry dust with specific gravity of 2.72. It was observed that the specific gravity increased respectively with the increment of the additions in all samples as shown in Table 4.1. The specific gravity of the mixture of soil and quarry dust was determined by using the Equation 4.1 (Iravanian, 2008).

$$\mathrm{Gs} = \frac{100}{\frac{\mathrm{SOIL}\,\%}{G_S\,\mathrm{SOIL}} + \frac{\mathrm{QUARRY\ DUST}\%}{G_S\,\mathrm{QUARRY}}} \tag{4.1}$$

Table 4.1: Specific gravity result for the obtained soils with different proportions of quarry dust

GS	T1	T2	T3	T4	QD
Soil+0QD	2.55	2.56	2.55	2.38	2.72
Soil+10%QD	2.564	2.571	2.568	2.408	2.72
Soil+20%QD	2.580	2.586	2.583	2.439	2.72
Soil+30%QD	2.596	2.601	2.599	2.470	2.72

By observing Table 4.1, it can be noticed that for T1 by adding a 10% by weight of soil specific gravity increased from 2.55 to 2.564, while by adding a 20% and 30% by weight of soil result showed an increasing to 2.58 and 2.596 respectively. For sample T2 by adding a 10% by weight of soil specific gravity increased from 2.56 to 2.571, while by adding a 20% and 30% by weight of soil result showed an increase to 2.586 and 2.601 respectively. Sample T3 also faced increment in specific gravity however by adding a 10% by weight of soil specific gravity increased from 2.55 to 2.568, while by adding a 20% and 30% by weight of soil result showed an increasing to 2.583 and 2.599 respectively. Testing sample T4 showed that by addition of a 10% by weight of soil specific gravity increased from 2.34 to 2.408, while by adding a 20% and 30% of quarry dust result showed an increasing in specific gravity to 2.439 and 2.47 respectively. Tests showed that addition variety of proportions of quarry dust increased specific gravity of samples.

4.2 Atterberg Limits

For Atterberg limits, test ASTM D4318 was adopted, results are illustrated for all sample with/without using quarry dust in Table 4.2

Table 4.2: Atterberg limits results for the obtained soils with different percentages of quarry dust

Atterberg limits	**Proportions of quarry dust**	**T1**	**T2**	**T3**	**T4**
		MH	**CH**	**CH**	**CH**
LL	0%	63.9	71.9	114.7	132.4
PL		32.6	28.2	31.8	40.3
PI		31.3	43.7	82.8	92.1
		MH	**CH**	**CH**	**CH**
LL	10%	71.9	62.5	77.7	101.1
PL		34.2	29.0	26.8	34.8
PI		37.7	33.5	50.9	66.3
		MH	**CH**	**CH**	**CH**
LL	20%	55.6	61.9	58.8	90.3
PL		31.9	27.4	25.4	27.2
PI		23.7	34.5	33.4	63.1

Table 4.2: Continued

Atterberg limits	Proportions of quarry dust	T1	T2	T3	T4
		MH	**CH**	**CH**	**CH**
LL		57.3	71.2	71.2	96.8
PL	30%	30.7	26.2	26.0	27.3
PI		26.6	45.0	45.2	69.5

From Table 4.2 it was observed that without addition of quarry dust the soils were categorized as silt with high plasticity and clay with high plasticity according to the Unified Soil Classification System (USCS), with addition of 10, 20, and 30% quarry dust the liquid limit, plastic limit, and plasticity index decreased over all, but the classification of the soil stayed in the same category.

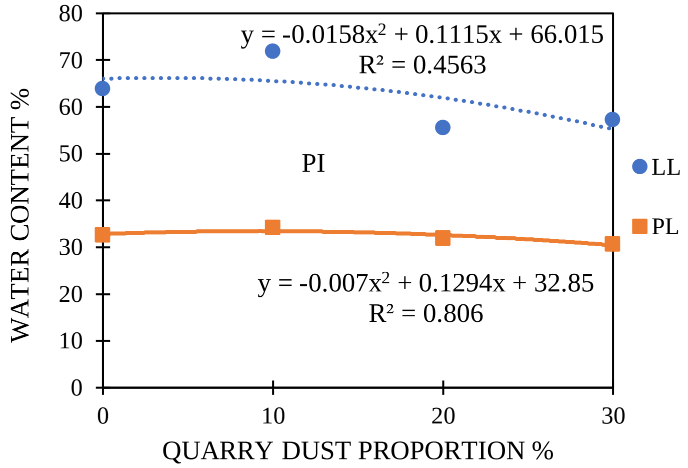

Figure 4.1: Liquid limit, plastic limit, and plasticity index for sample T1

Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of sample T1 was 63.9, 32.6, and 31.3% respectively, and by addition of 10% Q.D. the LL, PL, and PI were increased to 71.9, 34.2, and 37.7% respectively, while addition of 20% Q.D. decreased the LL and PL slightly to 55.6 and 31.9 %respectively as shown in Figure 4.1 and the PI was decreased to

23.7%, however, LL and PI faced an increment after adding of 30% by weight of soil to 57.3 and 26.6%, on the other hand there was a slight decrease in PL to 30.7%. According to (Jayapal, 2014) the plasticity index was decreased gradually with increasing the proportion of the quarry dust till 60%.

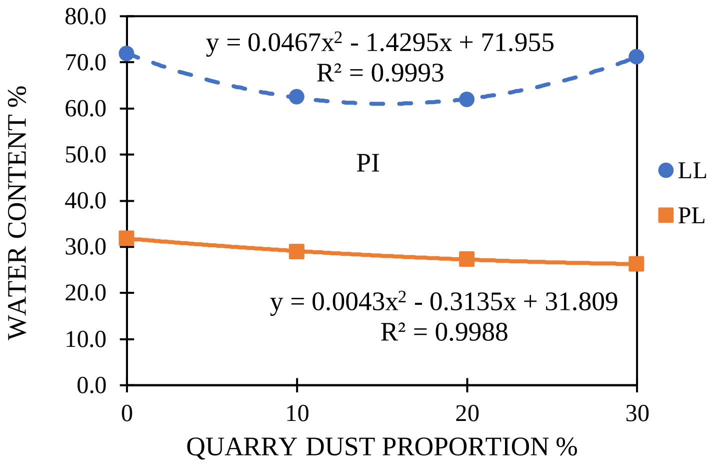

Figure 4.2: Liquid limit, plastic limit, and plasticity index for sample T2

For sample T2 as shown in Figure 4.2 the LL, PL, and PI were 17.9, 28.2, and 43.7 %, while by adding 10% of quarry dust both LL and PI reduced to 62.5 and 33.5% respectively and there was a slightly increasing in PL to 29%, on the other hand, addition of 20% quarry dust, LL, PL, and PI were decreased to 61.9, 27.4, and 34% respectively but in case of addition of 30% quarry dust, LL and PI faced an increment to 71.2 and 45% respectively while PL reduced to 26.2%.

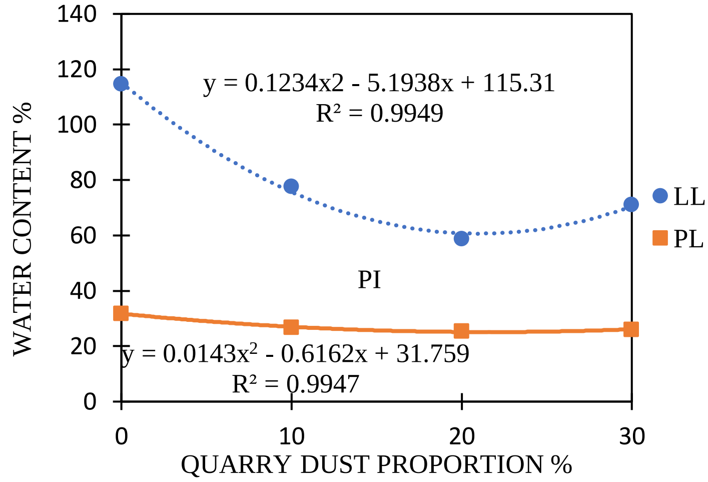

Figure 4.3: Liquid limit, plastic limit, and plasticity index for sample T3

For sample T3 the LL, PL, and PI without any addition of quarry dust were recorded as 114.7, 31.8, and 82.8% respectively, with adding 10, 20, and 30% by weight of soil the LL, PL, and PI were significantly reduced, in case of 10% addition the LL, PL, and PI were 77.7, 26.8, and 50.9% respectively. However, as shown in Figure 4.3 the lowest decrement can be observed with addition of 20% quarry which was recorded as 58.8, 25.4, and 33.4% for LL, PL, and PI respectively, also with addition of 30% of quarry dust they were recorded as 71.2, 26, and 45% for LL, PL, and PI respectively.

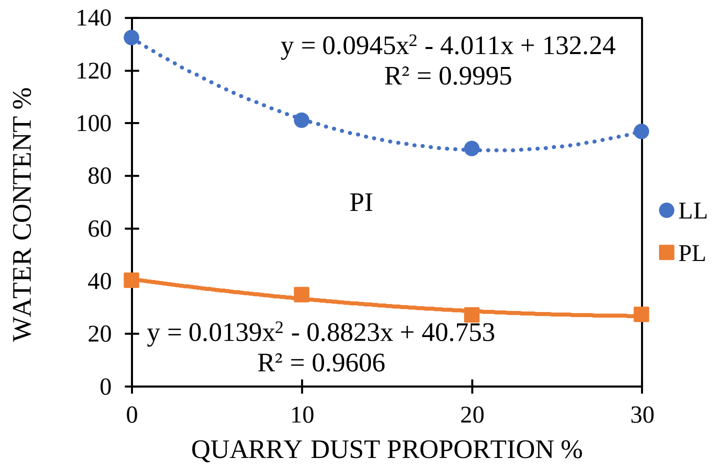

Figure 4.4: Liquid limit, plastic limit, and plasticity index for sample T4

In sample T4 the LL, PL, and PI without any addition of quarry dust were 132.4, 40.3, and 92.1% respectively, however as shown in Figure 4.4 with addition of 10, 20, and 30% of quarry dust LL decreased to 101.1, 90.3, and 96.8% respectively, moreover the PL also decreased to 34.8, 27.2, and 27.3% respectively, also PI decreased from 92.1 to 66.3, 63.1, and 69.5% at addition of 10%, 20%, and 30% respectively. The lowest decrement was at addition of 20% of quarry dust.

4.3 Standard Proctor Compaction

This test method was used to determine the relationship between the molding water content and dry density of the soil. By using the standard effort, the maximum dry density “MDD” and the optimum moisture content “OMC” can be determined from the dry density versus moisture content curves as shown in Figures 4.7, 4.10, 4.31, and 4.17. The results were compared between soil with different percentages of quarry dust in order to measure the change in the dry density and moisture content at different proportions as shown in Table 4.3. This test method was done according to the ASTM D 698.

Table 4.3: Results of standard Proctor compaction for the obtained soils

Soil	T1		T2		T3		T4	
	MDD (g/cm^3)	OMC (%)	MDD (g/cm^3)	OMC (%)	MDD (g/cm^3)	OMC (%)	MDD (g/cm^3)	OMC (%)
SOIL+0%QD	1.6	21.5	1.62	22	1.67	19	1.3	39.4
SOIL+10%QD	1.63	21	1.66	21.5	1.74	18	1.37	28.8
SOIL+20%QD	1.7	20.2	1.7	18.3	1.75	18.5	1.4	26.5
SOIL+30%QD	1.72	18.5	1.76	17.5	1.76	18.5	1.44	27.5

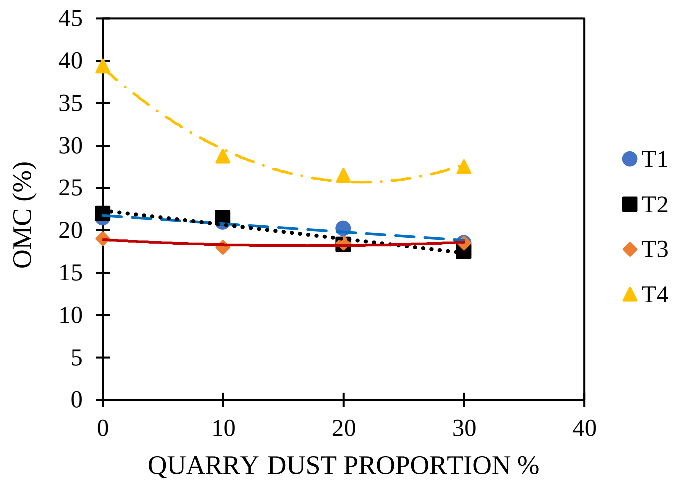

Figure 4.5: Optimum moisture content versus quarry dust proportion for the obtained soil

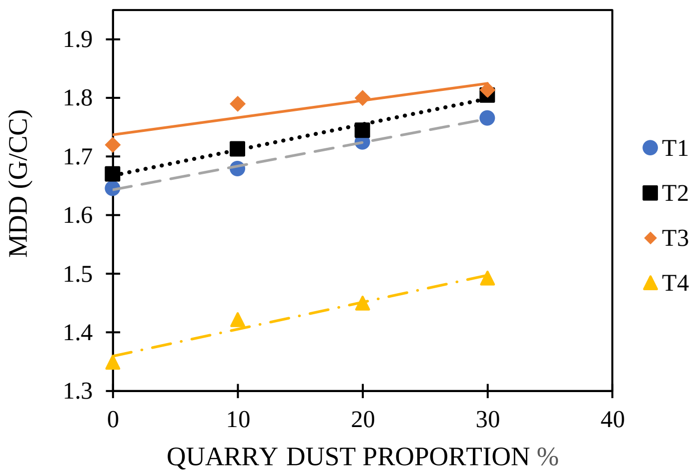

Figure 4.6: Maximum dry density versus quarry dust proportion for the obtained soil

Table 4.4: Hyperbolic constant for the obtained soil

	optimum moisture content			maximum dry density		
soil	**a**	**b**	**R^2**	**a**	**b**	**R^2**
T1	21.75	-0.0975	0.9298	1.5934	0.004	0.9967
T2	22.33	-0.167	0.9133	1.6177	0.0044	0.9847
T3	18.9	-0.085	0.6	1.6872	0.0029	0.8117
T4	39.15	-1.25	0.9883	1.3095	0.0046	0.9631

It was observed that addition of quarry dust has enhanced the clay soil. Therefore, the optimum moisture content was decreased along with decrease in the plasticity index PI, on the other hand, there was an increment in the maximum dry density this increasing it could be due to the high specific gravity of the quarry dust. Furthermore, the enhancement of quarry dust was boldly observed in soil T4, with the increasing of the quarry dust. There was an increasing in the dry density and decreasing the optimum moisture content as shown in Figure 4.5, and 4.6, as the plasticity index PI decreased; the water absorption capacity decreased as well. This enhancement could be due to the replacing of the clay minerals by quarry dust grains which are different from clays in water absorption capabilities.

Table 4.4 shows that for soil type T1 optimum moisture content for all proportions yielded a good and acceptable R^2 value, while for maximum dry density the best fitting yielded better value than the optimum moisture content fitting. For soil type T2 the best fitting optimum moisture content showed lower R^2 value by approximately 2% and this is the same reduction value in case of maximum dry density. Soil type T3 showed high reduction in fitting value, on the other hand a reduction also was observed for maximum dry density but the fitting was not as low as the case of maximum moisture content. For soil type T4 fitting for both maximum dry density and maximum moisture content showed comparable values with T1 and T2 with reduction of 4% and 3% respectively. These variations could be due to the variation in minerology among various types.

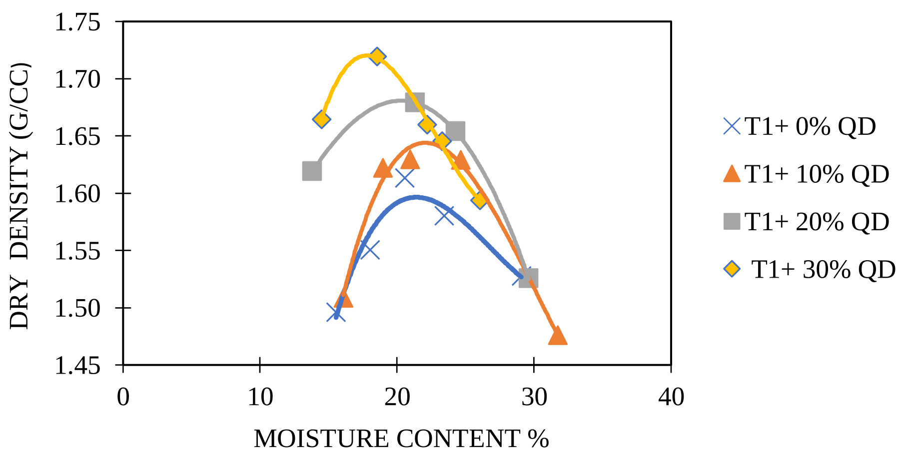

Figure 4.7: Compaction curves for soil T1 with different percentages of quarry dust

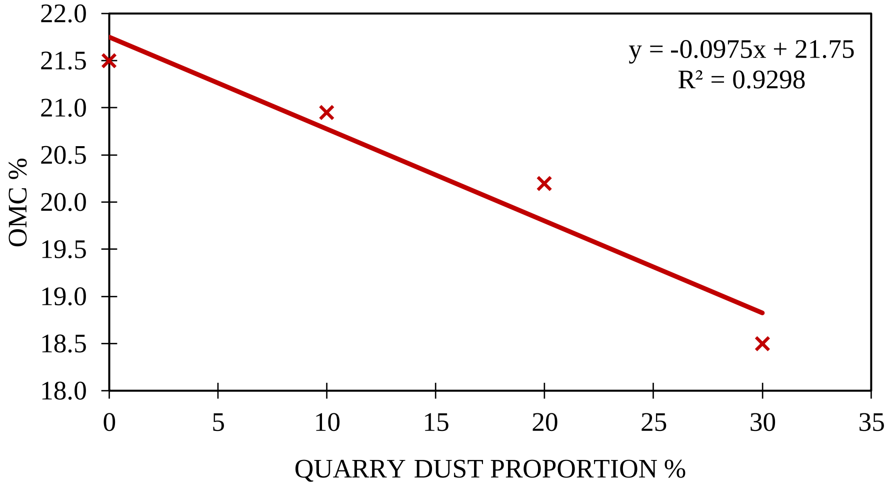

Figure 4.8: The optimum moisture content for soil T1 with different percentages of quarry dust

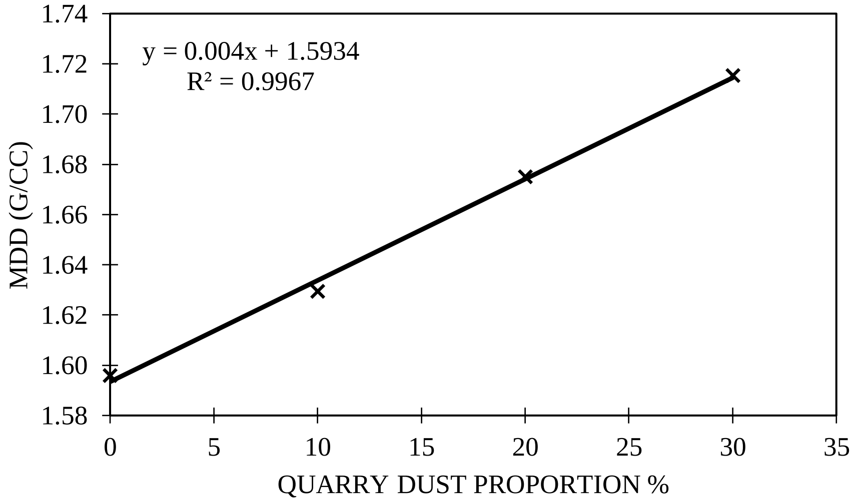

Figure 4.9: The maximum dry density for soil T1 with different percentages of quarry dust

The test method was done on four soils T1, T2, T3, and T4 all are characterized as expansive potential. In soil T1 the maximum dry density and the optimum moisture content were 1.6 g/cm^3 and 21% respectively without any addition of quarry dust. However, with addition of 10, 20, and 30% quarry dust the maximum dry density showed an increment at all proportions to 1.63, 1.7, and 1.72 g/cm^3 respectively as shown in Figure 4.9. On the other hand, the optimum moisture content decreased to 21, 20.2, and 18.5% respectively as shown in Figure 4.8. With increasing the percentage of quarry dust from 10% to 30% there was a decrement in the optimum moisture content as well as an increment in the maximum dry density (Jayapal et al., 2014).

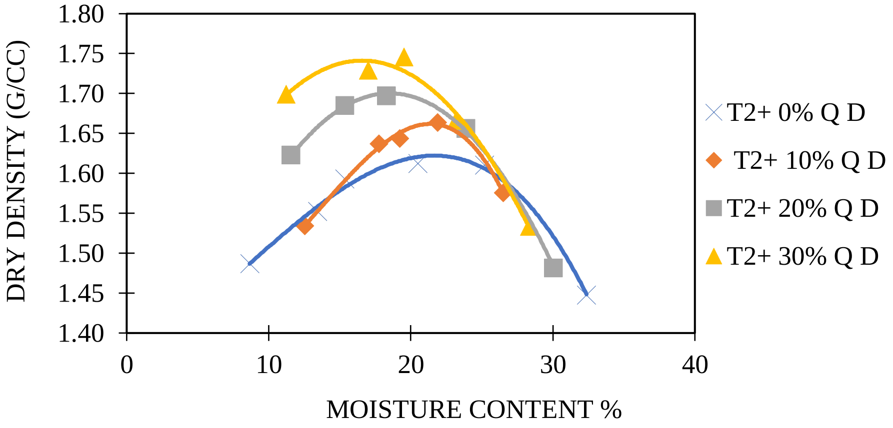

Figure 4.10: Compaction curves for soil T2 with different percentages of quarry dust

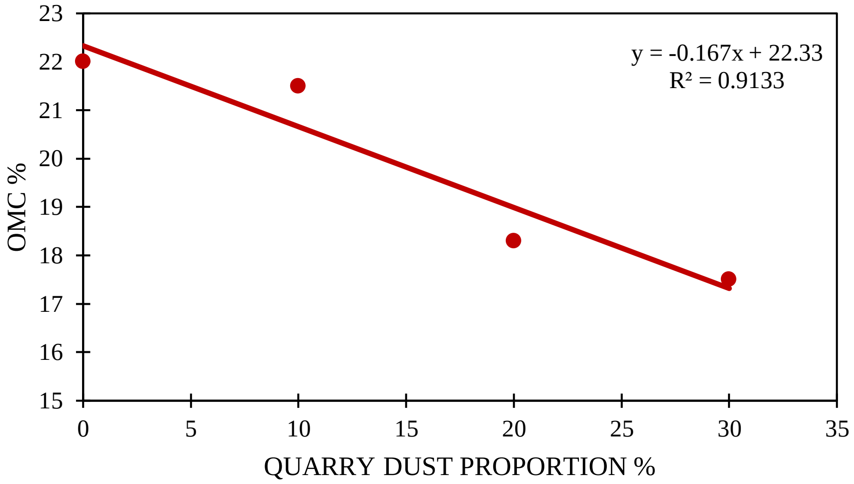

Figure 4.11: The optimum moisture content for soil T2 with different percentages of quarry dust

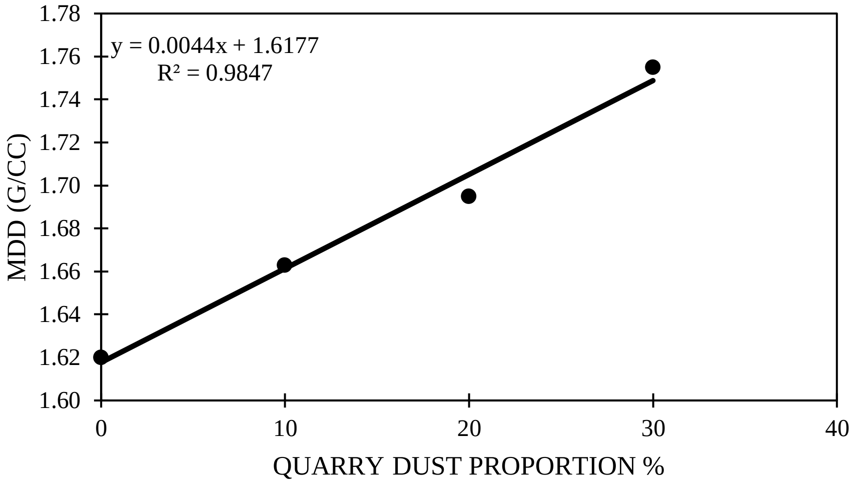

Figure 4.12: The maximum dry density for soil T2 with different percentages of quarry dust

For soil T2 the maximum dry density and the optimum moisture content at 0% addition of quarry dust were 1.62 g/cm^3 and 22% respectively. In addition of 10, 20, and 30% by weight of soil, the maximum dry density was increased at all proportion respectively to 1.66, 1.7, and 1.76 g/cm^3 as shown in Figure 4.12. In contrast, the optimum moisture content was decreased to 21.5, 18.3, and 17.5 respectively as shown in Figure 4.11.

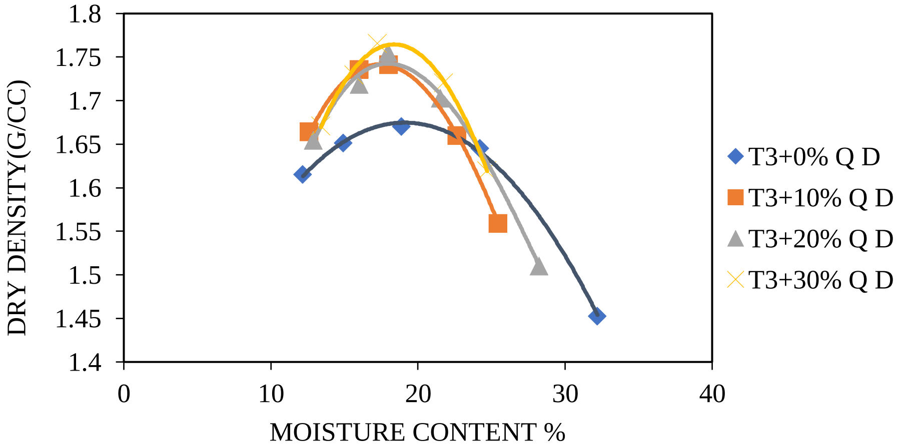

Figure 4.13: Compaction curves for soil T3 with different percentages of quarry dust

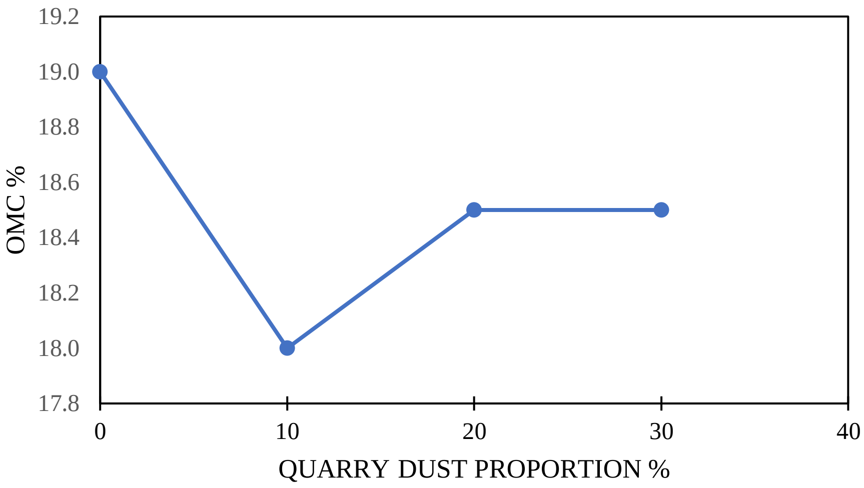

Figure 4.14: The optimum moisture content for soil T2 with different percentages of quarry dust

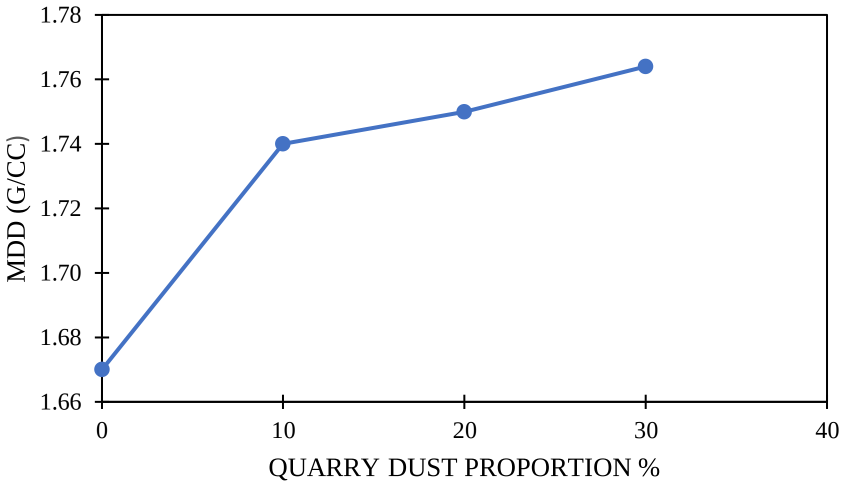

Figure 4.15: The maximum dry density for soil T3 with different percentages of quarry dust

In soil T3 the maximum dry density was increased from 1.67 g/cm^3 at 0% addition to 1.74, 1.75, and 1.76 g/cm^3 at 10, 20, and 30% addition of quarry dust as shown in Figure 4.15, but the optimum moisture content was decreased from 19% at 0% addition of quarry dust to 18% at 10% addition of quarry dust, however, it slightly increased again and settled in addition of 20 and 30 % of quarry dust to 18.5% as shown in Figure 4.14. The sharp decrease in optimum moisture content could be because of replacement of water absorbent clay minerals by quarry dust material. However, as the percent QD increases the impact of it on OMC reduces, that could be probably related to the packing of the mixed material. On the other hand, the increase in MDD continues with increase of QD percentages, though after 10% this increase shows a slighter slope.

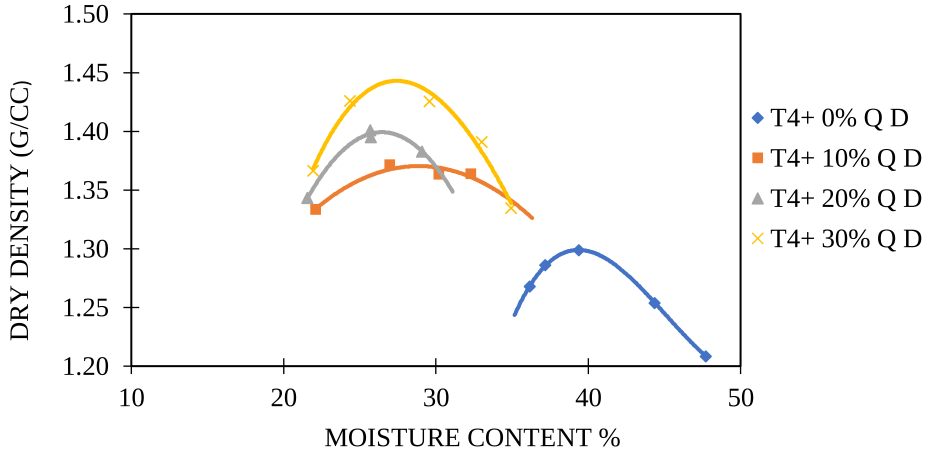

Figure 4.16: Compaction curves for soil T4 with different percentages of quarry dust

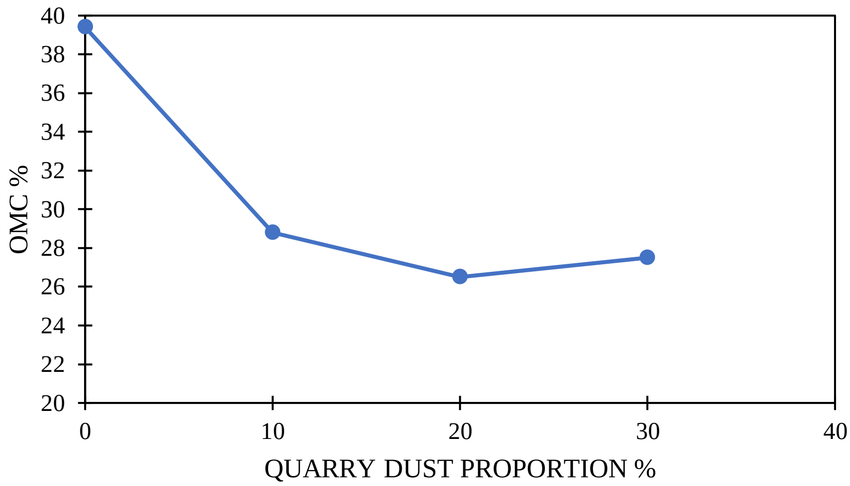

Figure 4.17: The optimum moisture content for soil T4 with different percentages of quarry dust

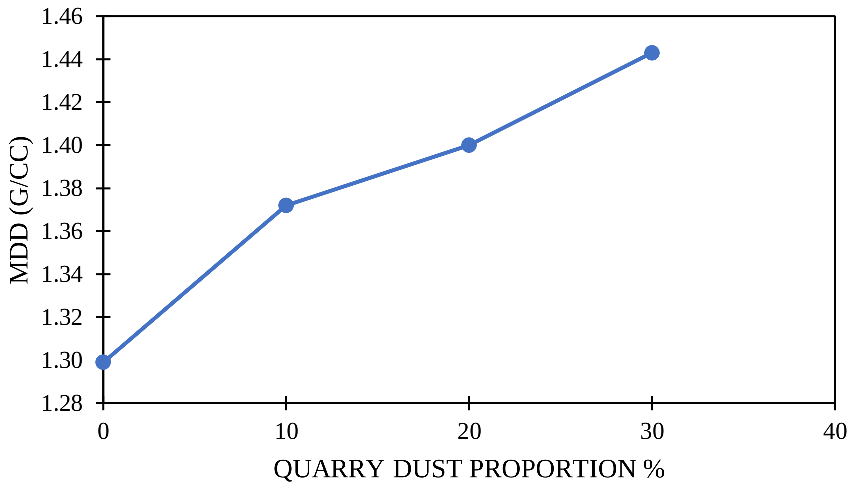

Figure 4.18: The maximum dry density for soil T4 with different percentages of quarry dust

In case of soil T4 the maximum dry density increased from 1.3 g/cm^3 at addition of 0% of quarry dust to 1.37, 1.4, and 1.44 g/cm^3 at addition of 10, 20, and 30% of quarry dust respectively as shown in Figure 4.18. Furthermore, there was a significant decreasing in the optimum moisture content as shown in Figure 4.17 from 39.4% at 0% quarry dust to 28.8, 26.5, and 27.5% at addition of 10, 20, and 30% of quarry dust respectively. It was clearly observed that all proportions were influenced positively on the maximum dry density and the optimum moisture content in all soils, yet, the changes in addition of 10% QD are the most visible in compare to 0% QD.

4.4 One Dimensional Swell

This test method was adopted according to ASTM D4546 for understanding and studying the swelling behavior of the expansive soil. The test method was applied on four soils with different proportions of quarry dust. From the swelling curve that was formed from the swell strain versus time, the initial swell, primary swell, and the maximum swell were extracted for each soil as shown in Table 4.5. The initial swell is considered as the first swell that graphs showed and it takes less time to form compared with the primary swell which presents long time to complete the formation (as it will be seen in Figure 4.19, 4.20, 4.21, and 4.22).

For soil T1 the maximum swell was scored 3.8% at 0% addition of quarry dust, however, with addition of quarry dust the expansion behavior of this soil was reduced respectively at 10, 20, and 30% 0f quarry dust addition and scored 2.5, 2.15, and 1.7% respectively. Furthermore, the initial and primary swell reduced respectively as well. According to the graph that is shown in Figure 4.19, it could be concluded that the addition of QD results in reducing swell quantities and the minimum swell values was recorded for 30% addition of quarry dust.

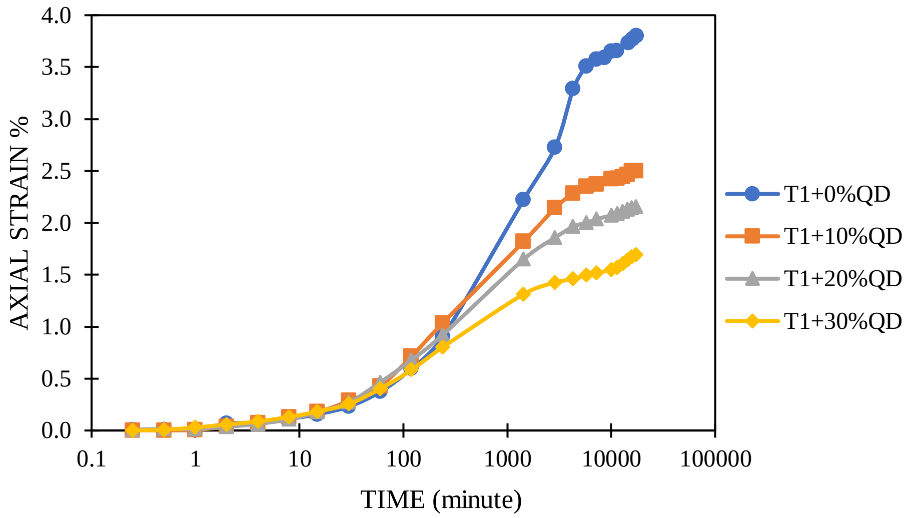

Figure 4.19: Axial strain versus time graph for T1 with different percentages of quarry dust

In the case of soil T2 total swell of 11.39% at 0% addition of quarry dust was recorded. The effect of quarry dust in order to reduce the swell behavior at 10% addition shows a slight decrement to 10.66% of total swell. Also, with addition of 20, and 30% of quarry dust the swell was well reduced as shown in Figure 4.20, but the effectiveness of adding these proportions was quite same having 6.43 and 6.35% of total swell for both proportions. Furthermore, the result shows respective reduction of initial and primary swell with addition of quarry dust. Therefore, it could be concluded that the addition of 20 and 30% of quarry dust to T2 sample results in almost identical trend in swell behavior.

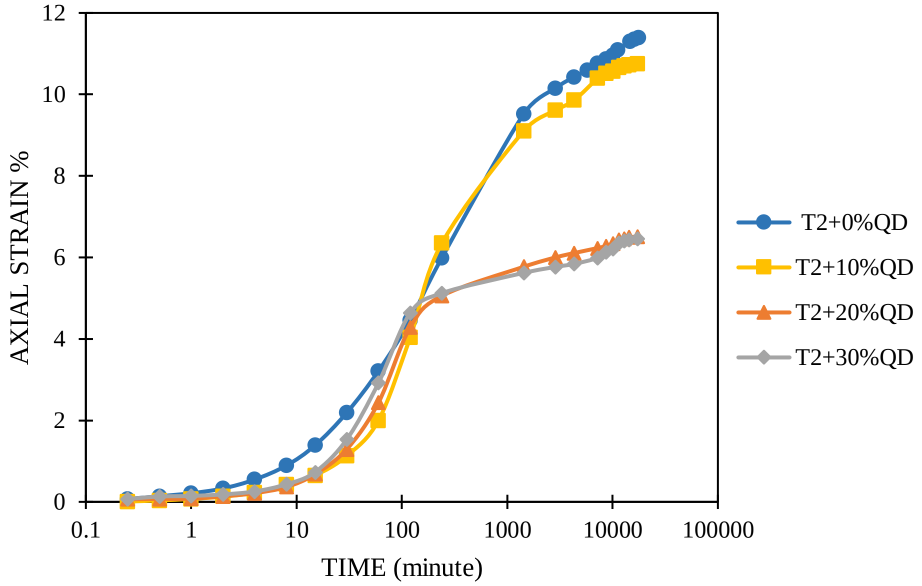

Figure 4.20: Axial strain versus time graph for T2 with different percentages of quarry dust

There was a respective reduction in the swell behavior for soil T3 with addition of 10, 20, 30% of quarry dust as shown in Figure 4.2. The total swell at 0% addition was 13.65% but with addition of 10, 20, and 30 % of quarry dust the swell was reduced to 9.1, 6.7, and 4.45% respectively. Also, the initial and primary swell was reduced gradually with addition of all proportions. It could be considered that addition of 30% of quarry dust is the effective percentage for this type of soil.

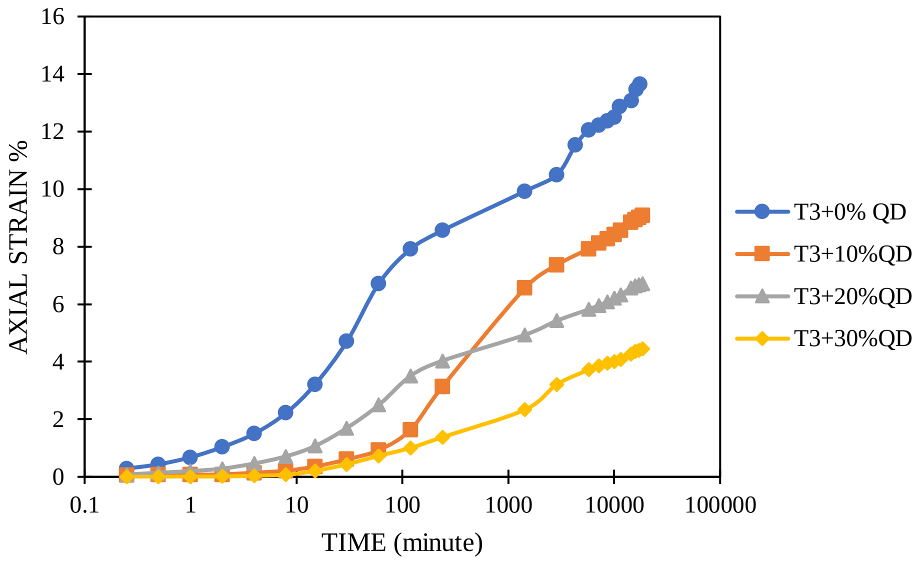

Figure 4.21: Axial strain versus time for soil T3 with different percentage of quarry dust

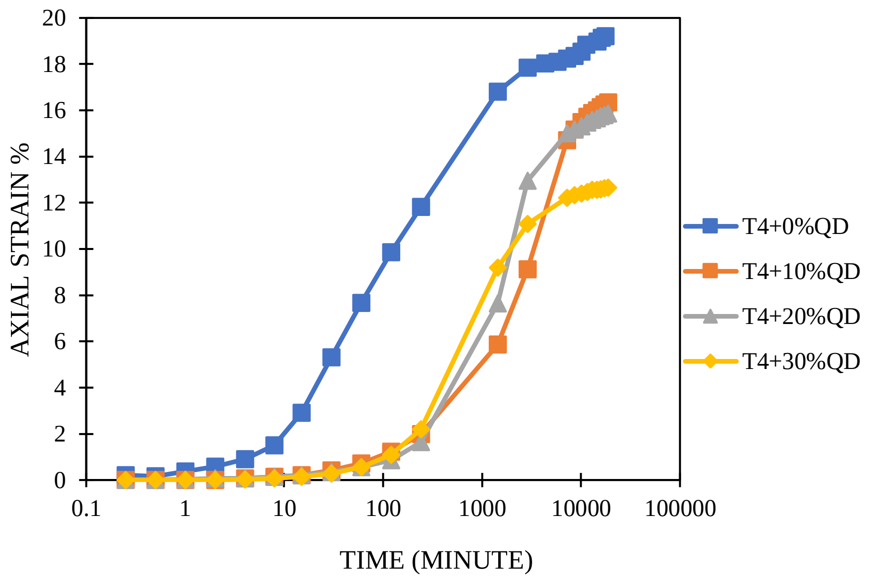

Figure 4.22: Axial strain versus time for soil T4 with different percentage of quarry dust

The maximum swell for soil T4 at 0% addition was 19.2%, however with addition of 10, 20, and 30% of quarry dust the swell decreased gradually to 16.34, 15.66, and 12.66% respectively. It could be concluded that addition of quarry dust to this soil results in minimizing the swell potential, yet the total swell of 10 and 20% QD mixtures illustrate quite parallel outcomes as shown in Figure 4.22.

Table 4.5: Result of one-dimensional swell

parameters	proportion	T1	T2	T3	T4
initial swell %	0% QD	0.5	1.6	2.2	1.4
initial swell time (min)		90	18.5	7.8	8
primary swell %		3.45	9.53	7.49	17.4
primary swell time (min)		5200	1440	91	1500
total swell %		3.8	11.39	13.66	19.2
total swell time (min)		17640	17640	17640	17640
initial swell %	10% QD	0.3	1.5	1.1	1.22
initial swell time (min)		32	42	60.8	120
primary swell %		2.2	6.9	6.56	15.6
primary swell time (min)		3500	310	1440	7000
total swell %		2.5	10.66	9.1	16.34
total swell time (min)		17280	11520	18720	18720
initial swell %	20% QD	0.28	1.19	0.9	1.1
initial swell time (min)		30	27	10.5	125
primary swell %		1.84	4.72	3.7	13.4
primary swell time (min)		2900	150	140	3100
total swell %		2.15	6.43	6.71	15.86
total swell time (min)		17280	11520	18720	18720
initial swell %	30% QD	0.24	1.11	0.26	1
initial swell time (min)		25	21.5	16	120
primary swell %		1.36	4.81	3.31	11.2
primary swell time (min)		1800	200	3200	3000
total swell %		1.7	6.35	4.45	12.66
total swell time (min)		17280	11520	18720	18720

Table 4.6: Expansive soli classification based on liquid limit, plasticity Index and in situ suction (Nelson & Miller, 1992)

LL (%)	**PI (%)**	**μ_{nat}^{*}, tsf**	**Potential swell (%)**	**Potential swell classification**
> 60	> 35	> 4	> 1.5	High
50-60	25-35	1.5-4	0.5-1.5	Marginal
< 50	< 25	< 1.5	< 0.5	Low

Table 4.7: Expansive potential classification for the obtained soils

Soil	Proportion	Potential swell (%)	Potential swell classification
T1	0% QD	3.8	High
	10% QD	2.5	High
	20% QD	2.15	High
	30% QD	1.7	High
T2	0% QD	11.39	High
	10% QD	10.66	High
	20% QD	10.43	High
	30% QD	6.35	High
T3	0% QD	13.66	High
	10% QD	9.1	High
	20% QD	6.71	High
	30% QD	4.45	High
T4	0% QD	19.2	High
	10% QD	16.34	High
	20% QD	15.86	High
	30% QD	12.66	High

It was observed that with addition of quarry dust till 30%, the potential swell reduced for the obtained soils but their expansion potential still high according to the classification shown in Table 4.6 and 4.7.

4.4.1 Absorption water during swell

The absorbed water during swell can be determined by using a simple equation in order to determine the percentage of water that was actually absorbed during the test with addition of different proportions of quarry dust.

As it is known, montmorillonite is responsible for the volumetric change due to its capability of water absorption, moreover, the volume of montmorillonite particles in expansive soil increases by the absorption water into the interlayers. (Setyo Muntohar & Hashim, 2006).

The water absorbed during the swell was calculated by using Equation 4.2.

$$w_a = \varepsilon\,(1 + e_0)\,\frac{\gamma_s}{\gamma_w} \qquad (4.2)$$

where

w_a = water absorption

ε = swell

e = initial void ratio pf the specimen.

γ_s = density of soil particles

γ_w = density of water.

The absorption versus time and the absorption versus swell for the obtained samples at different proportion were plotted in graph to observe the effect of quarry dust on the swell behavior and absorption capability during the swell processes. From Table 4.6 it can be observed that the absorbed water and the swell decrease with addition of quarry dust of 10, 20, and 30% respectively.

Table 4.8: Result of absorbed water during swell

soil	proportion	swell%	wa%
T1	0%	3.80	14.05
	10%	2.50	10.21
	20%	2.16	8.03
	30%	1.69	6.67
T2	0%	11.39	46.70
	10%	10.75	41.76
	20%	6.51	25.39
	30%	6.46	23.96
T3	0%	13.46	52.31
	10%	9.01	35.02
	20%	6.68	25.95
	30%	4.39	16.93
T4	0%	19.2	106.83
	10%	16.34	74.65
	20%	15.86	65.93
	30%	12.66	55.13

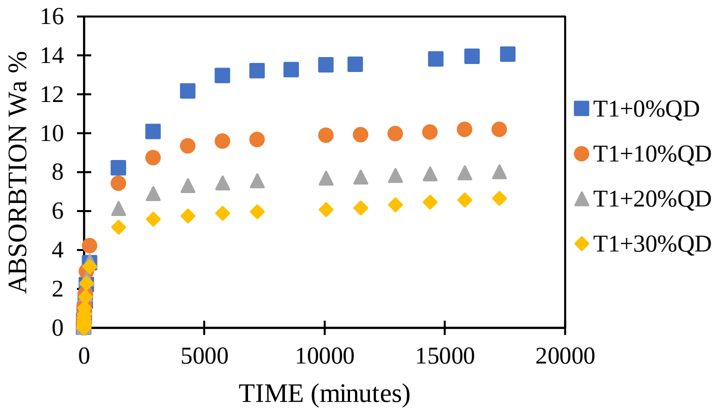

Figure 4.23: Water absorption versus time for soil T1with different percentage of quarry dust

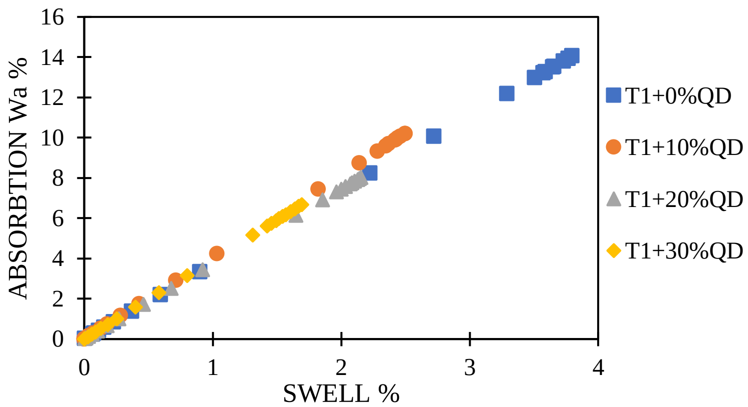

Figure 4.24: Water absorption versus swell for soil T1 with different percentage of quarry dust

It was observed that the water absorption during the swell was decreased in soil T1 with addition of quarry dust as shown in Figure 4.23 and 4.24. The water absorption for the natural soil was 14.05% and with addition of 10, 20 and 30% of quarry dust it was decreased to 10.21, 8.03, and 6.67% respectively. Moreover, it was observed that with increasing percentage of quarry dust there was a gradual decrease in the swell and water absorption.

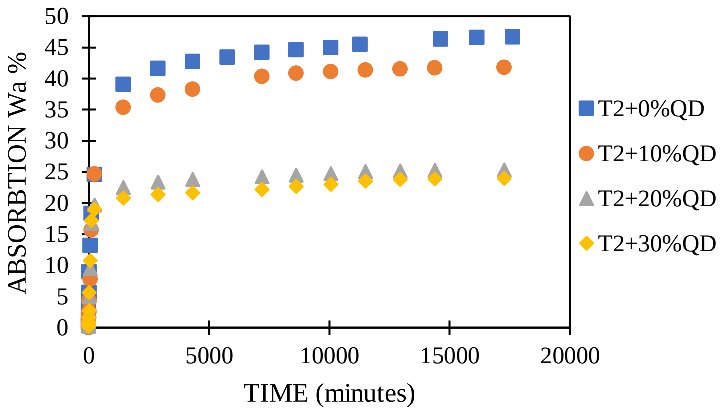

Figure 4.25: Water absorption versus time for soil T2 with different percentage of quarry dust

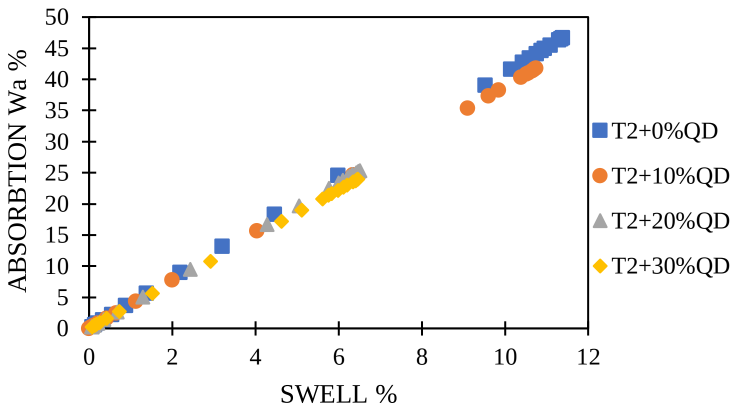

Figure 4.26: Water absorption versus swell for soil T2 with different percentage of quarry dust

For soil T2 the absorbed water with addition of 0% quarry dust was 46.7%, the absorbed water decreased with addition of 10, 20, and 30% of quarry dust to 41.76, 25.31, and 23.88% respectively. Furthermore, the swell decreased gradually with the increasing of percent quarry dust. However, the effect of 20, and 30% of quarry dust on the swell and water absorption were quite similar as shown Figure 4.25 and 4.26.

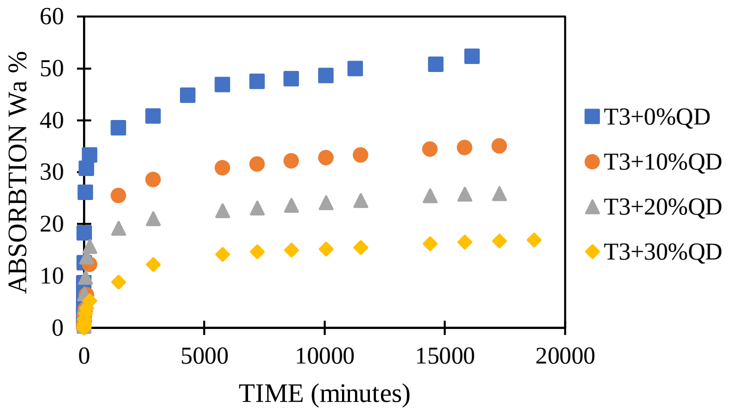

Figure 4.27: Water absorption versus time for soil T3 with different percentage of quarry dust

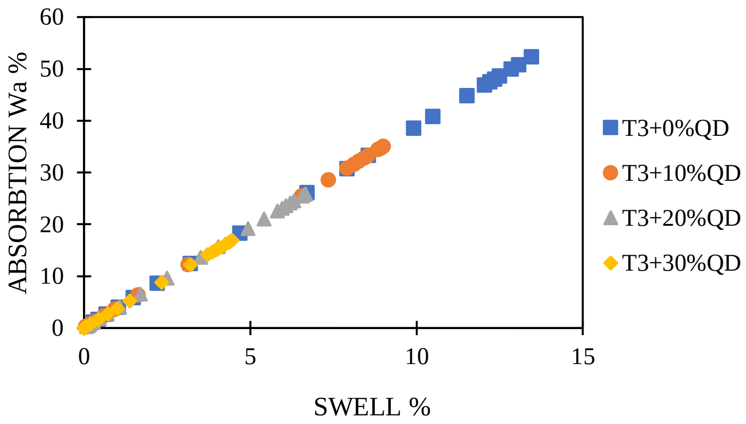

Figure 4.28: Water absorption versus swell for soil T3 with different percentage of quarry dust

The water absorption decreased gradually in soil T3 with the increasing of quarry dust percentage as shown in Figure 4.27. The water absorption was 52.31% without quarry dust, but with addition of 10, 20, and 30% of quarry dust the water absorption decreased to 35.02, 25.53 and 16.71% respectively. In addition, the swell decreased with increasing the proportion of quarry dust as it was expected (Figure 4.28).

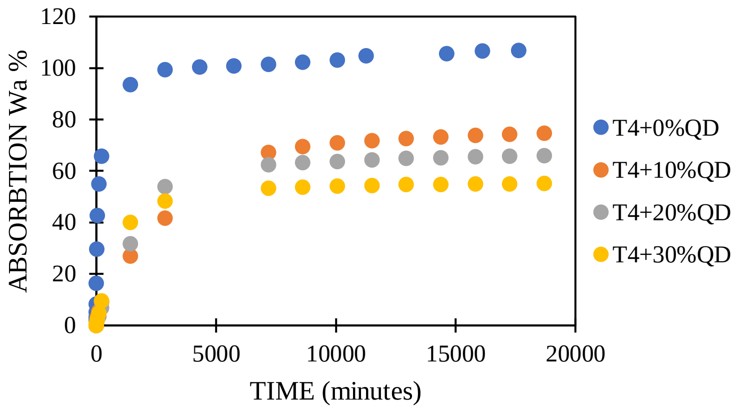

Figure 4.29: Water absorption versus time for soil T4 with different percentage of quarry dust

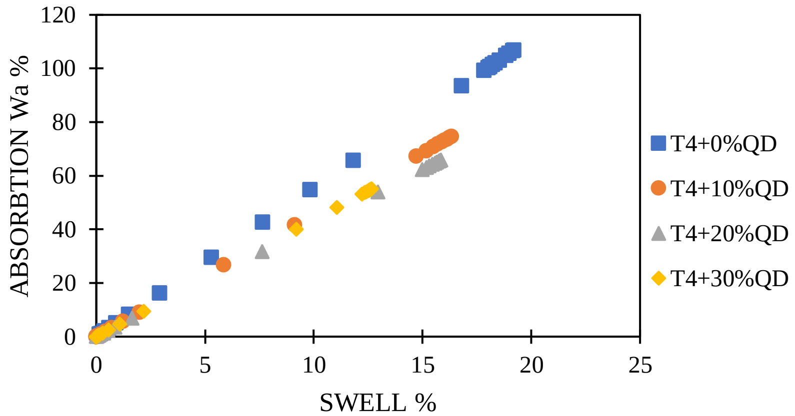

Figure 4.30: Water absorption versus swell for soil T4 with different percentage quarry dust

The water absorption line angle in soil T4 decreased with addition of quarry dust but with addition of 10, and 20% of quarry dust the water absorption was quite similar as shown in Figure 4.29. However, the water absorption decreased from 106.54% at 0% to 74.66, 65.69, and 54.97% at addition of 10, 20, and 30% of quarry dust. Further, the swell decreased about 7% with the addition of 30% quarry dust (Figure 4.30).

4.5 One Dimensional Consolidation

T3 soil was subjected to one dimensional consolidation test to observe the effect of QD on compressibility properties of an expansive clay sample. The samples were put to consolidate after they reached their maximum swell. The void ratio versus effective pressure (kPa) in logarithmic scale represents the obtained results as shown in Figure 4.31. The rebound index (c_r), compression index (c_c), and pre-consolidation pressure (kPa) are the parameters obtained from the one-dimensional consolidation test and they are shown in Table 4.7. Moreover, the hydraulic conductivity (k_s) was calculated for all samples under different pressure ranges and the results are shown in table 4.8. As it can be seen from Table 4.7 the compression index (c_c), rebound index (c_r) and pre-consolidation pressure (kPa) decreased by the addition of quarry dust except for the rebound index (c_r) which showed a slight increase at 10% quarry dust and then decreased at the subsequent additions.

Table 4.9: One-dimensional consolidation parameters

Soil	Compression Index (Cc)	Rebound Index (Cr)	Pre-consolidation Pressure (kPa)
T3+0%QD	0.28	0.08	143.4
T3+10%QD	0.22	0.1	130
T3+20%QD	0.15	0.07	125
T3+30%QD	0.16	0.05	110

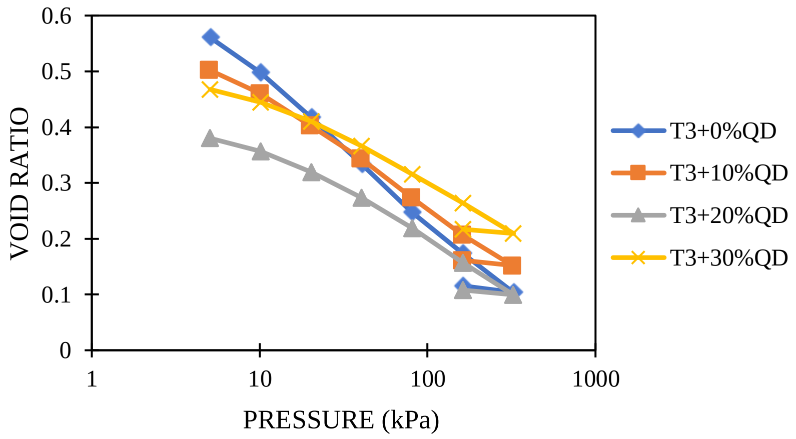

Figure 4.31: Void ratio versus pressure for soil T3

It was observed that compressibility was decreased with addition of 10 and 20 % of quarry dust by 21.4% and 46.4% respectively. With addition of 30% quarry dust compressibility displayed a slight increase by 6.25% as shown in Figure 4.29. However, considering the higher initial void ratio of 30% QD mixture the compressibility behavior of it is almost identical to 20% QD mixture.

Table 4.10: Hydraulic conductivity parameters from consolidation test

		T3+0%QD		
stress ranges	**av**	**Cv (m^2/s)**	**mv (m^2/kg)**	**ks (m/s)**
0-500	0.000462774	1.84679E-07	0.000264082	4.87705E-08
500-1000	0.000122	1.84679E-07	8.02104E-05	1.48132E-08
1000-2000	0.00007	1.84679E-07	4.79452E-05	8.85448E-09
2000-3530	0.00004060	1.84679E-07	2.92073E-05	5.39398E-09
		T3+10%QD		
stress ranges	**av**	**Cv (m^2/s)**	**mv (m^2/kg)**	**ks (m/s)**
0-500	0.000602601	2.2827E-07	0.000369399	8.4323E-08
500-1000	0.00014	2.2827E-07	0.000105263	2.40285E-08
1000-2000	0.000061	2.2827E-07	4.84127E-05	1.10512E-08
2000-3530	0.00003073	2.2827E-07	2.56304E-05	5.85066E-09
		T3+20%QD		
stress ranges	**av**	**Cv (m^2/s)**	**mv (m^2/kg)**	**ks (m/s)**
0-500	0.000211021	2.13697E-07	0.00015431	3.29756E-08
500-1000	0.000104	2.13697E-07	8.24089E-05	1.76105E-08
1000-2000	0.000061	2.13697E-07	5.04132E-05	1.07731E-08
2000-3530	0.00003241	2.13697E-07	2.82092E-05	6.02821E-09
		T3+30%QD		
stress ranges	**av**	**Cv (m^2/s)**	**mv (m^2/kg)**	**ks (m/s)**
0-500	0.00009433	2.4978E-07	6.70382E-05	1.67448E-08
500-1000	0.0001	2.4978E-07	7.35294E-05	1.83662E-08
1000-2000	0.00009	2.4978E-07	6.87023E-05	1.71605E-08
2000-3530	0.00000696	2.4978E-07	5.70217E-06	1.42429E-09

According to the hydraulic conductivity it was observed that addition of 10% quarry dust increased the hydraulic conductivity in all stress ranges. However, by increasing the percentage quarry dust from 10 to 20% most of the hydraulic conductivity values showed a slight overall decrease in all stress ranges. Further addition of quarry dust to 30% showed variating effect. In the small and high stress ranges the hydraulic conductivity showed decreasing, while middle range stress results a slight increase in hydraulic conductivity. Further, the importance of hydraulic conductivity of the soil depends on the purpose of using it. The saturated hydraulic conductivity obtained from consolidation test is considered as an indirect method and for a better understanding of hydraulic conductivity behavior of soil further direct experiments are required.

4.6 Unconfined Compressive Strength

This test method was adopted according to ASTM D2166 in order to determine the approximate compressive strength for cohesive soil (American society for testing and materials, 2013). This test was done on four types of expansive clays with 10, 20, and 30% of quarry dust added and were compacted to their optimum water content and maximum dry density in order to study the effect on their strength. According to Das (2009), the compressive strength for the cohesive soil decrease with the increase of the moisture content, also the unconfined compressive strength and consistency are related where increase in unconfined compressive strength changes the consistency from soft to hard as shown below in Table 4.9. The comparing of the compressive strength with its consistency for the obtained soils with addition different proportion of quarry dust is shown in Table 4.10.

Table 4.11: The relationship between consistency and compression strength (Das, 2009)

Consistency	compression strength q_u kN/m^2
Very soft	0–25
Soft	25–50
Medium	50–100
Stiff	100–200
Very Stiff	200–400
Hard	>400

Table 4.12: Consistency and unconfined compressive strength at different quarry dust content

soil	proportion	Max. load (kN)	stress (kN/m^2)	consistency
T1	0%	0.16	133.89	Stiff
	10%	0.196	164.02	Stiff
	20%	0.34	284.52	Very stiff
	30%	0.236	197.66	Stiff
T2	0%	0.233	194.98	Stiff
	10%	0.175	146.44	Stiff
	20%	0.246	205.86	Very stiff
	30%	0.27	225.94	Very stiff
T3	0%	0.159	133.05	Stiff
	10%	0.181	151.46	Stiff
	20%	0.194	162.34	Stiff
	30%	0.238	199.16	Stiff
T4	0%	0.08	66.95	Medium
	10%	0.153	128.03	Stiff
	20%	0.173	144.77	Stiff
	30%	0.219	183.26	Stiff

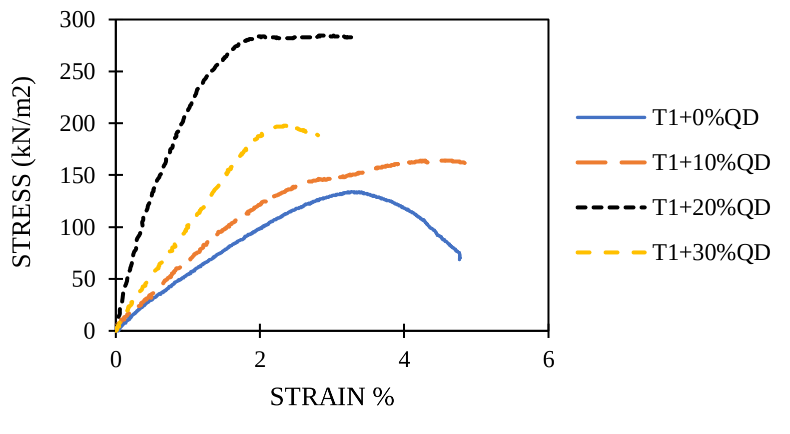

Figure 4.32: Stress versus strain based on unconfined compression test for soil T1

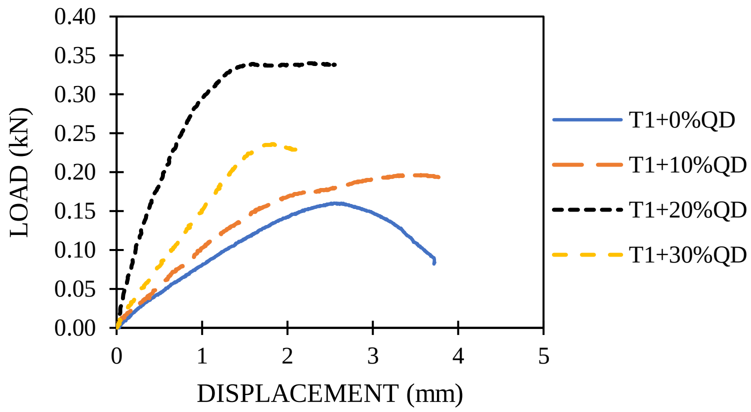

Figure 4.33: Applied load versus displacement based on unconfined compression test for soil T1

The unconfined compressive strength for T1 sample without addition of quarry dust was recorded as 133.89 kPa, with addition of quarry dust of 10, and 20% the compressive strength increased to 164.02, and 284.52 kPa respectively, on the other hand with further addition of 30% of quarry dust the unconfined compressive strength reduced to 197.66 kPa as shown in Figure 4.32, and 4.33. However, according to Subash et al., (2016) the compressive strength with addition of 6% of plastic granules increased, however when the addition increased beyond 6% the compressive strength start decreasing.

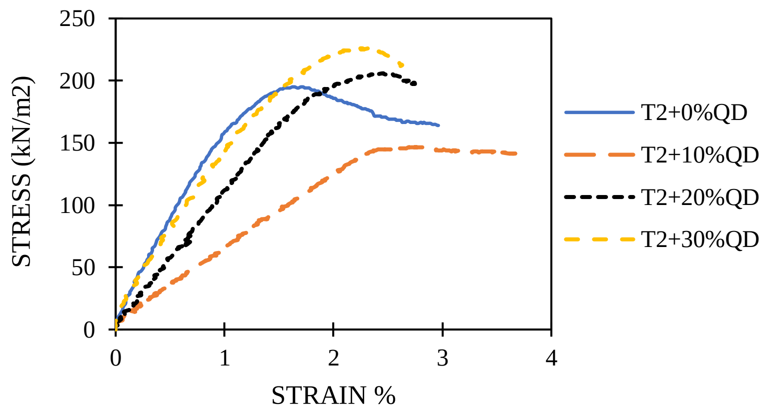

Figure 4.34: Stress versus Strain based on unconfined compression test for soil T2

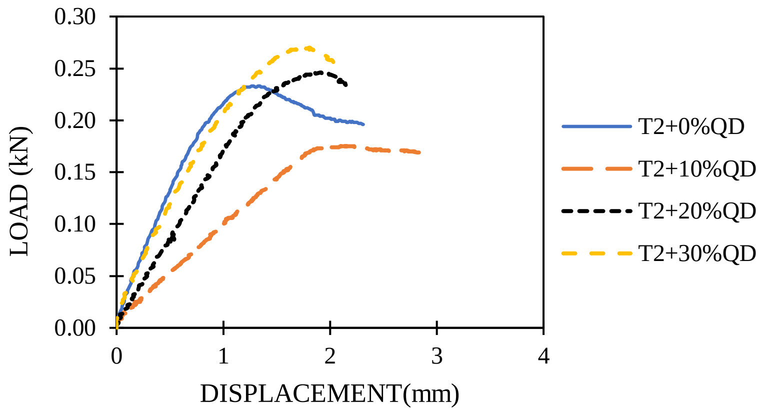

Figure 4.35: Applied load versus displacement based on unconfined compression test for soil T2

For soil T2, the unconfined compressive strength and stress was 194.98 kPa, but with addition of quarry dust by 10% the compressive strength reduced to 146.44 kPa, however, with increasing the addition to 20% and 30% the compressive strength start increasing again to 205.86 kPa, and 225.94 kPa respectively as shown in Figure 4.34, and 4.35.

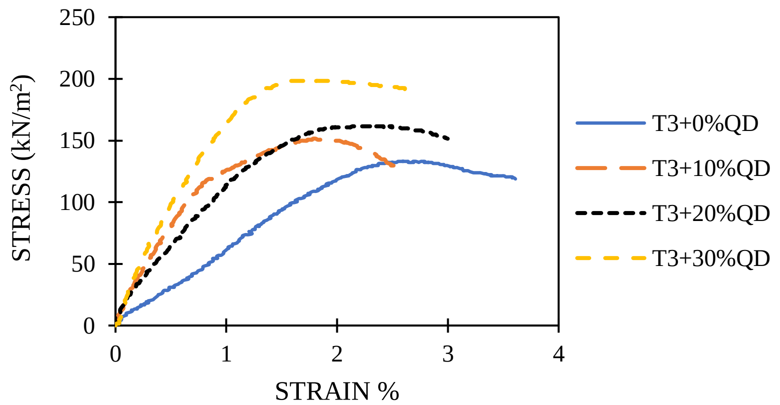

Figure 4.36: Stress versus strain based on unconfined compression test for soil T3

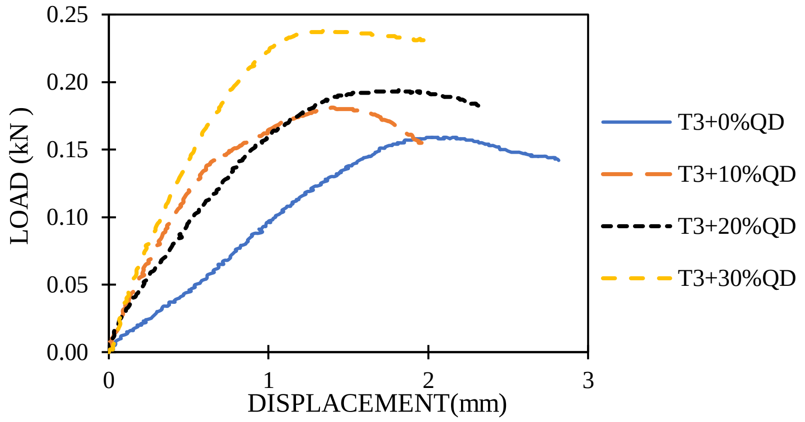

Figure 4.37: Applied load versus displacement based on unconfined compression test for soil T3

The compressive strength for soil T3 at 0% addition of quarry dust was 133.05 kPa. With addition of 10, 20, and 30% of quarry dust the unconfined compressive strength increased to 151.46, 162.34, and 199.16 kPa respectively as shown in Figure 4.36, and 4.37.

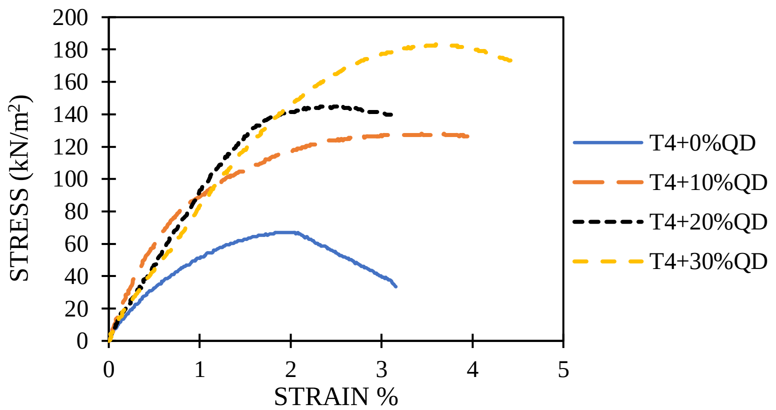

Figure 4.38: Stress versus strain based on unconfined compression test for soil T4

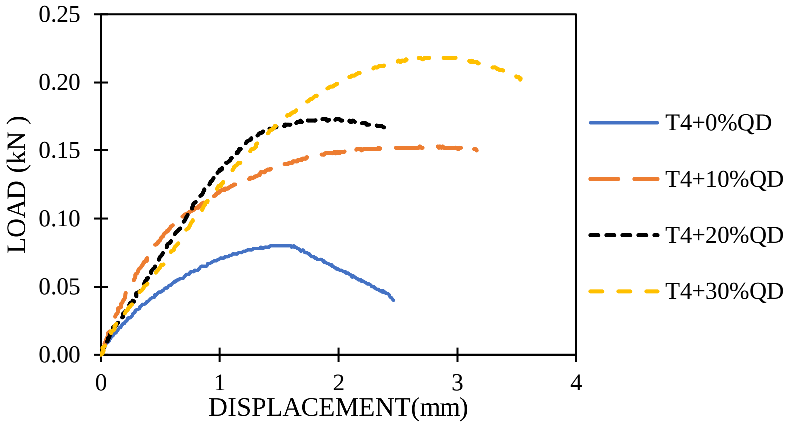

Figure 4.39: Applied load versus displacement based on unconfined compression test for soil T4

Also, for soil T4 the unconfined compressive strength increased gradually from 66.95 kPa at 0% addition of quarry dust to 128.03, 144.77, and 183.26 kPa at addition of 10, 20, and 30% of quarry dust as shown in Figure 4.38 and 4.39.

In general, it was observed that with increasing the addition of quarry dust there was a significant increase in the maximum dry density, therefore there was an overall increment in the unconfined compressive strength value for the obtained mixtures due to increase of effective stress in the samples.

CHAPTER 5

CONCLUSION

The effect of quarry dust in order to stabilize four local soils characterized as potential expansive is presented. The quarry dust was introduced and implemented as a stabilizer according to its effect on the environment. The percentages of quarry dust were introduced as 10, 20, and 30% by weight of soil. Related to the obtained results the conclusion can be listed as following:

1. The specific gravity value for the obtained soils increased with the addition of quarry dust at all proportions.
2. Liquid limit and plasticity index values showed a decrement in overall with addition of 10, 20, and 30 % of quarry dust in soil T1 and T2. Moreover, Soil T3 and T4 showed reduction in liquid limit and plasticity index at all proportions.
3. Addition of quarry dust enhanced the compaction properties of the obtained soil at all proportions, result showed decrease in the optimum water content and gradually increasing in the maximum dry density.
4. Implementation of quarry dust and employing it as a construction material in order to stabilize the obtained soils showed a significant decrement in the potential swelling for all soils at all proportions. The total swell was decreased at all proportions which shows an inverse relationship between quarry dust replacement proportion and absorption water, where increase in quarry dust proportion leads to decrease in the absorbed water during the swell process.
5. The consolidation test was done on one soil (T3), the compressibility was decreased at all proportions of quarry dust. Hydraulic conductivity is obtained from the consolidation test as an indirect method and it showed a slight decrease in hydraulic conductivity at all stress ranges with 10% addition, also with addition of 20% the hydraulic conductivity in overall increased. For further addition of quarry dust to 30% there was a variation in results such as increasing and decreasing related to the stress range.

6. The value of the unconfined compressive strength increased for all the obtained soils, except for soil T2 which showed a variation. In T2 sample with addition of 10% of quarry dust by weight of soil the compressive strength decreased, though it increased again with addition of 20%, and 30% of quarry dust.

   As a future study, it is recommended to study on strength properties of the obtained mixtures in drained and undrained conditions. Moreover, studying the hydraulic conductivity by using a direct method in order to determine the saturated and unsaturated hydraulic conductivity would be helpful to understand the hydraulic behavior of soil in different degrees of saturation. Also, other proportions of quarry dust mixed with the expansive soils can be studied to determine the optimum percentage of QD needed to fulfill the required engineering properties. Also, the study shows that quarry dust alone is not sufficient to decrease the swell capacity for soils with high swell potential as well as the soils with plasticity index above A-line in the plasticity chart, due to that another material, preferably pozzolanic, is required to reduce PI as well as create a non-swell mixture. Furthermore, examining the soil by using X-ray diffraction could be useful for determining the minerology of the soil as well as the quarry dust. Also, examining the soil by using scanning electron microscope is a great option to provide a clear vision of the microstructural behavior for the obtained soil during the stabilization process.

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