ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

INVESTIGATION INTO THE EFFECTS OF NANO-MATERIALS AND NANO-POLYMERS ON CLAYS

M.Sc. THESIS

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Department of Civil Engineering

Soil Mechanic and Geotechnical Engineering Programme

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ISTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

NANO MALZEME VE NANO POLİMERLERİN KİL ZEMİNLER ÜZERİNDEKİ ETKİLERİNİN İNCELENMESİ

YÜKSEK LİSANS TEZİ

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Amin Yeganeh Rikhtehgar, a **M.Sc.** student of İTU **Graduate School of Science Engineering and Technology** student ID **50131301**, successfully defended the **thesis** entitled "**Investigation into The Effects of Nano-Materials and Nano-Polymers on Clays**", which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

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To my dear Family,

FOREWORD

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ABBREVIATIONS

AASHTO : American Association of State Highway and Transportation Officials
CBR : California Bearing Ratio
CH : High Plasticity clay Soil
CL : Low Plasticity clay Soil
C : Nano-Carbon Fibers
LL : Liquid Limit
NC : Nano-Clay
PL : Plastic Limit
PI : Plasticity Index
SPT : Standard Penetration Test
SI : Nano-Silicon Powder
USCS : Unified Soil Classification System
ZY : Zycosil

SYMBOLS

q_u **:** Unconfined Compression Strength
w : Water Content
w_{opt} **:** Optimum Water Content
γ_d **:** Dry Unit Weight
ε **:** Strain
σ **:** Normal Stress

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INVESTIGATING OF EFFECTS OF NANO-MATERIALS AND NANO-POLYMERS ON CLAY

SUMMARY

This thesis consists of investigations into the effects of different Nano-Materials and Nano-based polymers on high plasticity clay, which had been obtained from Ciftalan district in the north of Istanbul.

In this thesis engineering properties of clay tried to be improved. To evaluate the strength characteristics of stabilized clayey soil, laboratory investigation tests had been performed at the Istanbul Technical University's Prof. Dr. Hamdi Peynircioğlu Soil Mechanics Laboratory. First of all, the soils which had been obtained from the field were classified after using hydrometer analysis and Atterberg limits. Then, Atterberg limit tests had done on the clayey soil mixed with Nano-Material and Nano-Material based Polymers as alternative materials with four different percentages. Afterward, optimum water content and the maximum dry unit weight of blended soil were determined by using modified Harvard miniature compaction apparatus. Then, samples, which were obtained from the compaction equipment, kept in a desiccator for one, seven, and twenty-eight days for curing. Unconfined compression tests had performed on these samples. Finally, the samples, which had better results in an unconfined comparison test, with a specific percentage of Nano-Material, were prepared with respect to each mixture's optimum water content and tested. Soaked CBR, Consolidation, Permeability, and Triaxial Shear tests were conducted on these samples. Moreover, during conducting tests interesting behavior of mixed soils were observed after drying, especially, the ones which had mixed with Zycosil and Nano-Silica Powder. Therefore, "Direct Shear Tests" were done only on these mixtures with the condition that the sample is still compacted and wet, and the other condition that the compacted sample dried and submerged in the water again.

The study examined the effect of different Nano-Materials like Nano-Carbon fibers, Nano-Clay, Nano-Silica Powder, and Nano-Polymers like CBR Plus, and Zycosil in four and six different percentage to improve the Atterberg limit parameters, shear strength and effective strength, consolidation, permeability, and soaked CBR. The results showed that Nano-Materials and Nano-Polymers increased the strength of Clay in different amounts and decreased the amount of swelling pressure and consolidation. The California Bearing Ratio of the soil increased with increasing the percentage of Nano-Materials, especially Nano-Polymers. Results obtained form consolidated undrained triaxial test showed that Nano-Materials and Polymers could not change the internal friction angle, significantly. However, except Zycosil, other additives highly increased the cohesion. At the end, after applying direct shear test on dried and wet clay, it revealed that behavior of soil could change after getting dried and put in the water again. This process increases the cohesion of the clay and does not change the internal friction angle considerably. Though, results shows that additive-mixed clay are more sensitive and can demonstrate uneven performance that is more extensive in both cohesion and internal friction angle aspects.

NANO MALZEME VE NANO POLİMERLERİN KİL ZEMİNLER ÜZERİNDEKİ ETKİLERİNİN İNCELENMESİ

ÖZET

Doğada serbest halde bulunan kil zeminler, farklılıklar gösteren özellikleri bakımından dünyanın en sıradışı malzemelerinden birisi olarak sınıflandırılmaktadır. Sağlık, sanat, mühendislik gibi endüstrilerde çok amaçlı olarak kullanılan killer dünyanın kurak ve yarı kurak bölgelerinde çok sık ortaya çıkmaktadır. Afrika, Avusturalya, Hindistan, Güney Amerika, Kanada gibi bölgelerde sık olarak karşılaşılan killer, karmaşık özellikleri bakımından inşaat mühendisliği uygulamalarında ciddi sorunlar yaratabilmektedir. Killi zeminler üzerine teşkil edilen inşaat yapıları, kil içeren bu zemin tabakalarının su ile birleşerek şişmesi sonucunda hasar görmektedirler. Killerin su ile teması sonrasında şişmesinden kaynaklı, bu zeminler üzerine teşkil edilen yapı temelleri, istinat duvarları, yol, kaldırım, su yapıları, havalimanı gibi sistemler hacimsel olarak deformasyona maruz kalmaktadır. Killerin şişmesinden kaynaklı olarak, bu tabakalara önlem alınmadan teşkil edilen üst yapılara ait döşemelerde, kapı ve pencerelerde yapı çatlakları gözlenmektedir. Bu problemlerin giderilmesine yönelik çalışmalar, öngörülemeyen yüksek maliyetler doğurmaktadır.

Yukarıda bahsedildiği gibi, bu tip şişebilen killer üzerine yapılan yol ve kaplama gibi stratejik önem arzeden yapılar, doğru tasarım izlenmeden inşa edildiğinde ciddi hasarlara maruz kalabilmektedir. Yol tasarımında alt temel malzemesinin kalitesi, inşaat süresince ve servis ömrü boyunca maliyetleri etkileyen başlıca unsurdur. Buradan hareketle, şişme potansiyeli gösterecek zeminlere karşı üstyapının güvenli bir şekilde tasarlanabilmesi için alt temelin sağlam bir şekilde sıkıştırılması, alt temel inşaatında geosentetik materyallerin kullanılması, kireç, çimento, polimer gibi katkılardan faydalanılması gibi mühendislik çözümleri geliştirilmiştir. Bu yöntemlerin tercih edilmemesinin bir sonucu olarak, oldukça kalın tabakalı alt temeller tasarlanmakta ve bu durum ekonomik açıdan projelerde sıkıntılar yaratmaktadır. Yollara ait alt temellerin tasarımında kireç ve çimento gibi katkı malzemelerinin kullanılması etkili bir çözüm olsa da, pek çok müteahhit firma, sıkıştırmaya bağlı imalat sıkıntılarından ötürü bu yola şüpheli yaklaşabilmektedir.

Son dönemlerde yeni bir uygulama olarak, araştırmacılar nano-teknolojiye dayanan yeni bir malzeme stabilizasyonu yöntemi üzerine çalışmaktadır. Nanoteknolojinin alt temel malzemesinin iyileştirilmesinde kullanılması fikri ilk defa 1959 yılında Richard Feynman tarafından önerilmiştir. Bu tarihten itibaren, nanoteknolojik malzemelerin mühendislik ürünlerinde kullanılması yaygınlaşmıştır. Nano malzemelerin boyutları 1-100 nm boyutları arasındadır.

Nanoteknolojinin geoteknik mühendisliği uygulamalarında kullanılması, ilk olarak nanoteknolojik malzemelerin zayıf zeminlere ait mukavemet parametrelerinin iyileştirilmesi amacıyla olmuştur. Nanoteknolojik malzemeler tanecik boyutunun küçük olmasından dolayı daha büyük bir özgül yüzeye sahiptir. Bu özellikleri

itibariyle, çok az bir miktarda dahi olsa, zemin içinde kullanılan nanoteknolojik malzemeler içinde girdikleri bileşimin morfolojik yapısını değiştirerek fiziksel ve kimyasal özelliklerinde iyileşmeye sebep olmaktadır. Bu nedenle farklı tipteki zeminlerin fiziksel ve kimyasal koşullarını iyileştirmek amacıyla nanoteknolojinin güvenle kullanımı günümüzde yaygınlaşmıştır. Buna ek olarak, zemin ve kayalarda bulunan minerallerin kimyasal reaksiyonları nano ölçekte gözlenebilmektedir. Bu durum, nanoteknolojik malzemelerin sızdırmazlık, jet grout, zemin stabilizasyonu gibi geoteknik uygulamalardaki rolünü belirlemiştir.

Bu alanda yapılan kısıtlı araştırmalardan elde edilen bazı sonuçlara gore, nanoteknolojik malzemelerin zeminlerin mühendislik özellikleri üzerindeki etkisi iki açıdan ele alınmaktadır. Bunlardan ilki zemin içerisindeki doğal nanoteknolojik özellikteki partiküllerin reaksiyonudur. İkinci olarak, doğal olmayan yollarda zemine ilave edilen nanomalzemelerin zemin içindeki reaksiyonudur. Bu yaklaşıma göre nanoteknolojik malzemelerin zeminlerin mühendislik özellikleri üzerine etkisini araştıran Zhang, az bir miktarda nano malzemenin dahi zeminlerin fiziksel ve kimyasal niteliklerini iyileştirdiğini ortaya koymuştur. Bunun yanında partiküller arasında nanoboşluklar içeren nanoteknolojik malzemeli zeminlerin daha yüksek likit ve plastik limit değerlerine sahip olduğunu, fiber nano malzemelerin ise zeminin kayma mukavemetini arttırdığını görmüştür.

Sahada yapılan çalışmalar, zeminlerin içinde kullanılan nanomalzemelerin, bileşimlerin kimyasal özelliklerini, fiziksel özelliklerinden daha çok etkilediğini göstermiştir. Bu çalışmalardan hareketle, nanomalzemelerin zayıf zeminlerin iyileştirilmesinde kullanımı pratik kazanmıştır.

Bu tez çalışmasında, yeni nanoteknolojik malzemelerin ve nanoteknolojik özellikteki polimer katkıların killi zeminlerin stabilizasyonundaki performans ve etkileri araştırılmıştır. Diğer bir ifadeyle, nanomalzemelerin yolların alt temel tabakalarında kullanımının uygunluğu irdelenmiştir. İstanbul şehrinin Çiftalan bölgesinden alınan kil numunesi üzerinde nanomalzemeler kullanılarak mühendislik parametrelerindeki değişim gözlenmiştir. Stabilize edilmiş kil malzemenin mukavemet parametrelerinin belirlenmesi amacıyla, İstanbul Teknik Üniversitesi Hamdi Peynircioğlu Zemin Mekaniği Laboratuarı'nda testler yapılmıştır. İlk olarak, sahadan alınan zemin numuneleri üzerinde hidrometre deneyleri ve kıvam limitleri tayini yapılmıştır. Sonrasında nano malzeme ve nano özellikteki polimer eklenmiş kil zemin numuneleri üzerinde deneyler tekrarlanmıştır. Katkı malzemelerinin etkisini daha açık gözlemlemek için nano malzemeler 4 farklı yüzdede numunelere eklenmiştir. Bu işlemlerin ardından minyatür Harward sıkıştırma aparatı yardımıyla numunelerin optimum su muhtevası ve maksimum kuru birim hakim ağırlık değerleri bulunmuştur. Sıkıştırma sonrasında numuneler aparattan alınarak 1, 7 ve 28 gün boyunca desikatörde kür için bekletilmiştir. Numuneler üzerinde serbest basınç deneyi yapıldıktan sonra, bu deney sonucunda daha iyi sonuçlar veren karışım numuneleri belirlenmiştir. Söz konusu numuneler CBR, konsolidasyon, permeabilite ve üç eksenli testlerine tabi tutulmuştur. Testler sırasında Zikosil ve Nanosilis toz katkılı numunelerin kuruma sonrasında sıradışı davranışlar gösterdiği görülmüştür. Zikosil ve Nanosilis toz içeren numuneler üzerinde direk kesme deneyi yapılmıştır. Deneyler ilk olarak sıkışmış ve ıslak numunelerde, sonrasında kurutulmuş ve yeniden suya doyurulmuş numunelerde tekrarlanmıştır.

Bu çalışma; nanokarbon lifler, nano kil, nano silis toz, CBR Plus nanopolimer malzeme ve Zikosil gibi farklı tipteki nanomalzemelerin 6 farklı yüzdede kil

malzemeye karıştırılarak, karışımların mühendislik özelliklerinin irdelenmesi amacıyla yapılmıştır. Karışımlar üzerinde Atterberg limitleri tayini, kayma mukavemeti, efektif gerilme, konsolidasyon, permeabilite ve ıslak CBR deneyleri yapılmıştır. Elde edilen sonuçlara göre, farklı yüzdelerde kil malzemelerin içinde kullanılan nano malzeme ve nano polimerler numunelerin mukavemetini arttırırken, konsolidasyon ve şişme potansiyellerini azaltmıştır. Nano malzemelerin yüzdesel olarak artışı, CBR değerlerini arttırmıştır. Nano polimer malzemede bu değer artışı daha belirgin olarak gözlenmiştir. Konsolidasyonlu drenajsız üç eksenli deneyden elde edilen sonuçlara göre, nano malzeme ve polimerlerin içsel sürtünme açısında bir değişiklik meydana getirmediği görülmüştür. Zikosil haricindeki tüm katkı malzemeleri numunelerin kohezyon değerlerinde artış meydana getirmiştir. Sonuç olarak, kuru ve ıslak numuneler üzerinde yapılan direk kesme deneyinde numunenin davranışının farklılık göstermediği görülmüştür. Deney sonrasında içsel sürtünme açısında artış gözlenmezken, kohezyonun arttığı belirlenmiştir. Deneysel çalışma sonucunda katkılı kil malzemelerin kayma mukavemeti ve kohezyon gibi dayanım parametreleri açısından daha hassas olduğu ve stabil davranışlar göstermeyebileceği anlaşılmıştır.

1. INTRODUCTION

Clays, because of their interesting properties, are always one of the most exotic materials in the world. They are used in many industries from medical to engineering and even art. Clays are existed in many parts of the world, mostly in the arid or semi-arid regions of the humid and temperate zones such as Africa, Australia, India, South America, United States, and some regions in Canada. This never means these soils do not exist elsewhere, because they can be found almost everywhere (Shuai and Fredlund, 1998; Wayne et al., 1984). However, sometimes these clays can be such a troublemaker especially for civil engineers, and make enormous difficulties. For further explanation, consider structures constructed on clayey soils, these soils start swelling when exposed to water and shrink as soon as water is squeezed out. Significant failure to the civil infrastructure, such as foundations, retaining walls, pavements, airports, sidewalks, canal beds, and linings happen due to volumetric changes in these type of soils. The problems cause damage such as diagonal cracks above doors and windows, pavement cracking, and heaving of floors. The charge of fixing the damages created by expansive soils to civil engineering structures is estimated many billions of dollars worldwide, annually (Katti and Shanmugasundaram , 2001).

Indeed, as a global problem, expansive soils known as a potential natural hazard which makes several challenges for civil engineers and causes massive damages to structures in case of inadequate treatment (Al-Rawas et al., 2002). High plasticity and being relatively stiff or dense are general features of expansive soils which happen because of the presence of some Montmorillonite clay mineral. The greater amount of monovalent cations absorbed in the clay mineral (e.g. sodium) means is more severe expansion. Near the ground surface, where the soil face with seasonal and environmental changes, is the most common place to observe the expansive aspect of clays (Fredlund and Rahardjo, 1993).

As mentioned above subgrades of roads and pavements, as one the most important and strategic structures, suffer from problems caused by expansive soils. The quality of subgrade has an intense effect on both the initial cost of pavement and the indirect maintenance costs as well. Therefore, scientists have tried to invent some ways to deal with soft pavement subgrades such as attempting to dry and compact the subgrade, reinforcing the subgrade with a geosynthetic material, applying a chemical stabilizer such as lime, cement, polymer, or other additives, and/or designing a very thick and expensive pavement section. Nonetheless, mixing the soil with lime and cement as a traditional treatment way is so effective, but many contractors are hesitant to use them due to issues with dust control and other handling problems. Then scientist start to find more new methods and materials that will be explained comprehensively in the next chapter, but to be able to make innovative ways, it is important to have a good understanding of the structure of the Clay-Water bonding and the history of previous studies.

Experiences in World War II had indicated that there is a need for stabilization of soils in the roads and airfields. Consequently, in the late 1940s at M.I.T and Princeton universities research programs initiated with the aim of developing new materials that would be able to mix quickly and easily with, or even better sprayed on, soft soils to make the treated material firm enough to carry military vehicles.

During 1950-56, Researchers at M.I.T provide useful background for the program currently offered by the U.S. Air Force to state much the similar problems of quick solidification of soft soil. This study, which had performed for the U.S. Army Engineer Research and Development Laboratory at Fort Belvoir, Virginia, aimed at rapid solidification of soft, wet, plastic soils that in their early state were too weak to bear the traffic.

That research had shown that a successful exploration of appropriate materials and methods would need a good understanding of the compositional and physicochemical properties of fine-grained soils. Therefore, under the leadership of Professor T.W. Lambe correlations between composition and properties had been developed, and the mysteries of clay-water- electrolyte structures were discovered.

This research proved that organic chemicals could mix with the wet soil followed by polymerization reactions by binding the soil particles together. Key in the process was

the attachment of the polymer to the clay particles by ion exchange reactions. Calcium acrylate recognized as the most suitable polymeric compound for this purpose. Wet clays turned to a stiff, rubber-like mass within minutes after thorough mixing of the chemical with the soil.

Although initially exciting this approach suffered several disadvantages. Adequate mixing of additives, either as a powder or as a solution, with wet clay, is very difficult, requiring special equipment, much energy, and time. Further, the treated material was water sensitive and underwent changes in strength and stiffness as a result of wetting and drying. Well into the research program it was realized, with the aid of analytical studies of the stress-deformation behavior of layered systems, that the strength and stiffness requirements for the vehicle loads were greater than could be achieved, given the very low strength and stiffness of the subgrade soil beneath the treated layer. Additionally, the material costs were high.

Given these limitations, attention began to focus on more typical forms of admixture stabilization, such as portland cement, lime, and asphalt, and these materials are the primary admixtures in use today. Actually, the study was a turning point for other researchers to work on new stabilization methods and materials for different types of soils and structures, as well.

Recently, as a new approach, researchers are working on another and new area of soil stabilizers based on Nano-Materials. For the first time Richard Feynman suggested the nanotechnologies idea in 1959, with this sentence "There's plenty of room at the bottom" (Feynman, 1960). After that, this technology developed in all branch of sciences. The Nano-Particle size usually is in range of 1-100nm (Horikoshi and Serpone, 2013).

The primary strategy of nanotechnology in geotechnical engineering is the improvement of soil parameters with application of Nano-Materials. The presence of only small amount of Nano-Material in the soil could influence significantly the physical and chemical behavior of soil due to a very high specific surface area of Nano-Particles, surface charges and their morphologic properties. Therefore, it becomes more reactive and potentially suitable for improving the properties of soil for various applications (Taha, 2009). Additionally, many of soil and rock minerals are nanomaterial and their chemical reactions occur in nanoscale. Because of this fact,

there is an enormous potential of nanotechnology's application in soil mechanics including seepage, grouting, soil stabilization etc. as well.

Actually, in the limited investigation performed in this field, the effects of Nano-Materials in engineering properties of soil have been considered mainly in two aspects including the effect of the presence of natural nanoparticles in the soil and the effect of adding Nano-Materials into the soil. In this way, Zhang studied the effect of natural Nano-Particles in the engineering properties of soil. He found that the presence of only a small amount of these Nano-Particles in the soil have significant force in the physical and chemical behavior and engineering properties of soil. He also concluded that the soils including Nano-Particles with interparticle nanovoids, usually demonstrated the higher liquid and plastic limits, and the presence of fiber shape Nano-Particles enhances the soil shear strength (Zhang, 2007).

Also, there is another study performed by Ghazi et al. on the plasticity and strength characteristic of a fine soil and its mixture with a nanomaterial that report the results of a series of Atterberg limits and unconfined compressive strength tests. The results showed that adding Modified Montmorillonite Nano-Clay into the soil increases the liquid limit and plasticity index and meaningfully improves the unconfined compressive strength of the soil (Ghazi and Baziar, 2011).

The performed studies indicate that the effect of application of Nano-Material in the field of chemical reactions produces is better than effect of the physical presence of nanomaterial in the soil structure, and this is significant in stabilization of weak soils.

In this thesis, it is tried to introduce a new Nano-Materials and Nano base Polymers on clayey soils stabilization and check their effects on. In other words, the main reason is to know where these materials can be suitable to use on clayey roads subgrades or not.

This study deals with an extensive experimental study to evaluate the effectiveness using nanoparticles size of different Nano-Materials like Nano-Carbon fibers, Nano-Clay, Nano-Silica powder, and Nano-Polymers like CBR Plus, and Zycosil in soil improvement to improve the Atterberg limit parameter, shear strength and effective strength, consolidation, CBR, and cohesion.

2. LITERATURE REVIEW

2.1 Methods of Soft Ground Improvement

Deserted areas because of their soil's unacceptable bearing capacities intensely increase, and the result of this is the shortage of suitable land and thus expanding requests for natural resources. Affected areas include those, which are susceptible to liquefaction, and those covered with soft clay and organic soils. Other areas are those in a landslide and contaminated land. However, in most geotechnical projects, it is not possible to obtain a construction site that will meet the design requirements without ground modification. The current practice is to modify the engineering properties of the native problematic soils to meet the design specifications. Nowadays, with the help of science soils such as, soft clays and organic soils are improved to meet the civil engineering requirements.

The target of soil stabilization is enhancing the soil's bearing capacity and make it more resistant to softening by holding the soil particles together with the help of water, waterproofing the particles or mix of them (Sherwood, 1993). Usually, the technology provides an alternative to a practical problem. The simplest stabilization processes are compaction and drainage (if water drains out of wet soil it becomes stronger). The other process is by improving gradation of particle size and further improvement can be achieved by adding binders to the weak soils (Rogers and Glendinning, 1996). Soil stabilization can be accomplished by several methods that are commonly used in different countries to improve the performance of the ground in situ. The techniques are divided into three categories: (Gunaratne, 2006)

1. Compaction — methods that typically are used to compact or densify soil in situ.

- Dynamic Compaction (Suitable soil types: Permeable, granular soils)
- Vibro Compaction (Suitable soil types: Granular soils)
- Compaction Grouting (Suitable soil types: Granular soils and low sensitivity soils)

- Surcharging with Prefabricated Vertical Drains (Soft, fine-grained soils)
- Blast-Densification and Vacuum-Induced Consolidation (Suitable soil types: granular soils)

2. Reinforcement — techniques that typically build a reinforcing element within the soil mass without necessarily changing the soil properties. The performance of the soil mass is improved by the inclusion of the reinforcing elements.

- Stone Columns (Suitable soil types: granular soils)
- Vibro Concrete Columns (Suitable soil types: soft and/or organic with under layered granular soil)
- Soil Nailing (Suitable soil types: Cohesive soil, weathered rock)
- Micropiles (Suitable soil types: Any subsurface soil or rock)
- Fracture Grouting (Suitable soil types: Any soil type)
- Fibers and Biotechnical (Suitable soil types: Any soil type)
- Geotextiles and Geosynthetics (Suitable soil types: Soft Soils)
- A new innovative technique: Rammed Aggregate Piers (Suitable soil types: Cohesive soils)

3. Fixation — techniques that fix or bind the soil particles together thereby increasing the soil's strength and decreasing its compressibility and permeability.

- Permeation Grouting (Suitable soil types: Sands and gravels, with less than 18% silt and 2% clay)
- Jet Grouting (Suitable soil types: Most effective for granular soils)
- Soil Mixing (Suitable soil types: Cohesive soils)
- Freezing and Vitrification (Suitable soil types: Wet cohesive Soils)

In which usually fixation technique is accompanied by chemical stabilization, which means the stabilization depends chiefly on chemical reactions between stabilizer (cementitious material) and soil minerals (pozzolanic materials) to achieve the desired effect. Through soil stabilization, unbound particles can be stabilized with cementitious materials (cement, lime, fly ash, bitumen, combination of these, and

polymers). The stabilized soil materials have a higher strength, lower permeability and lower compressibility than the native soil (Keller Inc., 2011). The decision to technological usage depends on which soil properties have to change. The main properties of soil that are essential for engineers are volume stability, strength, compressibility, permeability and durability(EuroSoilStab, 2002; Ingles and Metcalf, 1972; Sherwood, 1993).

As the definition of all stabilization techniques, which are mentioned above, is out of scope of this thesis, only four types of them that are suitable for cohesive soils will be explained.

2.1.1 Surcharging with prefabricated vertical drains

Surcharging consists of placing a temporary load (generally soil fill) on sites to pre-consolidate the soil prior to constructing the planned structure (Figure 2.1). The process improves the soil by compressing the soil, increasing its stiffness and shear strength. In partially or fully saturated soils, prefabricated vertical drains (PVDs) can be placed prior to surcharge placement to accelerate the drainage, reducing the necessary surcharge time.

Applicable soil types: Preloading is best suited for soft, fine-grained soils. Soft soils are generally easy to penetrate with PVDs and layers of stiff soil may require predrilling.

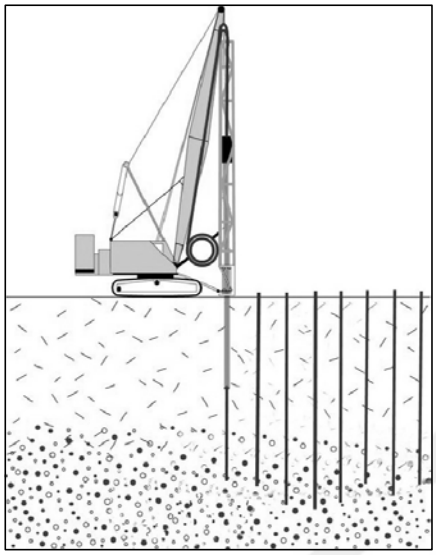
Equipment: Generally, a surcharge consists of a soil embankment and is placed with standard earthmoving equipment (trucks, dozers, etc.). Often the site surface is soft and wet, requiring low ground pressure equipment.

The PVDs are installed with a mast mounted on a backhoe or crane, often with little ground pressure tracks. A predrilling rig may be required if stiff layers must be penetrated.

Procedure: Fill soil is typically delivered to the area to be surcharged with dump trucks. Dozers are then used to push the soil into a mound. The height of the mound depends on the required pressure to achieve the necessary improvement.

The PVDs typically are in 308 m rolls and are fed into a steel rectangular tube (mandrel) from the top. The mandrel is pushed, vibrated, driven or jetted vertically into the ground with a mast mounted on a backhoe or crane. An anchor plate or bar

attached to the bottom of the PVD holds it in place in the soil as the mandrel is extracted. The PVD is then cut off slightly above the ground surface and another anchor is attached. The mandrel is moved to the next location and the process is repeated. If obstructions are encountered during installation, the wick drain location can be slightly offset.



(a)

(b)

Figure 2.1 : Surcharging with prefabricated vertical drains: (a) schematic, (b) field implementation. (Hayward Baker Inc.).

In very soft sites, piezometers and inclinometers, as well as staged loading, may be required to avoid the fill being placed too quickly, causing a bearing capacity or slope stability failure. If stiff layers must be penetrated, predrilling may be required. Settlement plates are placed in the surcharge. The elevation of these plates is measured to determine when the design settlement has occurred.

Materials: The first layer of surcharge generally consists of a drainage material to drain the water displaced from the ground during compression. Since surcharge soils are generally temporary in nature, their composition and degree of compaction are generally not critical. If the site settlement will result in some of the surcharge soil settling below finish grade, this height of fill is initially placed as compacted structural fill, to avoid having to excavate and replace it at the end of the surcharge program.

The PVD is composed of a 10 cm wide strip of corrugated or knobbed plastic wrapped in a woven filter fabric. The fabric is designed to remain permeable to allow the ground water to flow through it but not the soil.

Design: Generally, a surcharge program is considered when the site is underlain by soft fine-grained soils which will experience excessive settlement under the load of the planned structure. Using consolidation test data, a surcharge load and duration is selected to preconsolidate the soils sufficiently such that when the surcharge load is removed and the planned structure is constructed, the remaining settlement is acceptable. PVDs are selected if the required surcharge time is excessive for the project. The time required for the surcharge settlement to occur depends on the time it takes for the excess pore water pressure to dissipate. This is dictated by the soils permeability and the square of the distance the water has to travel to get to a permeable layer. The PVDs accelerate the drainage by shortening the drainage distance. The spacing of the PVDs are designed to reduce the consolidation time to an acceptable duration. The closer the drains are installed (typically 90 to 180 cm on center) the shorter the surcharge program is in duration.

Quality control and quality assurance: The height and unit weight of the surcharge should be documented to assure that the design pressure is being applied. The PVD manufacturer's specifications should be reviewed to confirm that the selected PVD is suitable for the application. During installation, the location, depth, and verticality are important to monitor and record. The settlement monitoring program is critical so that the completion of the surcharge program can be determined (Gunaratne, 2006).

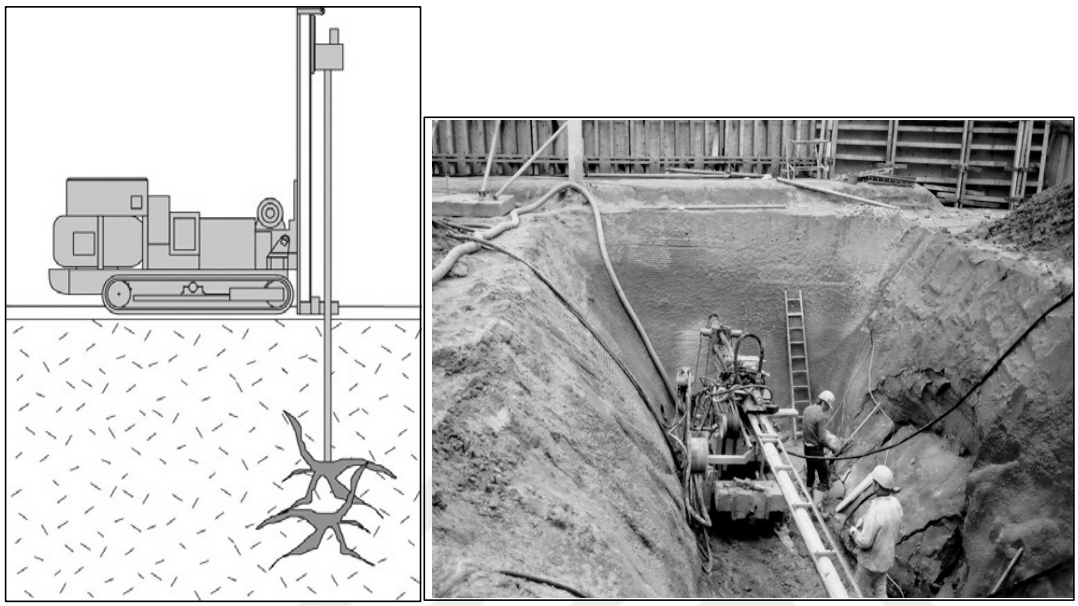
2.1.2 Fracture grouting

Fracture grouting, also known as compensation grouting, is the use of a grout slurry to hydrofracture and inject the soil between the foundation to be controlled and the process causing the settlement (Figure 2.2). Grout slurry is forced into soil fractures, thereby causing an expansion to take place counteracting the settlement that occurs or producing a controlled heave of the foundation. Multiple, discrete injections at multiple elevations can create a reinforced zone. The process is used to reduce or eliminate previous settlements, or to prevent the settlement of structures as underlying tunneling is performed.

A variation of fracture grouting is injection systems for expansive soils. The technique reduces the post-treatment expansive tendencies of the soil by either raising the soils' moisture content, filling the desiccation patterns in the clay or chemically treating the clay to reduce its affinity to water.

Applicable soil types: Since the soil is fractured, the technique can be performed in any soil type.

Equipment: For fracture grouting, the equipment consists of a drill rig to install the sleeve port pipes, grout injection tubing with packers, grout mixer, and a high-pressure



(a) (b)

Figure 2.2 : Fracture grouting: (a) schematic, (b) field implementation. (Hayward Baker Inc.).

grout pump. A sleeve port pipe is a steel or PVC pipe with openings at regular intervals along its length to permit grout injection at multiple locations along the pipes length. In addition, a precise real-time level surveying system is often required to measure the movements of the structure or the ground surface.

For injection of expansive soils, the equipment generally consists of a track mounted rig that pushes multiple injection pipes into the ground at the same time (Figure 2.3). A mixing plant, storage tank and pump prepare, store, and deliver the solution to be injected.

Procedure: For fracture grouting beneath existing structures, large diameter shafts (3 to 4.6 m, in diameter) or pits are constructed adjacent to the exterior of the structure to be controlled. From these shafts, a drill rig installs the sleeve port pipes horizontally beneath the structure. Then a grout injection tube is inserted into the sleeve port pipe. Packers on the injection tube are inflated on either side of an individual port and grout is injected. The packers are then deflated, the injection tube moved to another port,

and the process repeated as necessary to achieve either the desired heave or prevent settlement. A level surveying system provides information on the response of the ground and overlying structure which is used to determine the location and quantity of the grout to be injected.

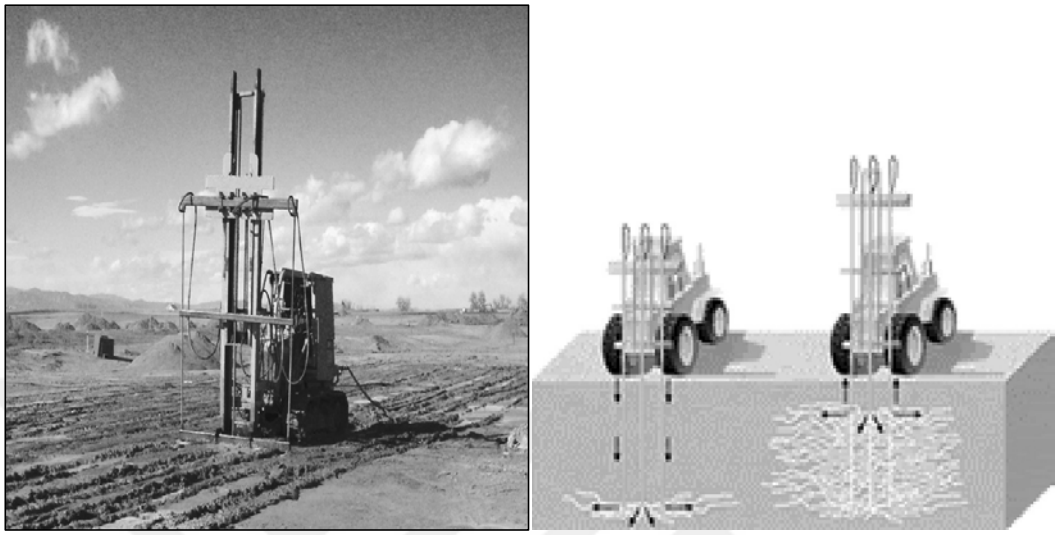


Figure 2.3 : Injection rig for treatment of expansive soils (Hayward Baker Inc.).

For injection of expansive soils, multiple injection rods are typically pushed into the ground to the desired treatment depth (typically 2.2 to 3.7 m) and then an aqueous solution is injected as the rods are extracted.

Materials: For fracture grouting beneath structures, the grout typically consists of Portland cement and water.

For injection of expansive soils, the following solutions have been used:

Water : Used to swell expansive clays as much as possible prior to construction.

Lime and fly ash : Used to fill the desiccation pattern of cracks, reducing the avenues of moisture change.

Potassium chloride and ammonium lignosulfonate : Used to chemically treat the clay and reduce its affinity for water.

Design: For fracture grouting beneath a structure, the design involves identifying the strata which has or will result in settlement, and placing the injection pipes between the shallowest stratum and the structure. For injection of expansive soils, the design includes identifying the lateral and vertical extents of the soils requiring treatment.

Quality control and quality assurance: For fracture grouting beneath existing structures, it is critical to know where all the injection ports are located, both horizontally and vertically. The monitoring of the overlying structure is then critical so that the affected portion of the structure is accurately identified and the injection is performed in the correct ports.

For injection of expansive soil, acceptance is typically based on increasing the in situ moisture content to the plastic limit and 2 to 3 moisture points, reducing pocket penetrometer readings to 288 kPa or less, and reducing the average swell to 1% or less within the treatment zone (Gunaratne, 2006).

2.1.3 Rammed aggregate piers

As a brief explanation the rammed aggregate piers (RAPs) are a type of stone column. Aggregate columns installed by compacting successive lifts of aggregate material in a preaugered hold (Figure 2.4). The predrilled holes, which typically have diameters of 0.6 to 1.2 m, can extend up to about 6 m. As seen in Figure 2.5, aggregate is compacted in lifts with a beveled tamper to create passive soil pressure conditions both at the bottom and the sides of the piers. RAPs are generally restricted to cohesive soils in which a predrill hole will stay open. Although constructed differently than store columns all provide similar improvement to cohesive soils.

Figure 2.4 : Installation of rammed aggregate piers, a type of stone column (The Foundation Engineering Handbook).

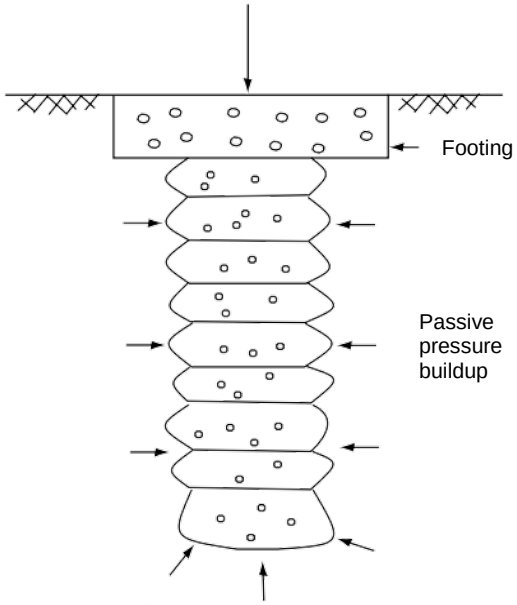


Figure 2.5 : Schematic diagram of a rammed aggregate pier (The Foundation Engineering Handbook).

RAPs can be used in some of the following stone column applications that are outlined below (Gunaratne, 2006):

1. Support shallow footings in soft ground.
2. Reinforces soils to reduce earthquake-induced settlements, however, does not densify sands against liquefaction.
3. Increase drainage and consequently expedite long-term settlement in saturated fine-grained soils.
4. Increase global stability and bearing capacity of retaining walls in soft ground.
5. Improve stability of slopes if RAPs can be installed to intersect potential shear failure planes.
6. Reduce the need for steel reinforcements when RAPs are installed below concrete mat or raft foundations.

2.1.4 Soil mixing

Soil mixing mechanically mixes soil with a binder to create in situ geometries of cemented soil. Mixing with a cement slurry was originally developed for environmental applications; however, advancements have reduced the costs to where the process is used for many general civil works, such as in situ walls, excavation

support, port development on soft sites, tunneling support, and foundation support. Mixing with dry lime and cement was developed in the Scandinavian countries to treat very wet and soft marine clays. In this method the soil can be stabilized either by forming columns of stabilized soil (so-called column stabilization) or by stabilizing the entire soil volume (so-called mass stabilization). However, the two methods may well be combined as shown in the example, figure 2.6. With existing equipment, the soil can be stabilized to a depth of about 31 meters when using column stabilization whereas mass stabilization can be used to a depth of about 5 meters.

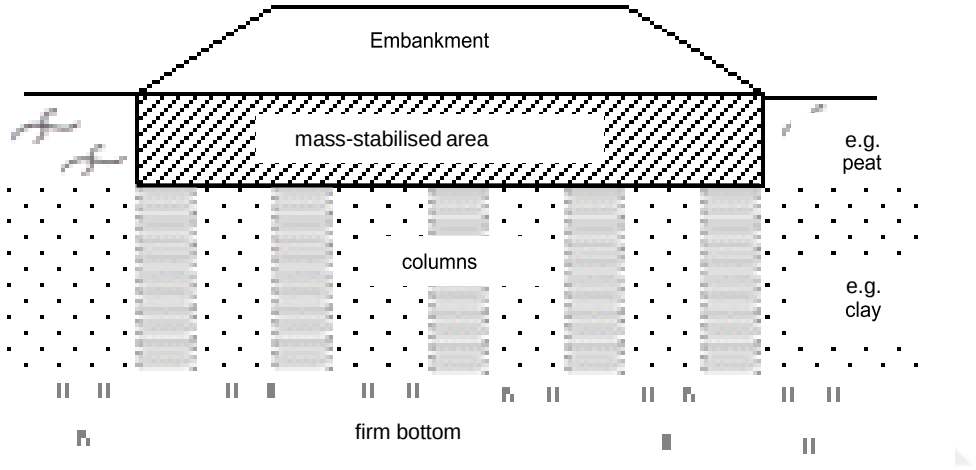


Figure 2.6 : The schema of a structure combining mass and column stabilization (EuroSoilStab, 2002).

When compared with other methods, this technique have some advantages, which are:

- Economic
- Flexibility
- Savings of materials and energy
- Rapidity
- Can be flexibly linked with other structures and with the surroundings (no harmful settlement differences)
- Flexible improved engineering properties of the soil

In figure 2.7 soil improvement using deep stabilization and some alternative methods are compared and their relative merits and drawbacks are listed.

Applicable soil types: The system is most applicable in soft soils. Boulders and other obstructions can be a problem. Cohesionless soils are easier to mix than cohesive soils.

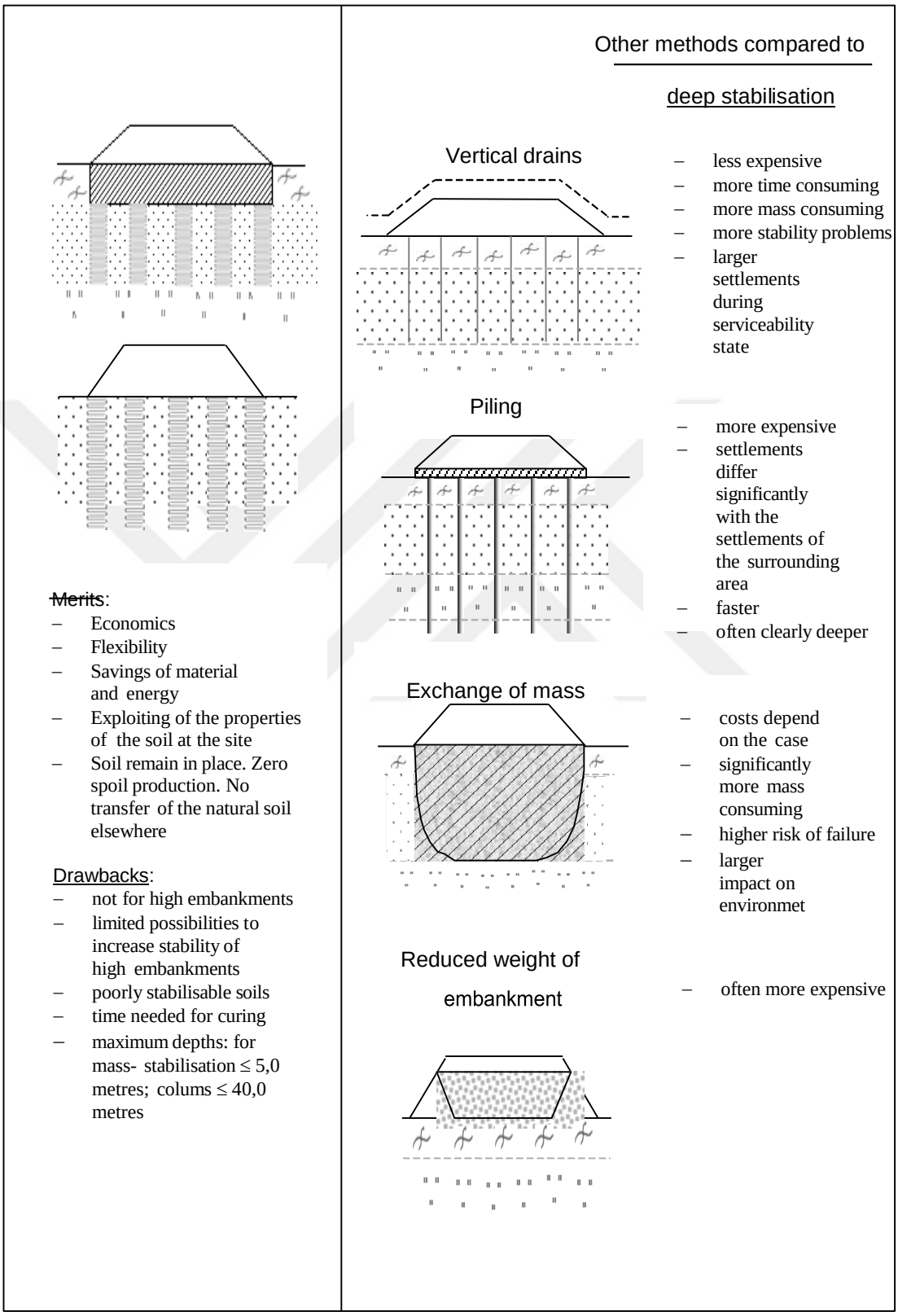
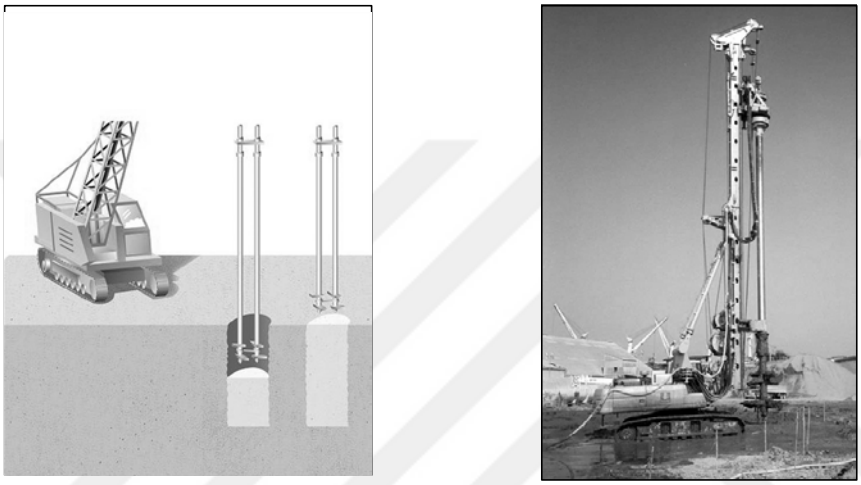


Figure 2.7 : Deep stabilization compared with some other methods (EuroSoilStab, 2002).

A drilling system is required to turn the mixing tool in the ground. The system varies from conventional hydraulic drill heads to dual-motor, crane-mounted turntables with torque requirements ranging from 41 to 411 kJ. Multiaxis, electrically powered drill heads are also used, primarily for walling applications.

The mixing tool is generally a combination of partial flighting, mix blades, injection ports and nozzles, and shear blades. It can be a single- or multiple-axis tool (Figure 2.8). Tool designs vary with soil types and are often custom-built for specific projects (Figure 2.9). The diameter of the tool can vary from 0.46 to 3.7 m.



(a) (b)

Figure 2.8 : Soil Mixing: (a) Schematic, (b) Field implementation (Hayward Baker Inc.).

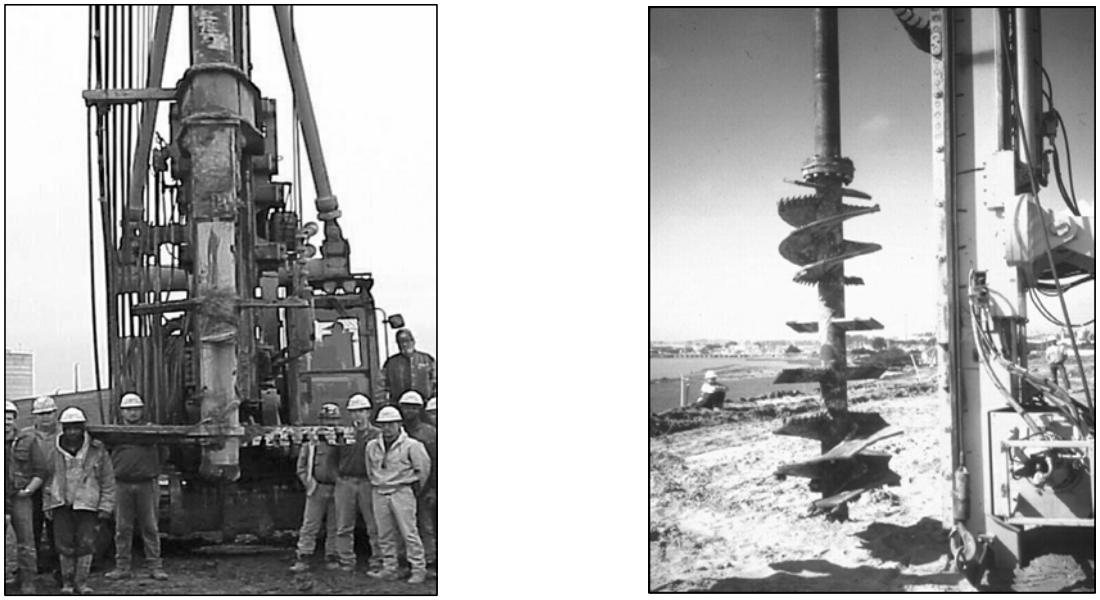


Figure 2.9 : Example of soil mixing tools (Hayward Baker Inc.).

Procedure: The binder is injected as the tool is advanced down to assist in penetration and to take advantage of this initial mixing. The soil and binder are mixed a second time as the tool is extracted. The rate of penetration and extraction is controlled to achieve adequate mixing. Single columns or integrated walls are created as the augers are worked in overlapping configurations. Treatment depths as great as 31 m have been achieved.

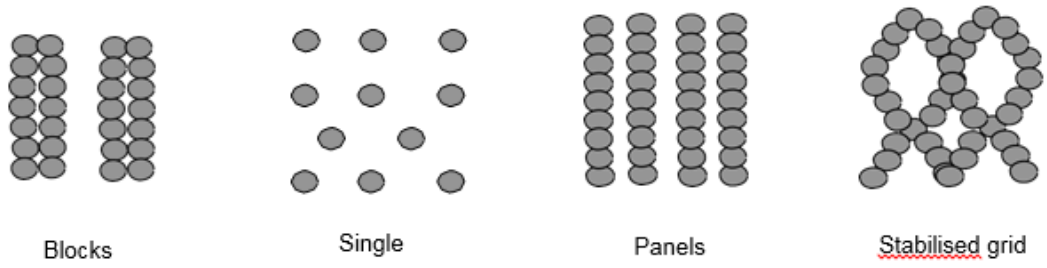
Materials: For wet soil mixing, the binder is delivered in a slurry form. Slurry volumes range from 20 to 40% of the soil volume being mixed. Common binders are Portland cement, fly ash, ground blast furnace slag, and additives. For dry soil mixing, the same materials (also line) are pumped dry using compressed air. Preproduction laboratory testing is used to determine mix energy and grout proportions.

Design: As with jet grouting, unconfined compressive strength and permeability are generally the design parameters. A standard analysis is performed to determine the required geometry based on the parameters achievable in the soil to be mixed. For excavation support walls, the mass can be designed as a standard excavation wall, or a thicker mass can be created and analyzed as a gravity structure, calculating the mass' shear, sliding and overturning, as well as the global stability of the system. When used as structural load-bearing columns, a standard bearing capacity and settlement analysis is performed as would be for any cast in place pier. Anchored retention using steel reinforcement is common for support walls. Furthermore, the mass and column stabilization can be applied in many different ways. Figure 2.10 gives some examples of the configuration of columns.

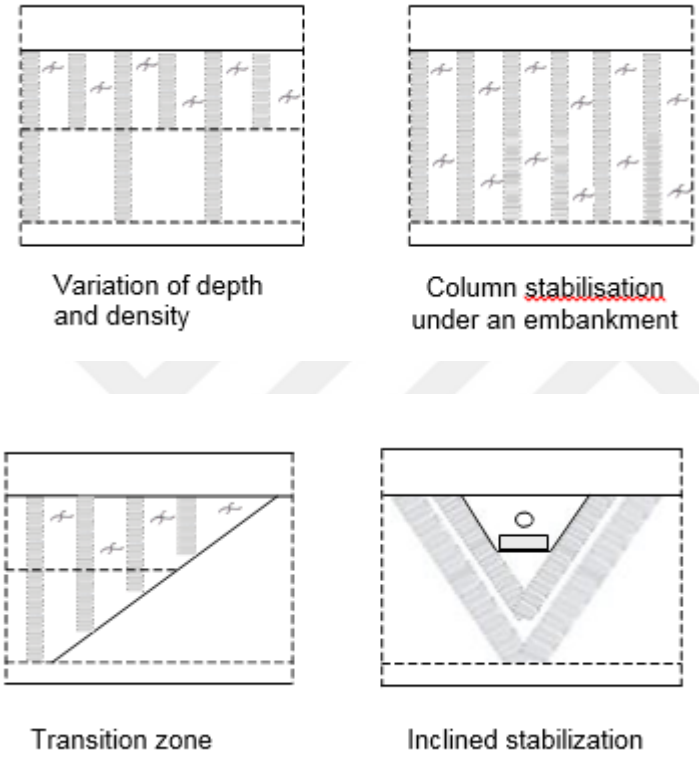
Moreover, The sequence of mixing for the deep column mixing will need to be adjusted to suit each specific site conditions but in general the most efficient sequence is to work the stabilization machine within its radius of operation as much as possible before it is moved. Most machines will have a limited angle of slew for maximum stability while mixing. A typical sequence for deep mixing in columns is shown in Figure 2.11(EuroSoilStab, 2002; Gunaratne, 2006).

2.1.4.1 Dry soil mixing

Dry soil mixing (Figure 2.12) is a low-vibration, quiet, clean form of ground treatment technique that is often used in very soft and wet soil conditions and has the advantage



(a) Examples of the placing of columns.



(b) Examples of placing of columns

Figure 2.10 : Examples of configurations for column stabilization (EuroSoilStab, 2002).

of producing very little spoil. The high speed rotating mixing tool is advance to the maximum depth, ‘‘disturbing’’ the soil on the way down. The dry binder is then pumped with air through the hollow stem as the tool is rotated on extraction. It is very effective in soft clays and peats. Soils with moisture content, greater than 60% are most economically treated. This process uses cementitious binders to create bond among soil particles and thus increases the shear strength and reduces the compressibility of weak soils.

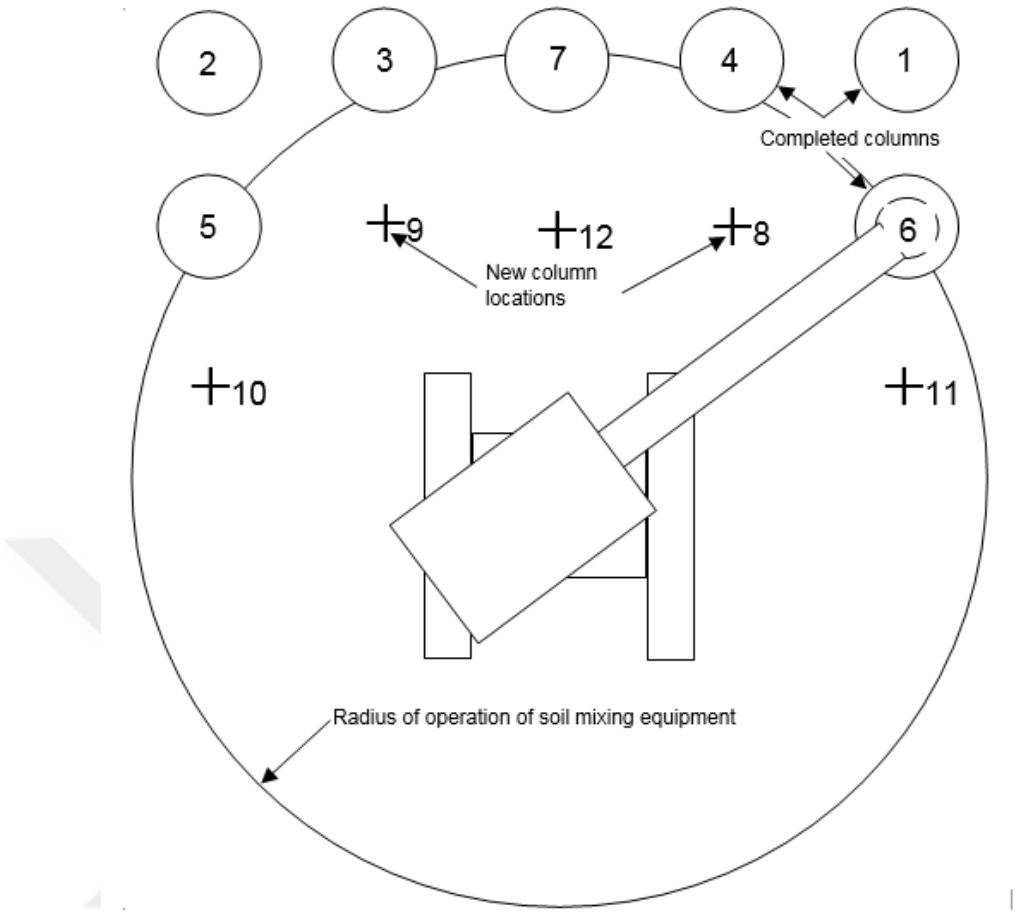


Figure 2.11 : Sequence of construction for deep soil mixed columns (EuroSoilStab, 2002).

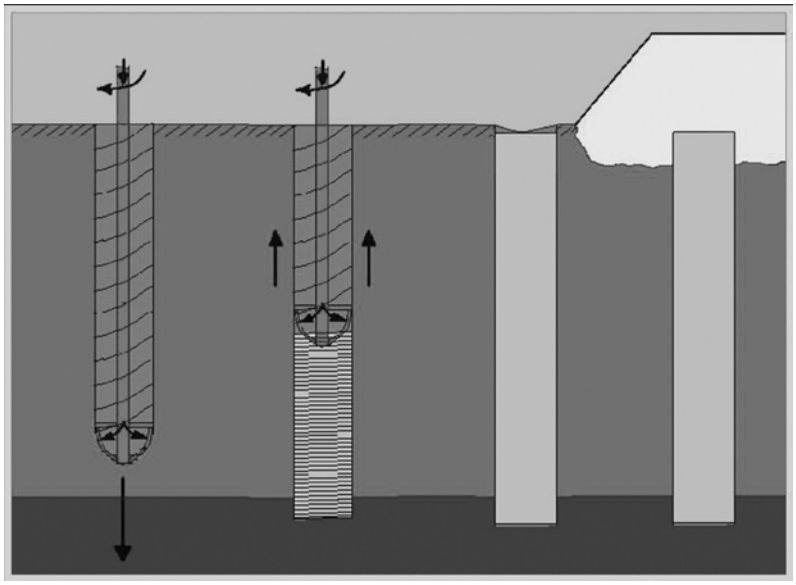


Figure 2.12 : Illustration of dry soil mixing technique (Hayward Baker Inc.).

The most commonly used binding agents are cement, lime, gypsum, or slag. Generally, the improvement in shear strength and compressibility increases with the binder dosage. By using innovative mixtures of different binders engineers usually achieve improved results. It is known that strength gains are optimum for inorganic soils. It is realized that the strength gain would decrease with increasing organic and water content. The binder content varies from about 80 kg/m^3 for soft inorganic clays to about 288 kg/m^3 for peats with a high organic content. Additionally the different tools of dry mixing machine are shown in figure 2.13 and 2.14 (EuroSoilStab, 2002; Gunaratne, 2006).

Figure 2.13 : Three versions of the Nordic dry mixing ―standard tool (Larsson, 2005).

Figure 2.14 : Injection of dry binder into the soil from the mixing tool (Keller Inc.).

2.1.4.2 Wet soil mixing

Wet soil mixing (Figure 2.15) is a similar technique except that a slurry binder is used making it more applicable with dryer soils (moisture contents less than 60%). Applications of wet deep mixing involve binder turned into slurry form, which is then injected into the soil through the nozzles located at the end of the soil auger(Massarsch & Topolnicki, 2005). Depending on the in situ soils, the volume of grout slurry necessary varies from 20 to 40% of the soil volume.

The technique produces a similar amount of soil (20 to 40%) which is essentially excess mixed soil which, after setting up, can often be used as structural fill. The grout slurry can be composed of Portland cement, fly ash, and ground granulated blast furnace slag. Quality control and quality assurance: Preproduction laboratory testing is often performed to prescribe the mixing energy and binder components and proportions. During production, it is necessary to monitor and document parameters such as mixing depth, mixing time, grout mix details, grout injection rates, volumes and pressures, tool rotation, penetration, and withdrawal rates.

Test cylinders or cubes can be cast from wet samples, but are problematic. The hardened columns can also be cored. In weaker mixes, penetration tests can be performed. Additionally the wet mixing machine and different parts of it are shown in figure 2.16 and 2.17(Gunaratne, 2006).

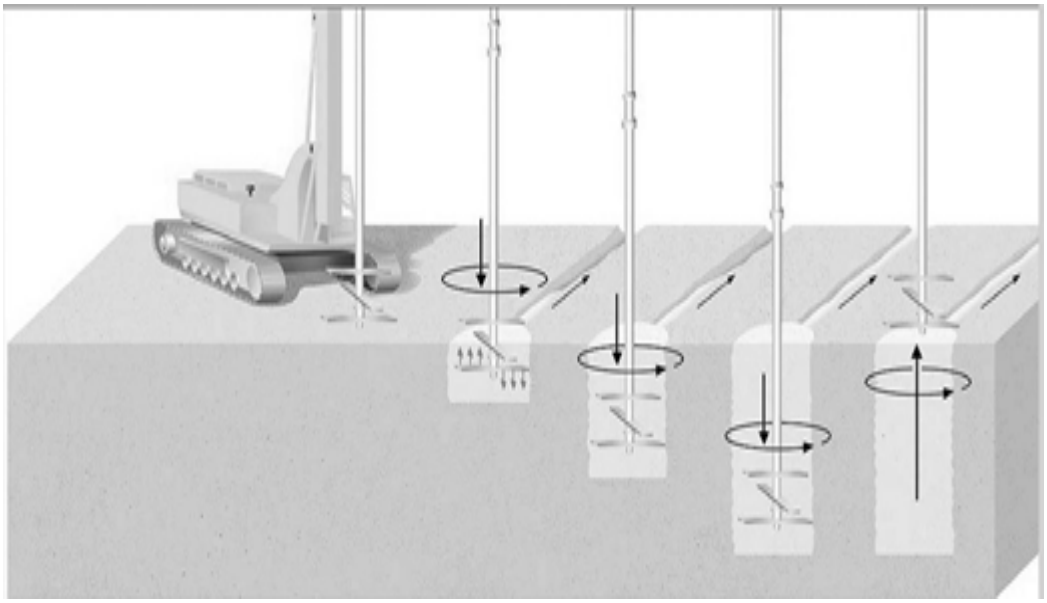


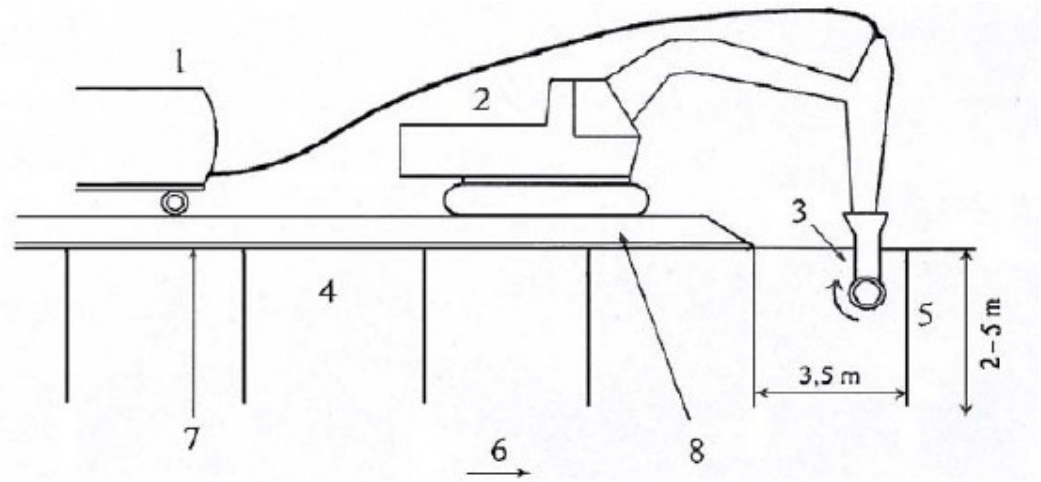
Figure 2.15 : Illustration of wet soil mixing technique (Hayward Baker Inc.).

Figure 2.16 : Parts of wet mixing tool showing injection of slurry into the soil (Porbaha et al, 2005).

Figure 2.17 : Bauer cutter soil mixing. (Fiorotto et al, 2005).

2.1.5 Mass stabilization

Mass stabilization is a shallow to deep stabilization method in which the entire volume of soft soil can be stabilized to a prescribed depth (Figure 2.18-19). The technique is relatively new and is highly suited for the stabilization of high moisture content such as clay, silty, organic soils and contaminated sediments (EuroSoilStab, 2002; Hayward Baker Inc., 2012).



Key features: 1. Stabilizer tank and scales; 2. Execution machine; 3. Mixing tools 4.Stabilized mass of soft soil; 5. Unstabilized soft soil; 6. Direction of mass stabilization; 7. Geotextile (Reinforcement); 8. Preloading embankment

Figure 2.18 : Schematic diagram of mass stabilization (Massarsch and Topolnicki, 2005; EuroSoilStab, 2002).

Figure 2.19 : Dry mass soil mixing to strengthen soft soils beneath a planned roadway expansion at U.S. Highway 1, Key Largo, Florida (Hayward Baker Inc).

Mass stabilization offers a cost effective solution to ground improvement in situ remediation especially with a huge amount of contaminants and high water content. Remediation of most deposits of contaminated dredged sediments, organic soils and

waste sludge usually make by using mass stabilization method (Keller Inc., 2011). The method provides an alternative to traditional method of soil improvement such as removal and replaces techniques. The blending of the soil mass is achieved by use of excavator mounted mixing tool with unique shuttles pneumatically delivering the binder to the head of the mixing tool and into the mix zone. The mixer rotates and simultaneously moves vertically and horizontally while mixing the soil block. The diameter of mixing tool normally lies between 600 mm to 800 mm, with rotation speed between 80 and 100 rpm. Usually, the soil is stabilized in a sequence of a block which is defined as the operating range of the machine. The typical range correspond to 8 to $10m^2$ in plan and 2 to 5 m in depth (i.e. 2 m wide x 5 m long x 3.5 m deep) with production rate between 200 and $300m^3$ of stabilized soft soil per shift. The amount of binder is typically in the range of 200 to $400kg/m^3$(EuroSoilStab, 2002).

In Nordic countries the amount of binder is in a typical range of 150 and 250 kg/m^3, and the targeted shear strength is 50 kPa (Massarsch & Topolnicki, 2005). The method has advanced to include use of rapid cement as a binder in stabilization of contaminated dredged material at Port Hamina and shoreline of Helsinki, Finland, where stabilized contaminated dredged materials deposited between embankments created new areas (Andersson et al, 2001). Prior to initial set of the stabilized mass, a geo-membrane separator have to be placed on top of stabilized soil on which a selected granular base course material lies. These fill materials compresses the freshly stabilized mass forcing out all air pocket that may have formed during mixing (EuroSoilStab, 2002; Hayward Baker Inc., 2012; Massarsch & Topolnicki, 2005).

2.2 Stabilization of Clays

Wide research has been completed relating to the use of traditional stabilizers, namely lime, cement, asphalt, and fly ash. The stabilization mechanisms for them are well documented, and the effectiveness of these traditional stabilizers has been demonstrated in many applications. However, relatively little research documenting the use of nontraditional stabilizers such as Nano-Materials and Nano-Material based Polymers is available, and their performance record is varied. Although promotional material exists attesting to the effectiveness of nontraditional stabilizers, such materials often lack documentation of measured engineering properties. This literature

review focuses on the known properties of both traditional and nontraditional stabilizers, as relevant to this research.

2.2.1 Clays

Clay is one of the oldest building materials on the planet. Clay is defined as a fine grained earthen material which contains clay minerals, and is plastic and cohesive (Obeng and Atiemo , 2005). Clays shrink when dry and expand when wet and gain in strength with retention of shape on firing. Physically, clays have particle range of 2µm and below (BS 1377: 1990). According (Rhodes, 1973) the most important source of rocks for clay formation is the granite, which have mixtures of feldspar, quartz and mica. Feldspar is by far the most common mineral, and it is the decomposition of this material, which largely accounts for clay formation. This decomposition process of feldspar is continuous and goes on everywhere; hence, clay is an extremely common and abundant material in nature.

Two main types of clay are: residual and sedimentary clay. Residual clays are those clays that have not been transported by natural agencies and are found to be where they were formed. These types are relatively pure and lack plasticity, have low strength in their dry state. Sedimentary clays, by contrast, are those which have been removed from their origin by natural agencies. They are rarely obtained pure, because many impurities are picked up and retained during transportation. The fine-grained nature of many such impurities makes them difficult and uneconomic to remove (Worrall, 1986). Elutriation process during transportation results in the attainment of plasticity, strength and color. The moving clay comes into contact with various materials, minerals and oxides giving rise to its physical and chemical properties (Rado, 1988).

Clays vary in both chemical and physical properties. The variation in clay properties is dependent on the geology, mineralogy and chemical composition of the parent material. A particular clay type is chosen depending on the type of work to be done. Clay to be used in the manufacture of structural clay products (bricks, tiles etc.) should be sufficiently plastic for satisfactory shaping, should not shrink excessively on drying and vitrify without excessive shrinkage at its maturing temperature.

2.2.1.1 Clay minerals

There are many different types of clay minerals, each with unique chemical and behavioral properties which arise from the structure of the clay minerals. Clay minerals by definition refer to phyllosilicate minerals and to minerals which impart plasticity to clay and which harden upon drying or firing (Guggenheim & Martin, 1995). However, nearly all clays contain just two basic components that occur in different arrangements. These two basic building blocks of all clay minerals are the silica tetrahedral and the aluminum octahedral that are shown in Figures 2.20. Kaolinite and montmorillonite are the types of clay minerals that play important role in their response to the chemical stabilizers used in clay stabilization. Kaolinite clay mineral responds better to chemical stabilizers than montmorillonite. As a result, the characteristics and behavior of the two main clay mineral types have been reviewed in this thesis (Gogo, 1985; Little, 1999).

When scientists talk about 1:1 or 2:1 clays, they refer to the ratio of silica tetrahedral sheets to aluminum octahedral sheets. A ratio of 1:1 clay has one of each sheet and a ratio of 2:1 clays have two tetrahedral sheets on either side of an aluminum octahedral sheet. These tetrahedral and octahedral sheets are variously arranged and modified during mineral formation to create different types of clay minerals.

According to Barak et al. (2003), kaolinite is one of the clay minerals with a ratio of 1:1 clay minerals. It does not shrink when dry or swell when wet, which makes it well suited for uses such as construction of roads and buildings, for septic adsorption fields, and pottery. The arrangement of closely packed sheets in kaolinite link to that of a closed book result in a much less external surface area than other clay minerals. No internal surface area and less capacity for holding water and cations. The ratios of 2:1 clay minerals look much different (Figure 2.20). Using X-ray diffraction analysis, montmorillonite looks like a sponge. The larger interlayer spaces in the 2:1 clay minerals have the capacity to hold water molecules and a variety of cations (some of which, like Na^+, cause the clay to disperse) with important advantages for plant growth. Also, with larger interlayer spaces comes a greater tendency for shrink/swell behavior (not all 2:1 clays expand). If a clay swells when wet, it is poorly suited for building site development or for septic leach fields. However, these clays are excellent for sewage lagoons or wildlife ponds; if they remain wet, they "seal" and hold water.

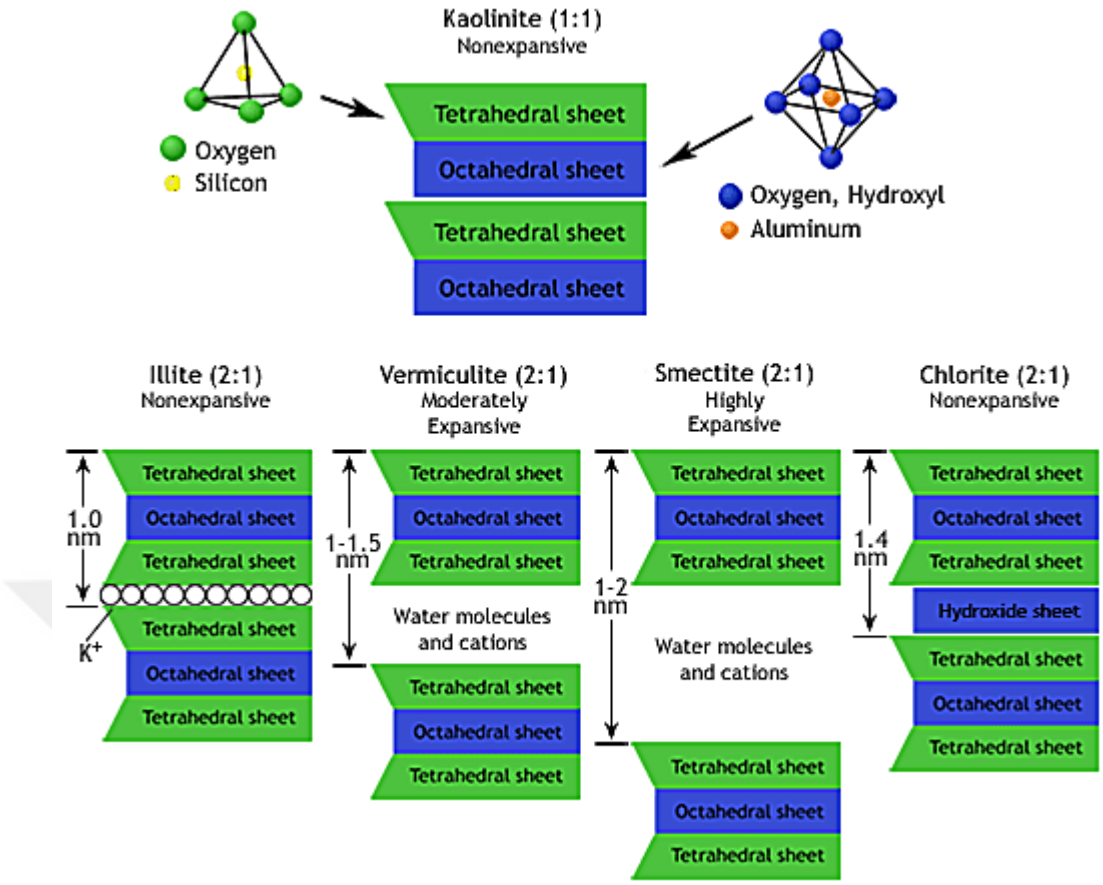


Figure 2.20 : Pictorial diagrams of various types of clay minerals (Barak and Natar, 2003).

Table 2.1 presents a summary of the difference in kaolinite and montmorillonite clay minerals.

2.2.2 Lime stabilization

Laboratory testing shows that lime reacts with medium, moderately fine, and fine-grained soils to produce decreased plasticity, increased workability, and increased strength (Little, 1995).

Strength gain is primarily due to the chemical reactions that occur between the lime and soil particles. These chemical reactions occur in two phases, with both immediate and long-term benefits. The first phase of the chemical reaction involves immediate changes in soil texture and soil properties caused by cation exchange. The free calcium of the lime exchanges with the adsorbed cations of the clay mineral, resulting in reduction in size of the diffused water layer surrounding the clay particles.

Table 2.1 : Summary of the differences in kaolinite and montmorillonite clay minerals (Barak and Natar, 2003).

Clay Minerals Specs	Group1	Group2
Characteristic	Kaolinite	Montmorillonite
Layer type	1:1	2:1
Typical chemical formula	$[Si_4]\ Al_4O_{10}(OH)_8$	$[Si_8]Al_{3.2}Fe_{0.2}Mg_{0.6}O_{20}(OH)_4$
Particle size (μm)	0.5 – 5.0	0.01 – 1.0
Specific Surface area (m^2/g)	7 – 30	600 - 800
Shrink/swell potential	non-expansive	highly expansive
Interlayer space	none (very small)	very large
Cation Exchange Capacity ($mmol_c/kg$)	2 - 15	80 – 150

This reduction in the diffused water layer allows the clay particles to come into closer contact with one another, causing flocculation/agglomeration of the clay particles, which transforms the clay into a more silt-like or sand-like material. Overall, the flocculation and agglomeration phase of lime stabilization results in a soil that is more readily mixable, workable, and, ultimately, compactable. Practically all fine-grained soils undergo this rapid cation exchange and flocculation/agglomeration reactions when treated with lime in the presence of water (Eades and Grim, 1960).

The second phase of the chemical reaction involves pozzolanic reactions within the lime-soil mixture, resulting in strength gain over time. When lime is combined with a clay soil, the pH of the pore water increases. When the pH reaches 12.4, the silica and alumina from the clay become soluble and are released from the clay mineral. In turn, the released silica and alumina react with the calcium from the lime to form cement, which strengthens in a gradual process that continues for several years (Eades and Grim, 1960). As long as there is sufficient calcium from the lime to combine with the soluble silica and alumina, the pozzolanic reaction will continue as long as the pH remains high enough to maintain the solubility of the silica and alumina (Little, 1995). According to Lees et al. (1982), strength gain also largely depends on the amount of silica and alumina available from the clay itself; thus, it has been found that lime stabilization is more effective for montmorillonitic soils than for kaolinitic soils.

In addition to pozzolanic reactions, carbonation can also lead to long-term strength increases for soils stabilized with lime. Carbonation occurs when lime reacts with carbon dioxide from the atmosphere to produce a relatively insoluble calcium

carbonate. This can be advantageous since after mixing, the slow process of carbonation and formation of cementitious products can lead to long-term strength increases (Arman and Munfakh, 1970). However, prior to mixing, exposure of lime to air should be avoided through proper handling methods and expedited construction procedures in order to avoid premature carbonation of the lime (Chou, 1987).

Through time, scientist invent other type of lime stabilization by adding other additives to the lime that will briefly explain below.

2.2.2.1 Lime - sodium hydroxide stabilization

Davidson et al. (1959), argue that the addition of small amount of sodium hydroxide (NaOH) to some clays activates stabilizing action of lime on these clays. The sodium hydroxide (NaOH) reacts with siliceous material to produce sodium silicate. The sodium silicate subsequently reacts with lime ($Ca(OH)_2$) to form sodium hydroxide (NaOH) and cementitious insoluble calcium silicates. The amount of (OH) ions increased due to the presence of NaOH. These accelerate the pozzolanic reactions by increasing the solubility of the siliceous material in the clay.

2.2.2.2 Lime - sodium silicate stabilization

The addition only sodium silicate to clay may negatively affect soil stabilization (Ding et al., 1996). Clay particles typically have a net negative charge on their face and a positive charge along the edges because of broken bonds. When sodium silicate is added to clay, the negative silicate ions from the sodium silicate are attracted and attached to clay particles edges, causing entire clay particles to become negatively charged. If the entire clay particles have a negative charge, they will repel one another and the clay structure will become dispersed and weak (Rafalko et al, 2006). Although sodium silicate may weaken clay when added alone, it may strengthen clay if lime is added along with the sodium silicate. The lime can be as a source of calcium ions and with the presence of both calcium ions and silicate ions, calcium silicate gel can be form, hydrate and harden thereby cementing the clay particles together (Rafalko et al, 2007).

2.2.2.3 Lime - salt (NaCl) stabilization

Gueddouda et al. (2011), evaluated the stabilization effect of salt, lime, cement, combinations of lime and cement, and combinations of lime and salt on the swelling potential of three Algeria expansive soils where several cases were reported disorders characterized by cracks in the superstructure and the foundation level. Among the encouraging results obtained from the additives added, cement additions produces similar results to that of lime. The combination of lime and cement also exhibited result similar to those of lime or cement alone. Nevertheless, with the Stabilization of lime-salt, the result is better than the combined lime-cement stabilization. Gueddouda et al. (2011), recommended the use of (lime - salt) as an alternative economical and effective for the treatment of swelling clays.

Yunus et al. 2012, did a similar work on organic clay using lime and chloride salts ($CaCl_2$ and NaCl). Scanning Electron Microscope (SEM) images of lime-treated organic clay show the flocculated and aggregated structure, without appearance of cementation. In contrast, a clear evidence of cemented structure is observed when lime-treated organic samples are stabilized with the addition of 0.5% $CaCl_2$ and 0.5% NaCl.

2.2.2.4 Lime - silica stabilization

Silica addition appears to significantly improve the reactivity potential of clay bearing soils with lime used in traditional lime stabilization techniques. The addition of silica in the concentration is effective to promote and speed up the formation of calcium silicate hydrates over the formation of calcium aluminate hydrates in the resulting pozzolanic reaction occurring in the bearing soils (McKennon et al, 1994).

2.2.3 Cement stabilization

Introduction of cement into soft ground, or cement-soil stabilization, either in the form of dry cement powder or slurry cement, is a popular method of ground improvement technique. The inclusion of cement into soil-water systems causes physicochemical changes at a microstructural level and therefore mechanical behavior of the treated soil at a macroscopic level. The short-term gain in strength is the result of primary hydration reaction, which also leads to a reduction in moisture content during the chemical reaction. This process forms two cementing minerals, namely Calcium

Silicate Hydrates (CSH) and Calcium Aluminate Silicate Hydrates (CASH). At the same time, the release of lime into the inter-particle voids leads to the formation of a flocculated structure. Subsequent long term gain in strength is a result of secondary pozzolanic reaction between the lime and the clay minerals (Bergado et al., 1996; Kezdi, 1979).

Over the years, cement stabilization has been developed from surface treatment (such as for road pavement) and extended significantly to a greater depth, wherein cement columns are created through deep mixing. In this method, specially designed machines with several shafts equipped with mixing blades and stabilizer injection nozzles are used to construct in-situ treated soil columns in various patterns and configurations. The use of the Deep Mixing Method (DMM) was probably started sometime in the early to mid-1970s. As DMM is implemented using cement slurry, it is often termed Cement Deep Mixing (CDM) (Porbaha, 1998). Since then, the equipment used has improved and the application of deep mixing as a ground improvement method has been extended throughout the world.

2.2.4 Flyash stabilization

Fly ash is a by-product of coal combustion in power plants. Fly ash contains silica, alumina, and calcium oxides, iron oxide and alkalis in its composition, and is considered as a pozzolanic material (Porbaha, 1998). The most common elemental compositions of fly ash include amorphous oxide (mainly SiO2, Al2O3), and metal oxides i.e. TiO2, Fe2O3, MnO, MgO, CaO, Na2O, K2O, P2O5, SO3 and organic carbons. A guideline for selecting fly ash as soil stabilizing agent is provided in ASTM C593.

There are two types of fly ash; type “C” and type “F”. This classification is based on the chemical composition. Fly ash type “C” contains 10% to 16% amount of free lime (Cockrell & Leonard, 1970).

This type of fly ash produces pozzolanic and cementitious reactions. Cockrell et. al. (1970), publicized that, color is one of the important physical properties of fly ash in terms of estimating the lime content qualitatively. Lighter color of fly ash indicates the presence of high calcium oxide and darker colors of fly ash represent high organic content. Fly ash can be used to improve the engineering properties of soil. However,

it must be well-known that fly ash properties are highly variable and depend on chemical composition of coal and combustion technology.

2.2.5 Stabilization using nano-material based liquid polymers

Liquid-formed Nano-Material based chemicals such as CBR PLUS (Con-Aid) and Zycosil among other Liquid-formed chemicals like Choline chloride, Choline bicarbonate and Potassium chloride have been used as additives or stabilizers on improvement of engineering properties of soils especially clay soils instead of the prevalent solid chemical additives such as lime, cement or fly ash among others (Ali, 2012; Ou et al., 2011). These liquid chemically are non-toxic and environmentally friendly. Based on the results obtained in the used of these liquid stabilizers, the following conclusions can be made on the performances of these liquid stabilizers:

- It reduces plasticity and shrinkage by eliminating re-absorption of water molecules.
- It reduces optimum moisture content by ionizing and exchanging the water molecules on the surface of the clay platelets.
- It increases maximum dry density by neutralizing and orderly re-arranging the clay platelets.
- It increases the compressive strength by increasing the inter particles bonding. Besides the encouraging results obtained with good performance on the liquid stabilizers, they are not really available locally.

The theory behind of using these chemicals comes from the problems that poor rural roads are suffering from. Rural roads in general are of unacceptably low standards, which prohibited any form of transport during wet rainy conditions (Lyatuu et.al, 2000). Unsurfaced roads, particularly those that do not have any gravel wearing course are extremely vulnerable to wet conditions, but the placing of gravel on all roads is quite high-priced even if sufficient gravel was universally available. The use liquid-formed Nano-Material based chemicals now becomes a matter, which deserves serious consideration. These products which are soluble adsorbed water displacing ionic additives (SAWDIA) can and do make a poor quality in-situ soil resist the effects of rain so as to enable the road to be used at all times by light traffic without the hauling in and placing of a gravel layer which make them a cheaper solution.

2.2.5.1 Nano-polymer CBR PLUS

Nowadays, plenty types of Nano-Polymer composites are used for soil stabilization in engineering operations. CBR PLUS is a unique cation reactive organic compound, which forms protective, oily layers on the surface of soil and clay particles. It reduces ion mobility and ion exchange and simultaneously makes the clay material hydrophobic by eliminating the adsorption of water. The result is a soil material that is much less sensitive to moisture, more workable and is able to be compacted to a better particle-interlock state by equipment and traffic forces (Figure 2.21).

CBR PLUS is a concentrated liquid, of which 100 liters consolidate between 10,000 and 20,000 square meters of soil to a depth of 15 cm when diluted with water. It will treat a wide spectrum of materials ranging from clays and silty sands to gravel, but the effect of it on clay materials is permanent and the other soil types must display some cohesive properties. However, CBR PLUS has been successfully used on gravel roads to reduce maintenance costs (Petry & Little, 2002). Taherkhani et al. (2012) used CBR PLUS for improving physical and mechanical properties of different combinations of clay, sand and gravel. They concluded that the CBR PLUS decreases the plasticity index and permeability and increases the compressive strength and CBR of the combinations. The combination of 50% gravel, 30% sand and 20% of clay is found to attain the highest strength with the stabilizer (Piratheepan et al., 2010).

CBR PLUS allows engineers to use in-situ material in either single or multi-layer pavement construction by reducing the importation of materials from unsightly and environmentally degrading borrow pits; thus minimizing the environmental degradation.

Briefly the advantages of CBR Plus are:

- Construction Aid: Improves workability of soils.
- Increases soil shear strength.
- Consolidation Aid: Increases density/bearing capacity.
- Reduces dust on unpaved roads.

- Reduces maintenance on gravel roads.
- Reduces the need for gravel replacement.
- Reduces construction costs.
- Saves haulage on borrow material.

Figure 2.21 : Effect of CBR Plus before and after of the stabilization (International CON-AID Ltd.).

Beside of these advantages, physico-chemical properties of CBR PLUS are another good aspects of this chemical material which can make it a good choice.

CBR PLUS has the following physico-chemical properties:

- Totally water soluble with no solid residue.
- Non-flammable.
- Non-corrosive.
- Non-toxic and safe.
- Non-hazardous
- User and environment friendly.
- Unlimited shelf life.

At last, CBR PLUS has been extensively used in many countries to overcome the problems of using marginal quality construction material for road pavement layers.

Construction and maintenance costs of CBR PLUS treated roads can save vast amounts of money, meaning that more kilometers of road can be built with the same budget (International CON-AID Ltd.).

Mechanism of CBR Plus-clay reaction:

A clay mineral particle is very small and is made up by a number of sheets which can be compared to that of a book (Figure 2.22).

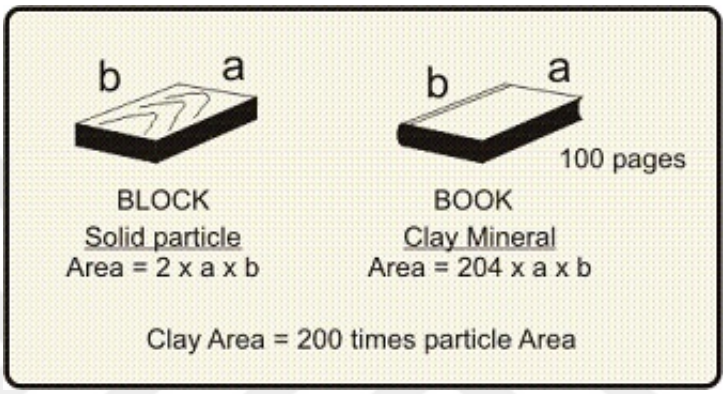


Figure 2.22 : Constituent sheets in a mineral clay (International CON-AID Ltd.).

This "book nature" of the clay gives it an extremely large surface area which attracts metallic ions which themselves attract large quantities of water called adsorbed water. Adsorbed water absorbs further water causing clay to expand and lose stability. The adsorbed water is strongly bonded to the clay surface and is not removed by sun drying or oven drying. It can be removed by chemicals, which cause an ion exchange to take place. CBR Plus does just this in clay soils (Figure 2.23).

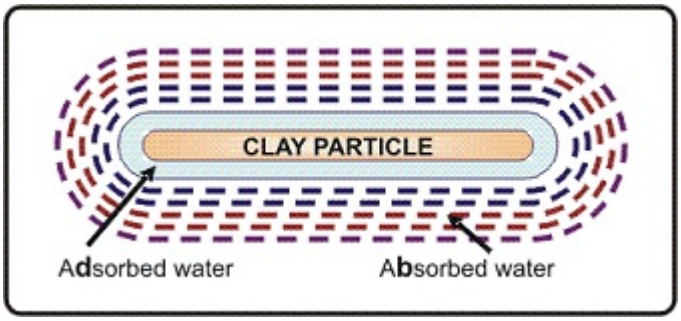


Figure 2.23 : Clay-Water bonding (International CON-AID Ltd.).

CBR Plus ions are complex molecules of two parts, a hydrophilic head and a hydrophobic tail.

These ions attach themselves to the clay surfaces which displace or lock in metallic ions and remove adsorbed water to make the clay hydrophobic and almost oily in effect (Figure 2.24).

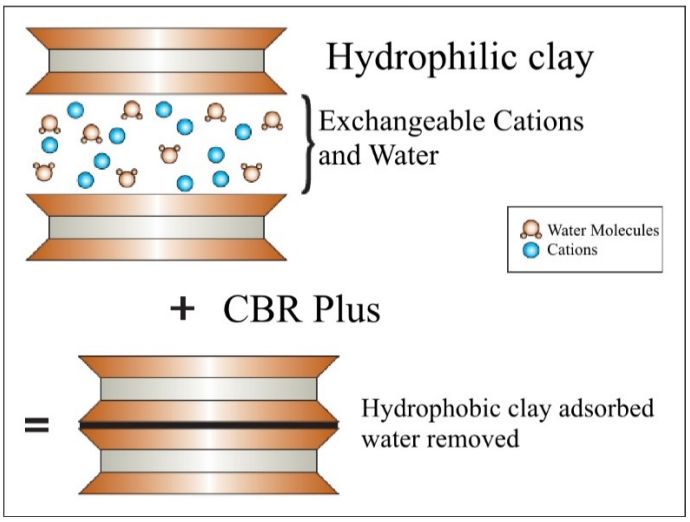


Figure 2.24 : Reaction of clay particle with sulphonated oil (International CON-AID Ltd.).

The process of getting the CBR Plus to all the clay in a soil needs time known as a green or maturation period. Immediately after application, very little improvement in the soil is observed, but at the end of maturation (2 to 4 weeks) the soil becomes hard and strong.

It should be noted that the presence of water is needed for the CBR Plus to migrate to all the clay and a dry spell can retard the process, resulting in a long maturation (International CON-AID Ltd.).

Construction procedure:

The sequence of construction as follows:

a) Clearing:

Remove vegetation, root systems and excessive organic topsoil material where encountered by means of a grader blade. The width, line and curves should be

set out in such a manner that it can be maintained to ensure that construction will be possible within these boundaries.

b) Excavate side drains:

Excavate side drains adjacent to the proposed shoulder on either side of the formation using a grader, back-actor or bulldozer, placing this material to make up the required layer thickness (if the material is suitable for the layer construction) or to produce a formation fill that is at least 0.5 m above the high water flood level, and shaped at 3-5% camber or cross fall for surface drainage.

c) Layer thickness:

When the total specified thickness of the stabilized layer is between 15 and 30 cm, the stabilized layer should be constructed in two layers of equal thickness; when greater than 30 cm, it shall be constructed in three layers of equal thickness.

d) Imported material:

Imported material may be required to provide a loose (bulked) thickness of at least 200 mm if there is insufficient suitable material in position for the layer construction. Furthermore, the material control will be strength (CBR), PI (8-35%), and percentage passing 0.075 mm (15-55%). If these requirements are not met, the imported material should provide the required blending.

e) Loose layer material:

The layer material to be stabilized must be loose (to a depth of at least 200mm) and may have to be scarified by grader to produce this condition. The bulking factor is usually between 25% and 35%.

f) Quantity of CBR PLUS:

The quantity of CBR PLUS to be applied to the stabilized section is obtained by multiplying the given field application rate (laboratory application rate (LR) supplied by headquarter multiplied by % passing the 0.425 mm sieve) (Table 2.2) by the stabilized area (length x width) for 15 cm layer thickness.

Required liters of CBR PLUS is R=L×W× Application rate in liter/m^2

Where: W = Width of road in meters

L = Length of construction section in meters.

R = Application rate of CON-AID in liter /m^2.

Table 2.2 : Suggested Application Rate on Different Soil Types.

Group	Subgroup	% Passing 0.075 mm Sieve	Plasticity Index	CBR PLUS Application Rate (liter/m^2)	Purpose of CBR PLUS Products
A-1	A-1-a	15 max	6 max	0.01	Facilitate compaction Stabilize fines
	A-1-b	25 max	6 max	0.01	
	A-2-4	35 max	10 max	0.01	Facilitate compaction
	A-2-5	35 max	10 max	0.015	Stabilize silt fines
A-2	A-2-6	35 max	11 min	0.015	Stabilize clay fines
	A-2-7	35 max	11 min	0.02	Increase CBR
A-3		10 max	Non-Plastic	N/A	N/A
A-4		36 min	10 max	0.02	Facilitate compaction Stabilize silts
A-5		36 min	10 max	0.025	Stabilize clays
A-6		36 min	11 min	0.02 – 0.03	Stabilize silty clays
A-7	A-7-5	36 min	11 min	0.02 - 0.03	Increase CBR
	A-7-6	36 min	11 min	0.02 - 0.03	Improve Workability
A-8				N/A	N/A

g) Quantity of water:

The percentage water required will be the difference between the optimum moisture content (OMC) and the in-situ moisture content on site. The quantity of water will thus be:

L × W × T (thickness - 15 cm) × D (required density of 2 ton/m3, say) × MC (Difference between OMC and in situ MC) = Quantity (liters). Divide this

quantity by tanker load capacity to establish the number of water tanker loads required.

h) Quantity of CBR PLUS per tanker:

Divide the total quantity of CBR PLUS required by the number of tanker loads required to yield quantity of CBR PLUS to be added to each tanker load. Although this is the preferred method (to use as many tanker loads as possible for more even CBR PLUS application), it may arise that high in-situ moisture content or short production lengths exist that may dictate that the CBR PLUS be added to fewer tanker loads (even only 1) with the resultant increase in the possibility of poor or uneven distribution.

i) Mixing CBR PLUS into water:

The mixing of CBR PLUS in the tanker is done by adding the required amount of CBR PLUS per tanker to the water and then alternatively driving the water tanker backwards and forwards over a short distance (+10 m), by circulation with water pump fitted to the tanker or the tanker traveling to site (over a long distance). It should be noted that if the groundwater or water used for mixing should have a pH value not exceeding 8, otherwise the reaction of CON-AID on the soils will not be fully effective.

j) Mixing of materials:

The diluted solution is sprayed onto the layer while being processed by grader, Rotovator or disc harrow to break down material while removing solid large material (+ 100 mm) by hand until all the required water and CBR PLUS have been applied and the soil mixture is homogeneous and above OMC (1-2%). Should the material be below OMC additional water without CBR PLUS should be applied. If during construction, a rainstorm is about to fall the layer should be shaped into a pronounced camber and lightly compacted to prevent additional water getting into the layer being processed. If during construction, a rainstorm falls onto open work then work should be halted until the weather is sunny and/or windy when the material should be opened out to dry back to OMC + 1-2% before completing the mixing. Occasional grader cutting to open the material will help in the drying process.

k) Compaction:

The layer material should be spread by grader to provide thin uniform layer (+ 5 cm) so that compaction can begin by using conventional equipment of at least 10 tons mass compacting from the lower layers to the top final layer at the correct shape and to the required specified density. When mixing clay, a tamping or sheep-foot roller is most effective. A rubber tyred roller must be used on the surface for final compaction together with other flat wheel compactors. During the compaction process, a grader should lightly skin the surface to provide the final shape. It should be noted that large stones might be dragged causing depressions or furrows. These should be filled by hand with CBR PLUS treated material from the roadside and compacted. If areas of course segregated material occurred on the surface, then these should be sprayed with a heavy application of water and rolled with a rubber tyred roller to produce a bound surface with fines brought up from below the surface. It should under consideration that the typical degree of saturation in a compacted soil during construction is between 80 and 90%. Since it is always encouraged to compact on the wet side of the optimum (moisture content) for CBR PLUS treated road, hence the initial degree of saturation may be as high as 90%. Hence, for a given required dry density, it is possible to estimate the required field moisture content during construction by the above equation (International CON-AID Ltd., 2008).

Curing:

After the stabilized layer has been compacted to the required density and brought to the required lines and grades in accordance with the typical cross section, the completed section must be moist cured. If the layer is the uppermost layer to be stabilized, this moist-curing shall be for a minimum period of 10 days by watering two-three times each day with a water tanker, unless otherwise directed by the engineer. If further courses are to be added, the engineer may cover the stabilized layer immediately after acceptance of the layer. If this layer is not covered immediately, then moist-curing must proceed for 10 days. It should be noted that small amounts of CBR PLUS would remain on the road surface that may create slippery surface. However, after continual watering (curing) or a number of rainstorms, the CBR PLUS on the surface will be absorbed by the soil or washed off, and then the surface will no

longer be slippery. It is suggested that traffic sign to be erected for this short period to notify the public of the possible slipperiness (International CON-AID Ltd., 2008).

2.2.5.2 Nano-polymer Zycosil

Zycosil is new generation of nanotechnology, which has been developed for waterproofing and has UV resistance, thermal resistance, reactive soil modifier to reduce swelling, and has ability to withstand against wind erosion due to its nano scale size and penetrative power (Figure 2.25, Figure 2.26). Water interruption in building materials has been a problem for the last 1000 years. Fortunately, new developments in science and technology have incorporated the use of nanotechnology to produce eco-friendly, Organo-Silicon products that can render most materials hydrophobic for cycles of 20 to 30 years at a very economical cost. Zycosil is eco-friendly, because it is applied in water solution and VOC per applied m^2 is less than 20 percent compared to solvent-based silanes (Zydex Industries Ltd.).

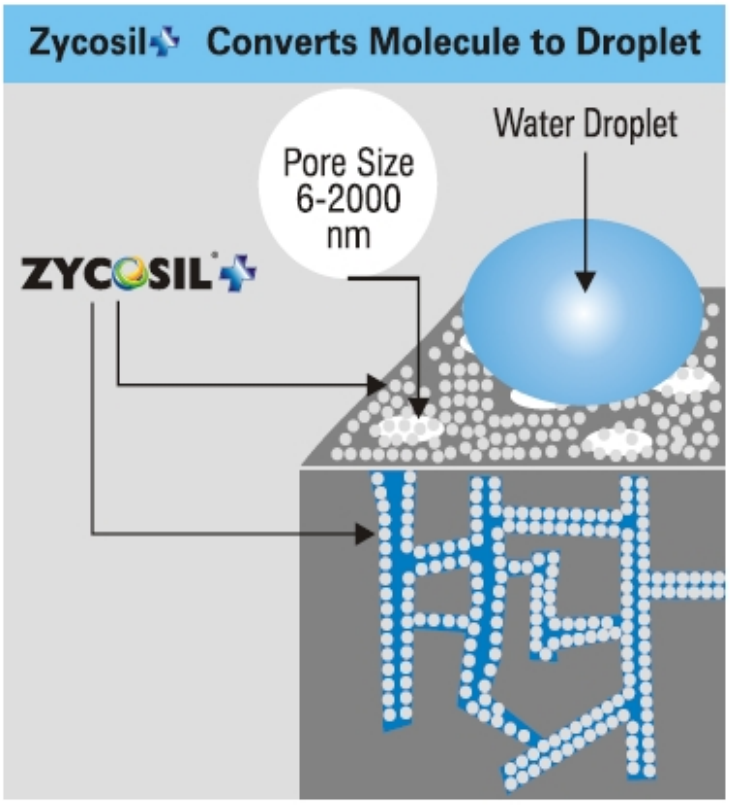


Figure 2.25 : Size and penetrative aspect of Zycosil molecule in compare with water droplet (Zydex Industries Ltd.).

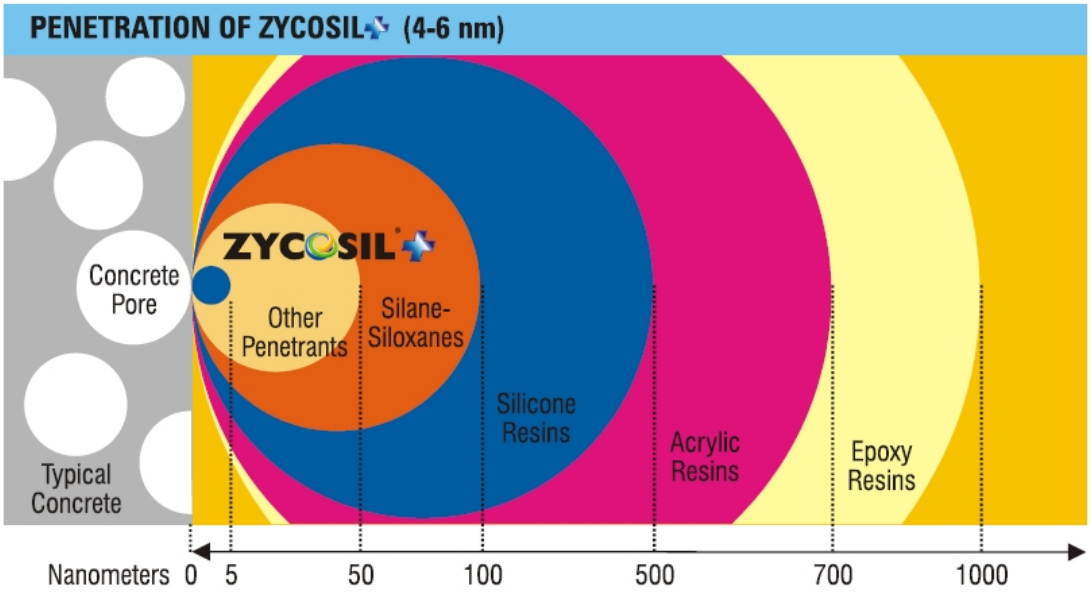


Figure 2.26 : Particle size of Zycosil in compare with different types of waterproofing materials particles (Zydex Industries Ltd.).

Zycosil is being used in different industries as water repellence material, but the data of effect of this chemical on soil and geotechnical engineering is so limited. Therefore, in this thesis it is tried to reveal the different effects of Zycosil on clayey soil by conducting some different laboratory tests. However, the owner company have demonstrated some good features of it and advantages of using of Zycosil on soils.

Zycosil has following features:

- Water based, no harmful solvents, meets the stringent VOC Standards of California.
- Easy application by spray
- Reduces mold and mildew growth
- Reduces material and labor costs
- UV and heat stable
- Chemically converts water absorbing silanol groups to water resistant alkyl siloxane surfaces at room temperature
- Si-O-Si Siloxane bond (sand) survives for centuries
- Works with all types of soils
- One time full drying essential for performance

Advantages of Zycosil are listed below:

- Maintaining CBR Value
- Reduction of Soil Plasticity
- Reduces expansivity of soils by 90 % at dosage of 0.3 – 1 kg per cum for soils ranging from PI 5 to 40
- Waterproofs compacted soil surface up to 10 mm depth
- This layer is breathable, as it does not allow water to get in but allows vapor to escape
- Prevent capillary rise at the base and water entrance from the top, keeping the bases dry throughout all seasons
- Reduce erosion up to 50%

Limitation of Zycosil:

1) Adverse effects of the solvent used- ethylene glycol.
2) Many precautions have to be taken.
3) More effective on pre-existing cracks than cracks, which occur after the application.
4) It cannot be applied if:

- Ambient temperature is below 10º C or above 50ºC
- Rain is expected within 2 hours following the application.
- Precipitation has occurred within 24 hours prior to application.
- High winds or other conditions prevent proper application and overspray that may have an adverse effect on surrounding materials.

The chemical action of Zycosil:

Zycosil chemically converts water absorbing silanol groups to water resistant alkyl siloxane surfaces at ambient temperature. Additionally, nano layer of alkyl siloxane chains on the soil particles provides charge shielding to reduce the repelling effect between soil particles and gives better lubrication for compaction (Figure 2.27).

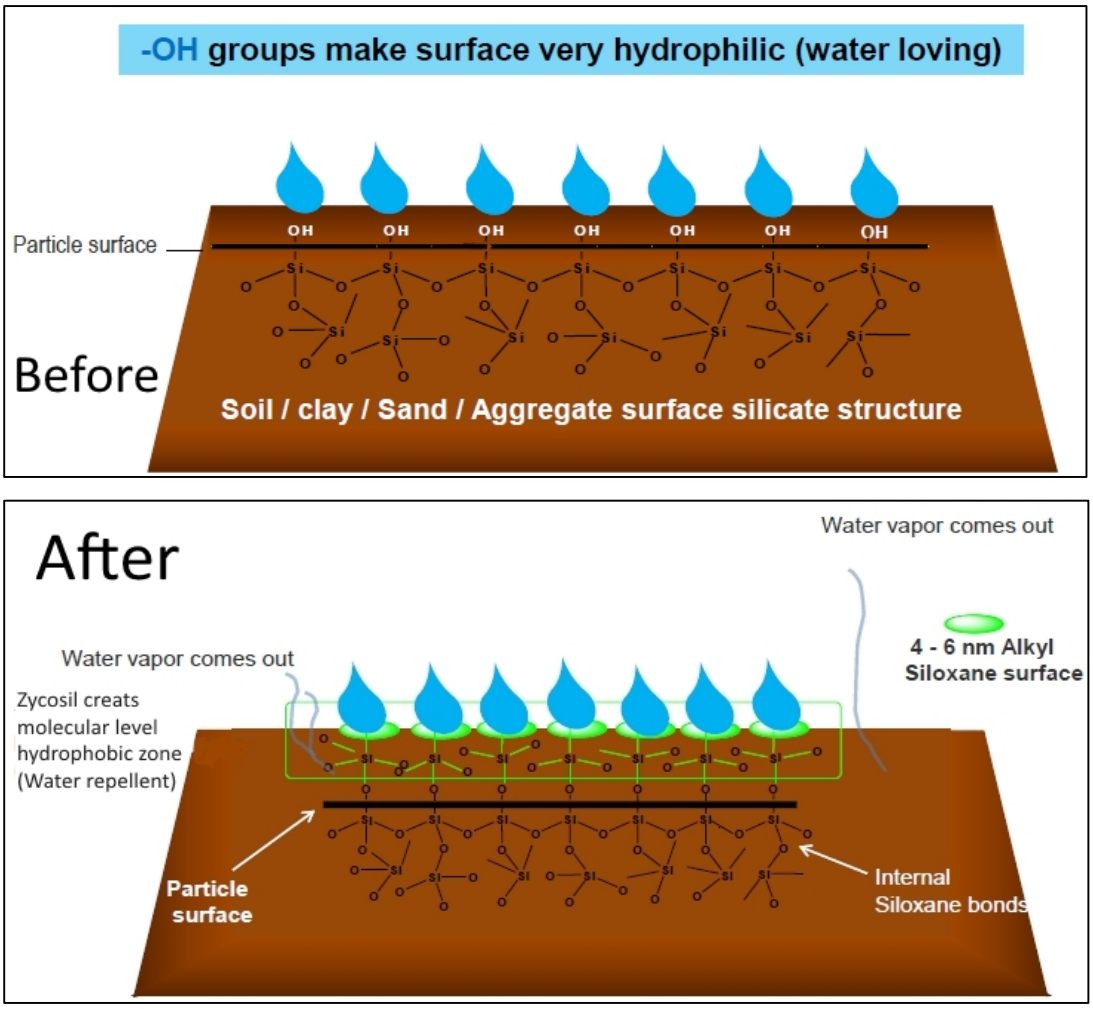


Figure 2.27 : Soil / Clay / Sand / Aggregate surface silicate structure before and after of Zycosil reaction (Zydex Industries Ltd.).

The effect of Zycosil on soils in reality is shown in Figure 2.28:

Figure 2.28 : Maintain friction value of CBR ofter getting soaked (Zydex Industries Ltd.).

Mixing instructions:

- Use cool, clean water, clean tools and clean containers
- Add water first, slowly pour in Zycosil concentrate. Stir slowly for 3-5 minutes, or with a very slow speed paddle mixer for two minutes. Avoid creating foam.
- The sealer is concentrated and needs to be diluted with clean water at the rate of 10 parts water to 1 part Zycosil or 20 parts water to 1 part Zycosil base on demand.
- Mix only can be used within 24 hours

Application:

- 20mm of soil base should be opened up and mix with specified amount of cement as per mix design, typically up to 3%.
- Zycosil solution in water should be sprayed for achieving optimum moisture content (Figure 2.29).
- The dosage of Zycosil can vary from 0.3 kg to 1 kg per m3 of compacted soil, based on clay content and expansiveness (Pl).Clayey and rexpansive soils need higher dosage. After thorough mixing, the soil at optimum moisture content, can now be compacted using vibro roller.
- The compacted soil surface, should now be spray-saturated with Zycosil solution in water (based on designed mix) (Zydex Industries Ltd.).

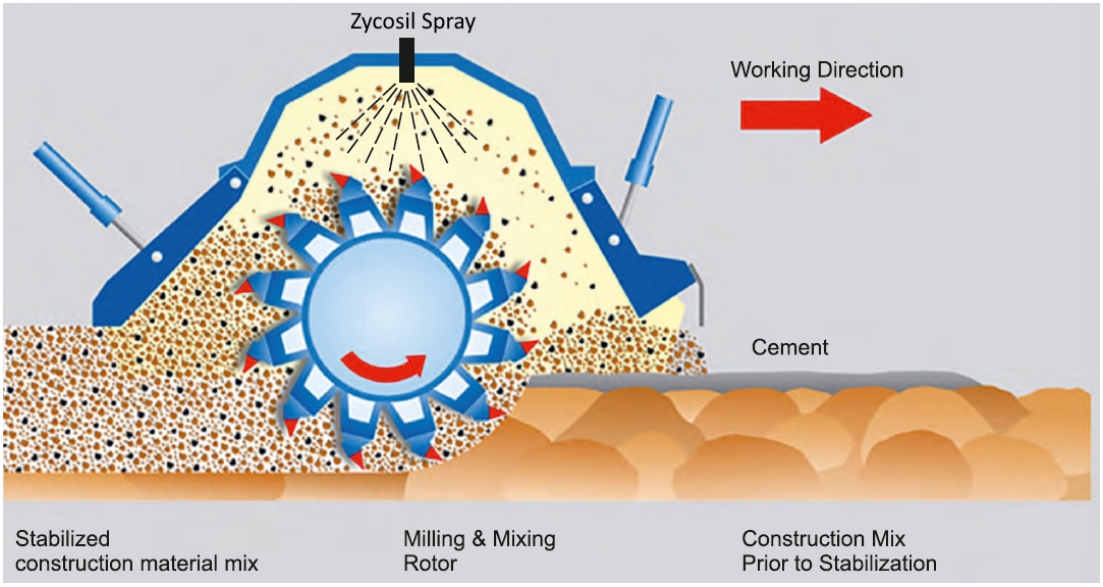


Figure 2.29 : Application of Zycosil to roads (Zydex Industries Ltd.).

2.2.5.3 Nano-polymer Alphasoil®-06

Alphasoil®-06 is a chemical product, based on Nano-Technology, what is surface-active and thereby releases the adhesive water film of the soil colloids. This allows a permanent collection of the fine and fine particles of the treated soils.

Alphasoil®-06 is not a binding agent like cement. However, it can release the own ground bonding force of the soil and affect soil behavior like this, that a permanent increase of compaction under load and transport occurs (Alphasoil technical solutions GmbH).

Effects of Alphasoil®-06 on soils:

- Better compressibility by changing the nature of water
- Greatly reduced water absorption by capillarity-stopping
- Reduced fluid and water permeability
- Extensively reduced source and shrinkage behavior
- Extensively reduced Water sensitivity
- The effect of the consolidation/agglomeration is continued in the treated soil; under load and transport the density reaches values in excess of 100%
- The Proctor Optimum of the treated soil is lower, the density higher.

The advantages of Alphasoil®-06:

- Reduces the water permeability and capillary water management
- Reduces P.I. (plasticity index)
- Shifts the Proctor Optimum to left to an lowered OWG and to an higher density
- Massively increased the carrying value (with soaked CBR 3 -5 times in more than 50% of cases by more than 5 times)
- Reduces the water absorption and thus the swelling - and shrinkage behavior of the treated soil
- Reduces the softening due to water absorption; after the soil has dried out even in part, the moisture content levels off at or below the PO (Proctor optimum)

- The full effect of the treatment is visible after the treated soil under the PO could dry out and improved by time under traffic load enormously. The improvement effect is permanent.
- No traffic stop; traffic can be used to compress;
- Under traffic there is a permanent densification, which leads to a continuous increase in strength .

The disadvantages of Alphasoil®-06:

- Alphasoil®-06 works in the base layer-range, and makes a wear-layer required to prevent mechanical removal
- The so treated ground roads without wear layer is not recommended; the mechanical abrasion would dissipate the so treated layer and slowly but surely raising dust, because of the high fines content
- These fine particles make the surface slippery and offer no grip
- Extremely water-soaked soil what want to be treated with Alphasoil®-06, must be dry down to the below the recommended moisture content of 14% for the P.O. (Proctor optimum). Therefore is to consider, the weather conditions and the addition of Alphasoil®-06-Working-Solution
- Laboratory investigations in advance: Determination of the pH value of the soil (too acidic or too alkaline); determination of salinity in the soil over 2% (hygroscopic); gradation analysis to determine the grain size distribution; bearing capacity measurement of the bedrock (CBR-value); water storage test by producing a soil test specimen treated with Alphasoil®-06

Application range:

This material is useable in different construction sites such as:

- All kind of roads, agricultural and forest roads and tracks, Cycling-, hiking- and riding- trails, Parking- and storage-areas
- For Erosion Reduction in: Dams and Dikes, Drains and Wire ways, Trash Dumps, Bio-Treatment Plants

Mixing method:

Often the "mix in place" mixing process, is the demanded strategy to improve existing soil material by mixing in Alphasoil®-06. In this method, the mixing equipment is brought to the material.

The experimentally determined amounts of Alphasoil®-06, is here applied to the loosened soil and mixed intensively. Additional material can be added to improve the mechanical capacity of the soil, before the Alphasoil-06 mix.

Alphasoil®-06 must be mixed 1:4 with water to produce desirable solution. Thus, prepared Alphasoil®-06 will be diluted again with water in an amount of 0.6l/m3 into Alphasoil®-06-Working-Solution, what will be mixed into the soil. Take as much water for dilution, with regard to its own water content, to come after the incorporation into the soil near the optimum moisture content after proctor optimum.

As a standard, Alphasoil®-06 will be mixed in a 25 to 30cm layer in an appropriate amount.

So that the full effect of Alphasoil®-06 is visible and measurable, the so treated soil must have the opportunity to dry back once to at least 50% of the P.O. (Proctor optimum) (Alphasoil technical solutions GmbH).

2.2.6 Stabilization using nano-materials

Nanotechnology was first introduced in 1959 when Richard Feynman proposed the idea of nanotechnology in his famous lecture “there’s plenty of room at the bottom”(Feynman. R, 1960). This technology afterward made progress increasingly in all sciences. Nano technological achievements have provided a modern approach in geotechnical engineering as well. Nanotechnology is commonly defined as the understanding, control, and restructuring of matter on the order of nanometers (i.e., less than 100 nm) to create materials with fundamentally new properties and functions. Moreover, it should not be neglected that adding only small amount of Nano-Particles can have a huge effect on materials due to their high rate of specific area. Considering these definitions, nanotechnology is a novel approach in all sciences. Such an approach can be proposed in geotechnical engineering as two issues: 1. Studying the soil structure in nanometer scale and hence gaining a better understanding of soil nature, together with studying performance of soils with different nanostructures; 2. soil

manipulation at atomic or molecular scale, which has currently been facilitated through addition of nanoparticles as an external factor to soil. This thesis especially have focus on the second aspect regarding adding Nano-Particles to clayey soil and study effects of them.

There are limited studies, which have been conducted regarding use of nanoparticles for improvement of soil strength parameters. Thus, this study's aim is to investigate the effect of most frequently Nano-Materials such as Nano-Clay, Nano-Silica Powder, and Nano-Carbon Fibers on different aspect of clayey soils properties. In this chapter we address previous articles about these materials and gained results.

This section will mention the results of other researchers that have used Nano-Materials in their tests.

Note that the mixing method and dosage, used in this study, is extensively explained in the next chapter.

2.2.6.1 Nano-Clay

Among other Nano-Materials, Nano-Clay is one the most frequent materials that is being used in such studies. The essential Nano-Clay raw material is montmorillonite, a two to one layered smectite clay mineral with a platy structure with van der Waals force between the neighboring layers (Water Environment Federation, 1992). The thickness of each layer is about 1 nm, diameter from 10 nm to several microns, and the interlayer space around 1 nm depending on the modification methods (Park & Sposito, 2003). Due to its high aspect ratio and good physical and thermal properties, Nano-Clay has the potential for exceptional improvements in barrier, flammability resistance, thermal and mechanical properties for polymer composites at very low filler loading (Mitchel, Hooper, & R.G. Campanella, 1965).

Effect of Nano-Clay on CH clay

Mohammadi and Niaziann (2013), reached to this conclusion in their tests that adding Nano-clay to soil causes to increase liquid limit and plastic limit of soil but plastic index reduces in the soil. However, by adding Nano-clay soil shear strength of soil increases and that is because of cohesion enhance; this keeps increasingly to 1.5% and then no change happen. Final strength increases by increasing Nano-clay in unconfined compression test and CBR tests. When Nano-clay percent in soil reaches to 1.5%

percent, soil has the maximum final strength. As Nano-clay increase to 2% the final strength of the specimen reduces. According to gained results, it can be concluded that Nano-materials can influence in soil in the low percent.

Effect of Nano-Clay on CL clay

The results of conducted tests showed that Atterberg could be increased by adding Nano clay into the soil. This variation is greater in plastic one. Given the results of unconfined compressive strength on Nano clay samples, the soil containing 1.5% of Nano clay has the greatest resistivity. Increasing Nano clay content from 1.5 to 2% reduces the ultimate compressive strength. CBR test results showed that the sample containing 1.5% of Nano clay had the highest CBR value and rising the amount of Nano clay by 2% decreases the CBR Value. Despite this reduction, the total result is more than CBR value of sample without Nano clay. It is concluded that Nano clay due to high specific surface area makes a major impact on soil engineering properties, and very little percentage of this material can be used to achieve better results (Nohani & Alimakan, 2015).

Effect of Nano-Clay on MH and ML silt

Nano-Clay causes slightly decrease in plastic limit, the liquid limit and plasticity index increased slightly as the Nano-Clay content increased. At 4% Nano-Clay, the type of soil in composite samples changed from high plastic silts (MH) to high plastic clays (CH).

Permeability rapidly decreased by adding 3% Nano-Clay after that it seemed to be less affected by Nano clay content, however at neutral condition adding 3, 4 or 5% of Nano clay seemed to have minor impact on permeability in both acidic and alkaline situations. As a result, 4% Nano clay was chosen as an additive to reduce the silt's permeability. Furthermore, the experimental results showed that the percentage of swelling increased as the Nano clay content increased. Greater swelling was observed in acidic condition. Yet swelling in neutral condition was more considerable than in alkaline range (Kananizadeh et al., 2011).

On the other study which has done by Nikookar et al (2013), it has gained that the optimum Nano-Clay content of about 1.5% by weight for ML and 1% by weight for MH can produce maximum strength and increasing the Nano-Clay content beyond this is not beneficial.

Regarding CBR test, adding 1.5% Nano-Clay to ML and 1% to MH can increase the CBR values from 8 (for ML) and 5 (for MH) to 36 and 16, respectively. The CBR value obtained for ML is suitable for road construction and MH soil CBR value seems good but not as well as ML (Nikookar et al., 2013).

They also claim that the addition of Nano-Clay increased liquid limit and plastic limit of silty soils and the adhesion and internal friction angle have gotten noticeable changes.

Effect of Nano-Clay on sand-cement mixture

The results of 7, 14 and 28 days unconfined compressive strength test of sand-cement and mixed with different percentages of Nano clay shows that compressive strength of soil-cement mixture with 1% ratio of Nano clay was 67% higher than control mixture. But, 14 and 28 days compressive strength of Nano clay soil cement mixture is found to decrease with the increase in Nanoparticles ratio from 3 to 5% and the reason can be disorderly dispersed of Nano Clay. This study claim that the reaction of alumino-silicate elements in Nano clay with the calcium hydroxide in cement leads to the increasing in bond strength and consequently resulting in higher tensile strength of hardened cement paste.

Moreover, adding the 2% of Nano Clay can increase the Elastic Modulus of Compression strength and Tensile strength up to 490% and 35%, respectively. It means that adding Nano-Clay can lead to decrement in cement content of stabilized soil (Arabani et al. 2012).

2.2.6.2 Nano-silica powder

In 1992, Yonekura and Miwa utilized silica nanoparticles to increase sand compressive strength (Yonekura & Miwa, 1993). In addition, Noll et al. investigated the use of silica nanoparticles in 1992 for enhancing soil's strength against consolidation and permeability (Noll et al., 1992). In 2005, silica nanoparticles were utilized by Gallagher for increasing soil's cohesion/adhesiveness and decreasing its viscosity, and behavior of the sand improved by nanomaterials was analyzed in cyclic loading conditions. As a result, it was indicated that cohesion/adhesiveness depends on percentage of nanoparticles increase (Gallagher & Lin, 2005). In 2007, Patricia et al. in the United States used nanomaterials practically in a place whose soil was of sand type with high viscosity and reported 40% improvement in settlement after applying

artificial earthquake and evaluation of the yielded settlement (Gallagher et al., 2007). To study the effect of silica nanoparticles in dimension range of 5-100 nm, Butrón et al. carried out oedometer test, triaxial test, and compressive test, and showed that soil strength increases with time, such that the soil containing nanoparticles is ductile in initial stages and subsequently becomes elastoplastic (Butrón, 2009).

Effect of Nano-Silica on CL clay

In the research, which has been done by Changizi and Haddad, shows that increasing in Nano-Silica content contributes to increase in the maximum dry unit weight and the optimum moisture content of the stabilized soil. By adding Nano-Silica, the maximum dry unit weight and the optimum moisture increase by 13% and 20% respectively for Nano-Silica content of 1.0 %.

In addition, the peak shear strength of stabilized soil increases in such a way that by adding 0.7 % Nano-Silica, in normal stress 300 kPa, the peak shear stress increases around 74%. The increase in strength for the soil with 1.0 % Nano-Silica is only about 14 % in comparison with the soil with 0.7 % Nano-Silica.

As a result, increasing the amount of Nano-Silica content leads to increase in both angle of internal friction and cohesion. For all of the cases, the maximum increase in both angle of internal friction and cohesion is observed at 1.0 % Nano-Silica content. By adding 1.0 % of Nano-Silica, the angle of internal friction and the cohesion of soil stabilized increase by factors 2.1 and 1.23 respectively.

In view of increase in Nano-Silica content, the unconfined compression strength of soil stabilized increases, also the residual strength and the strain failure of soil stabilized decrease. By adding 0.7 % of Nano-Silica, the unconfined compression strength of the stabilized increases around 56%. By adding 1.0 % of Nano-Silica, the residual strength and the strain failure of soil stabilized decrease by factors 0.48 and 0.62 respectively.

When the tension cracks caused by loading begin to appear, Nano-Silica can efficiently cause further development of tension cracks of the soil. It means that Nano-Silica to increase in the brittleness of stabilized soil (Changizi & Haddad, 2016).

Effect of nano-silica on lime stabilized CH clay

The high reactivity of Nano-Silica particles with lime is because of small size of particles that leads to improve properties of soil such as plasticity, compression, swelling and increasing strength in the shortest time.

Due to small size of Nano-Silica, the addition of these nanoparticles increase samples' reactivity even at an early age. Consequently, compressive strength is increased. This can be very effective in projects which soil strength needs to be increased and its engineering properties to be improved in short term.

The effect of Nano-Silica has tested by Pashabavandpouri and Jahangiri in 2015. They claim that soil plasticity is not improved with the addition of Nano-Silica particles alone. Moreover, because of high softness of nano silica particles, addition of nano silica up to high percentages can increase soil plasticity. However, soil plasticity properties have been improved considerably in samples in which Nano-Silica and lime are used, so that in samples containing 4% and 5% of nano silica and lime, in respect, soil plasticity index value decreases as much as pi= 6, hence soil plasticity is lost and soil becomes more workable.

Addition of Nano-Silica alone caused a slight increase in soil optimum moisture content and a slight decrease in specific weight of samples. But because of rapid reactions and forming coarse particles as a result of adding nano silica and lime, optimum moisture content and maximum dry density of samples substantially increased and decreased respectively. So that by adding 4% of lime and 5% of nano silica optimum moisture content and maximum dry density change from 18.8% and 16.9(kN/m^3) to 23.5% and 14.8(kN/m^3) in respect.

Nano-Silica alone did not increase strength of the samples and samples' strength was reduced because of more water absorption. Nevertheless, addition of lime with silica nanoparticles, due to pozzolanic reactions between ca++ and sio2, which formed CSH, caused strength of samples to increase dramatically. For instance, the strength of a soil sample, containing 4% of lime and 5% of Nano-Silica, increased from 45 kPa to 1330 kPa.

Curing time diagrams indicated that because of small size of particles, addition of nano silica to samples mixed with lime has increased reactivity rate and hydration process with lime and produced cementitious materials even at an early age(7 days).

Consequently, strength is increased from 45kpa to 610kpa in a 4% of lime and 5% of the nano silica mixture. It can be very helpful in projects that soil strength should be increased and engineering properties of soil need to be improved in short term. It is evident that using high percentages of nano silica has little impact on strength of early age samples.

Regarding swelling issue addition of nano silica in presence of lime makes a significant decrease in the percentage of swelling of clays with high plasticity. Test results showed that swelling of the soil after addition of 5% of nano silica and 4% of lime decreases from 37.5 to 2.5 percent within 28 days and practically soil swelling becomes impossible. This could be applied in areas where expansive soils put lots of pressure to foundations of structures and roads. Due to immediate reactions, production of cementitious and strengthening of soil texture, addition of nano silica to soils stabilized with lime makes swelling to decrease in the early days of testing. This advantage can be used in projects that need to control swelling in the early days (Pashabavandpouri & Jahangiri, 2015).

2.2.6.3 Carbon nano-tubes

Carbon Nano-Tubes can now be considered as the “king” of nanomaterials as it is being used in many applications such as medicine, electronics, energy and environment, etc. Its exceptional strength have made it as one of the candidates for stiff and robust structures (Cao, 2004).

Carbon nanotubes (CNTs) are particularly attractive for use in cementations systems because they appear to be close to ideal reinforcing materials. Their unique physical properties, including ultra-high specific surface, extremely high yield strength and moduli of elasticity, and elastic behavior all point to the potential of CNTs in reinforcing applications (Makar, 2011). Actually, the carbon nanotubes are not a cementitious material but once introduced in a soil they are expected to reduce the interparticle’ spacing, which will promote the construction of a stronger and stiffer soil skeleton matrix, together with the cementitious materials, therefore improving the mechanical properties of the soil. Thus, optimization of nanoparticles distribution is required (solving problems related with particle agglomeration) to obtain a final material with the best characteristics at a competitive cost.

However, there are so limited research has been done regarding effect of carbon Nano-Tubes or Nano-Fibers on engineering properties of soils. A. Alberto et al. in 2015 was one of researchers group that investigate the effects of applying multiwall carbon nanotubes on cement stabilized soil. They have used MWCNTs which MWCNTs were supplied by Nanocyl and, according to their data, the MWCNT CN7000 have an average diameter of 9.5 nm, average length of 1 500 nm and a specific surface between 250,000 and 300,000 m^2/kg (about 1 000 times higher than cement particles).

They reached to this conclusion that the presence of nanoparticles in a soil- cement matrix has the ability to reduce the interparticles' spacing, which will promote the construction of a stronger and stiffer soil skeleton matrix together with the cementitious materials, therefore improving the mechanical properties of the material. Finally, it was shown that the addition of even a very small quantity of MWCNT, effectively dispersed, improves the mechanical properties of a soil chemically stabilized with cement, this improvement reaching values of the order of 77.1% and 110.4%, respectively for the $q_{u\,max}$ and E_{u50} (Alberto et al., 2015).

In another study, which has done by M. Taha and T. Ying on Kaolinite, the results show that the liquid limit and plastic limit of the mixtures increases with the amount of CNT in the mixture. This means that the mixture have a lower soil strength, higher compressibility and reduced hydraulic conductivity. Moreover, the parameters derived from the consolidation tests show that the compression and swelling index increases, with the amount of CNT (Taha & Ying, 2012).

2.2.6.4 Nano-magnesium oxide

M. Raihan Taha et al.in 2015 conducted a study on "Treatment of Soft Soil with Nano-Magnesium Oxide". In their research, they used Nano-Magnesium oxide (N-MgO), which were supplied by Inframat Advanced Materials Company, Manchester, USA. The specific surface area of N-MgO was 50 m^2/g with density of 3.60 g/cm^3.

Based on obtained results they claimed that the plasticity index exhibits significant reduction compared with untreated soil. This reduction is in proportion with curing time and magnesium oxide surface area. Also, The unconfined compressive strength of treated soil increased significantly over time with increasing percentage of magnesium oxide. Compressive strength of the Soil - 1% N-MgO mixture increased around 270% and 400% for 1-day and 28-day respectively.

Regarding the mechanical behavior, the treated soil performance changed from ductile to brittle associated with remarkable increase in Young's modulus.

The outcomes also revealed that the stiffness developed from soft and medium stiff soil to a very stiff soil for Soil – N- MgO mixtures.

2.2.6.5 Nano-titanium dioxide (TiO2)

In a paper that is written by S. Babu and S. Joseph in 2015, the effect of Nano-Tio2 on silty clay was studied. They used Nano - Titanium Dioxide that was collected from KMML, chavara, Kollam, and the size was 15 nm. After conducting several tests, they concluded that increase in Nano-TiO2 decreases the Atterberg's limits around 60 %.

The results showed that with an increase in percentage of Nano-TiO2 the maximum dry density increased by 2.94 % and optimum moisture content decreased by 5.2 %.

Regarding the the shear strength, it increases up to 87 % for 0.5 % of Nano-TiO2. Additionally, the consolidation settlement behavior was reduced up to 60 % for 0.5% of Nano-TiO2.

In respect of CBR value, it increased from 2.23 to 3.13 and 2.08 to 2.58 for 2.5mm and 5mm of penetration, respectively.

2.2.7 Other non-traditional methods of stabilization

In this section briefly Non-Traditional methods of soil stabilization will mention and explain.

2.2.7.1 Asphalt stabilization

Asphalt stabilization is the process of adding asphalt to an inorganic soil to act as a binder thus serving as cementing agent. Asphalt soil stabilizer consists of asphalt globules of microscopic sizes, which remain suspended in water without any coalescence of the asphalt. However, the stabilizer must be stored and used at a temperature above freezing. Freezing causes the asphalt to settle out of emulsion and it becomes unusable for brick making. When the stabilizer is mixed into a clay-bearing soil in the presence of water, the water carries the asphalt globules into direct contact with the surface of the clay particles (Baxter, 1972).

Since the water-carrying capacity of the clay many times exceeds that of the sand and small rock particles, practically all the asphalt is brought into close contact with the clay. As evaporation progresses, the asphalt globules are drawn into very thin film asphalt globules so dense that the asphalt forms a practically solid coating which is irremovable from the surface of each clay particle. The amount required to coat clay particles is minimal when compared to most other coating particles. The coating is so thin that the soil darkens only very slightly. When fully dried, the entire mass of the clay treated with asphalt emulsion has an improved compressive strength than the dried untreated soil mixed with water only (Baxter, 1972).

Moreover, the asphalt stabilizer does not diminish the cohesive qualities of the clay particles in the soil. However, it should be noted that since the asphalt films are repellent to water, the clay particles could not become wet and return the soil to mud state again. Some absorption of water may occur upon prolonged exposure, but the fine clay particles do not expand and lose cohesion in the presence of such moisture (Kimmons & Matteson, 1968).

Stabilization of soils and aggregates with asphalt differs greatly from cement and lime stabilization. The basic mechanism involved in asphalt stabilization of fine- grained soils is a waterproofing phenomenon. Soil particles or soil agglomerates are coated with asphalt that prevents or slows the penetration of water which could normally result in a decrease in soil strength. In addition, asphalt stabilization can improve durability characteristics by making the soil resistant to the detrimental effects of water such as volume. In non-cohesive materials, such as sands and gravel, crushed gravel, and crushed stone, two basic mechanisms are active: waterproofing and adhesion. The asphalt coating on the cohesionless materials provides a membrane which prevents or hinders the penetration of water and thereby reduces the tendency of the material to lose strength in the presence of water. The second mechanism has been identified as adhesion. The aggregate particles adhere to the asphalt and the asphalt acts as a binder or cement (Joint Departments of the Army and Air Force, 1994). Though with the good performance of the asphalt as a chemical stabilizer to clay, they are not readily available and are expensive in Ghana.

2.2.7.2 Aluminum sulphate stabilization

The use of aluminum sulphate as a chemical stabilizer has received very little attention. As pointed out by Demirel et al. (1961), aluminum sulphate has been used to some extent to provide metallic ions to be used with phosphoric acid in soil stabilization to give sufficient gain in strength. Hayden (1975), also used aluminum sulphate as a chemical stabilizer for dispersive clay. From the analyses of Demirel et al (1961) and Hayden (1975), this stabilizer appeared to be good for the treatment of dispersive clay, its negative effect is the increase in exchangeable aluminum resulting in an increase in soil acidity.

2.2.7.3 Sodium hydroxide stabilization

As pointed out by Ingles in his publications (1968, 1970, 1972), clays rich in aluminum minerals, for instance kaolinite perform better when mixed with sodium hydroxide (NaOH) as stabilizer than clays which are montmorillonitic. This was also confirmed by Olaniyan (2008), that the sodiums hydroxide reacts very effectively with soil rich in aluminum. They initially showed a slight decrease in strength but with time, they increase in strength. This is because sodium hydroxide (NaOH) on clay attacks the clay mineral lattice and produces sodium silicate and sodium aluminate. Sodium aluminate then proceeds to precipitate insoluble sodium aluminum hydro-silicate, which gives the soil considerable durability.

Gogo (1985) did a similar work on two clays supplied from France. He used sodium hydroxide (NaOH) as a single stabilizer and concluded that kaolinitic clays can successfully be stabilized with sodium hydroxide to produce materials with highly improved strength and water resistance. In realizing the success and benefits in clay stabilization, Gogo (1985) recommended that a replacement of sodium hydroxide (which was the stabilizer he used for his work) with local substitutes be made since they are not readily available and are expensive to buy. The replacement of these stabilizers with local substitutes can lead to the development of a durable and cheap building material that could play a key role in improving the living conditions in many poor communities.

From the review of the related literature on the response of using aluminum sulphate, lime, cement, asphalt, liquid chemicals and sodium hydroxide, sodium silicate among others as chemical additives or stabilizers in stabilization of clays and combination of

two or more of these additives or stabilizers, encouraging results were reported from these studies. These stabilizers or additives are not readily available locally; those readily available are costly which make their usage expensive. There is therefore the need for replacement of these stabilizers or additives with local substitutes which can lead to the development of a durable and cheap building material for many poor communities.

2.2.7.4 Stabilization using salt (NaCl, MgCl2, CaCl2)

Hassnen (2013), reported increase in unconfined compressive strength of soil treated with 8% NaCl up to 700kN/m^2, also results showed that maximum dry density of soil was increased from 1.85-1.92gcm-3 with increase of 8% NaCl in soil sample. Soil samples were prepared from commercial clay, River Aired soil, sand, and gravel. The study further showed that addition of salt resulted in increase in resilient modulus. This is potentially useful for long-term highway pavement subgrade applications.

Tamadher et al. (2007), conducted laboratory test to investigate the effect of adding different chloride compounds i.e. (NaCl, MgCl2, CaCl2) on the engineering properties of silty clay soil. Various amounts of salts (2, 4, and 8% by weight) were added to the soil to study the effect of salts on the compaction characteristics, consistency limits and compressive strength. Test results showed a maximum dry density increased from 17.5kN/m3 to 19.0kN/m3 and decreased the optimum moisture content from 15% to 13%. The liquid limit, plastic limit and plasticity index decreased with the increase in salt content. The unconfined compressive strength increased as the salt content increased.

2.2.7.5 Stabilization using molasses

Molasses is the most valuable by-product from the sugar industry. The molasses referred to in this research is blackstrap molasses, which is the product of raw sugar from sugar cane. Blackstrap molasses is the final byproduct of the third boiling cycle in the sugar making process. This type of molasses has a very dark color and is extremely viscous and contains approximately 20% sucrose, 20% reducing sugar, 10% ash, 20% organic non-sugar, and 20% water (Lewis, 1993). Molasses products act as weak cement by binding the soil particles together (Expert Panel, 2002). When high additive contents are used (5% plus) gravel loss reduction realized (Phil , 2014).

Testing performed by Shirsavkar & Koranne (2010), verified that molasses can be an effective soil stabilizer. Soil modified with molasses by adding 5%, 5.5% 6.0%, 6.5%, 7.0% and 7.5% to gravel-clay sample, test results show that, value of California Bearing Ratio (CBR) found to increase by 5.12%, 22.67%, 24.68%, 34.00% 23.12% and 22.02%. Also by adding 6.5% of molasses in soil sample, the value of liquid limit and plastic limit increased while plasticity index of modified soil is reduced.

M'Ndegwa (2011) suggested that stabilization of expansive clay soil with molasses increased the California Bearing Ratio (CBR) values and load bearing ability of the soil. Therefore, molasses can be used as stabilizing agent for expansive clay soil. In addition, molasses mixed with expansive clay soil reduced its swelling tendencies.

Therefore, it is clear that laboratory works by other researchers have not highlighted the impact and improvement on permeability, cohesion and internal angle of friction of soil following the addition of molasses during field stabilization.

3. METHODS AND MATERIALS

In this chapter standards, test methods, materials, mixing devices and dosages that had been used in tests for performing this thesis are explained.

3.1 Soil Classification

3.1.1 Moisture content (ASTM D2216-98)

Oven-drying method was used to determine the moisture contents of the samples. For the oven-drying method, small, representative specimens obtained from large bulk samples were weighed as received, then oven-dried at 105°C for 24 hours. The sample was then reweighed, and the difference in weight was assumed the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, giving the water content on a dry weight basis. This procedure was repeated until a nearly constant mass was obtained, and then the moisture content was calculated in the same fashion as for the oven drying method.

3.1.2 Particle size distribution (ASTM D422-63)

Approximately 50 grams of dry soil was treated with a dispersing agent for 18 hours. A hydrometer analysis was then performed to measure the amount of silt and clay size particles. The sample was then washed through a series of sieves with progressively smaller screen sizes to determine the percentage of sand-sized particles in the specimens.

3.1.3 Atterberg limits (ASTM D4318-00)

Representative samples of each soil were subjected to Atterberg limits testing to determine the plasticity of the soils. An Atterberg limits device was used to determine the liquid limit of each soil using the material passing through a No. 40 sieve. The plastic limit of each mixture was determined by using soil passing through a No. 40 sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity

index was then computed for each soil based on the liquid and plastic limit obtained. The liquid limit and plasticity index were then used to classify each soil.

3.1.4 Classification (ASTM D2487-00)

The soil was classified using the Unified Soil Classification System (USCS). Using the particle size distribution and the Atterberg limits, the USCS designates a two-letter symbol and a group name for each soil. A visual-manual procedure can also be used to identify soils easily in the field; however, all classifications provided in this research are based on the laboratory testing-based procedure.

3.1.5 Specific gravity (ASTM D854-00)

Values for specific gravity of the soil solids were determined by placing a known weight of oven-dried soil in a flask, then filling the flask with water. The weight of displaced water was then calculated by comparing the weight of the soil and water in the flask with the weight of flask containing only water. The specific gravity was then calculated by dividing the weight of the dry soil by the weight of the displaced water.

3.2 Mixing and Curing of Soil and Stabilizer

3.2.1 Mixing device

A stand mixer with 5-liter capacity mixing bowl was used for mixing of the soil and stabilizer. A mixer of this size allowed for production of four specimens per batch (one for each of three curing times) to be made.

3.2.2 Mixing procedure

To achieve the desired moisture content for the batch, additional water was first blended into the soil and mixed for three to five minutes. After water addition, the appropriate amounts of stabilizer were then added to the mixture and blended thoroughly for three to five minutes. The mixer was set at the lowest speed, and the water and stabilizer were each added slowly to promote uniform blending and to prevent clumping of the soil and/or stabilizer. It was sometimes necessary to stop the mixer, and scrape unmixed portions from the sides and bottom of the bowl into the mixture and resume mixing.

3.2.3 Dosage rates

Dosage rates can be specified in many different ways, but the most common way to define the dosage rate is based on the oven dried weight of the soil mass to be treated. For Nano-Polymer stabilizers, Manufacturer's recommendations for the stabilizers used in this research are given as a percentage of the optimum water content's volume, which has to be diluted in the water. Therefore, cumulative volume of mixture would be added to the soil.

For Nano-Materials, research data from other researchers indicate that typical dosage rates commonly used range from about 0.25 to 1 percent of the soil weight. The mixture of soil and additives are all base on cumulative weight.

3.2.4 Curing

Since this research did not involve investigation of variations of curing temperature, all samples were cured at room temperature (approximately 20°C). In addition, the tightly sealed samples were put in containers that have water to provide a curing environment of 100% relative humidity. Curing times of 1, 7 and 28 days were used in this work. Two samples for each curing time were prepared in order to provide an indication of reproducibility as well as to provide sufficient data for accurate interpolation of the results. Additional curing times beyond 28 days may be desired in some cases to investigate longer-term changes in strength.

3.3 Using Harvard Miniature Apparatus for Obtaining Moisture-Unit Weight

Optimum moisture content and maximum dry unit weight are two important criteria for evaluating the state of compactness for most cohesive soil masses whether they are deposited naturally or placed by man. Two disadvantages of the currently used test method are (1) the amount of material required to complete the test – approximately 2.5-3.5 kg, depending on the type of material being evaluated; and (2) the length of time required to perform the test. As the Nano-Materials were used in this study were expensive and this study aimed to do vast amount of tests, choosing the Harvard Miniature Device was the best decision to reduce and optimize the amount of soil and additives for conducting the tests.

Both new (Harvard Miniature Apparatus Test) and traditional (Standard Proctor Test) techniques impart the same compactive effort, 600 $kN - m/m^3$, to the soil specimen. A moisture-unit weight plot is obtained when a series of soil specimens are compacted at predetermined moisture contents using the standard Proctor compaction test. The corresponding dry unit weight at each moisture content is determined, and a moisture-unit weight plot is obtained.

Recently, the Harvard miniature compaction apparatus (Figure 3.1), introduced by Wilson (1950), has been used by researchers for preparing triaxial specimens (Highter et al., 1970; Wilson, 1950) and by others to obtain moisture unit weight relationships of soils (Wilson, 1950, 1970). The use of this device results in a quick moisture-unit weight determination and requires only 1 to 2 kg of material.

Use of the Harvard miniature compaction apparatus produces moisture-unit weight curves in less time than the Proctor compaction test and requires only a fraction of the material. Comparative results indicate that the Harvard miniature method can be used to match standard compaction values when the spring force, the number of layers, and the number of tamps per layer are adjusted according to the soil type. Because time and materials could be saved by using the Harvard miniature apparatus instead of the standard Proctor device, similar investigations should be conducted on a variety of soil types to develop a database, from which laboratory and field personnel can draw, to ensure quality moisture-unit weight determinations.

Figure 3.1 : Harvard Miniature Compaction Apparatus.

3.4 Unconfined Compressive Strength Testing (ASTM D2166-00)

The unconfined compression test was performed according to the ASTM D2166-00 method. Each test was performed in a strain-controlled mode. Procedure is the specimen is placed between two loading platens and then stress is applied to compress the soil. A typical machine used for this test is shown in Figure 3.2.

Since there is no confining pressure and no membrane around the specimen, only cohesive soils can be used for this. The loading rate typically around 1mm per minute. The test produced load vs. displacement data until a sign of specimen failure was observed. The raw data were then converted into stress vs. strain plots, with unconfined compression strength (undrained shear strength) and strain at failure. The highest stress applied on this curve is defined as the unconfined compressive strength (q_u).

Figure 3.2 : Unconfined Compression Machine.

3.5 Soaked CBR Test (ASTM D1883-16)

The CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. The CBR is obtained as the ratio of load required to effect a certain depth of penetration of a standard penetration piston into a compacted

specimen of the soil at some water content and density to the standard load required to obtain the same depth of penetration on a standard sample of crushed stone. CBR tests were conducted on the compacted specimens at the optimum moisture content using standard compaction test. In this experiment a sample of stabilized material is compacted into a CBR mold in approximately three equal layers, using a 2.5 kg hammer (56 blows/layer).

The procedure for soaked CBR involves placing the specimens in a water bath at 20±2 centigrade. A collar is added to the top of the specimen and a perforated base plate is attached to the bottom to allow the ingress of water. The water level is kept just below the top of the collar (Figure 3.3).

The CBR molds are soaked for 10 days in a water bath to get the soaked CBR value and the CBR swell of the soil. The CBR swell of the soil is measured by placing the tripod with the dial indicator on the top of the soaked CBR mold. The initial dial reading of the dial indicator on the soaked CBR mold is taken just after soaking the sample. At the end of 10th day, the final dial reading of the dial indicator is taken hence the swell percentage of the initial sample length.

Afterward, mold are taken from water, and put in the penetration device to measure the load in 2.5mm and 5mm of penetration (Figure 3.4).

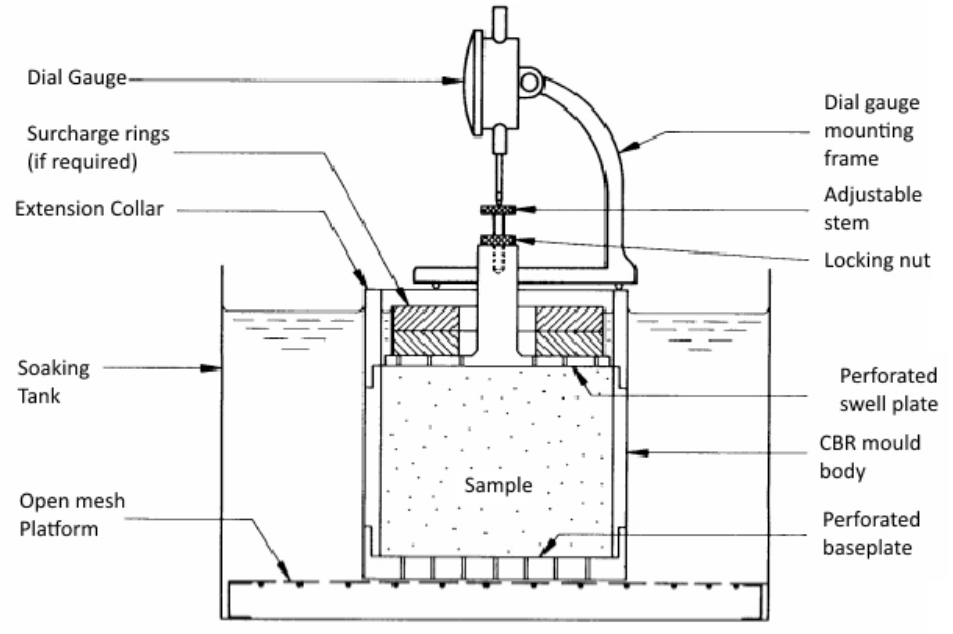


Figure 3.3 : CBR Swell Test. (BS 1924-2: 1990).

3.6 Swelling Pressure Test (ASTM D4546-14)

The swelling pressure for the selected soils was estimated by using zero swell test. In this test, after preparing samples and compacting them in their optimum water content and reaching to their maximum dry unit weight (with the help of Harvard Miniature Device), specimen was transferred from guide ring into consolidation ring. Afterward, adding water to clay started to swelling and at the same time the vertical deformation

Figure 3.4 : CBR Test.

was prevented and the samples were kept at their initial void ratio by continuously adding loads at every vertical expansion of the samples placed on the consolidation frame. The loads were continually added until no deformation was observed. At this stage the swell pressure was calculated as the load required to retain zero swell divided by the specimen area.

3.7 Consolidation and Permeability Test (ASTM D2435M-11)

In general, theory of consolidation deals with the response of soil systems to imposed load and predicts stresses and displacements of the loaded soil as a function of space and time. This theory, since its introduction by Terzaghi in 1923, has formed the foundation of modern geotechnical engineering. The concept is fundamental to the practice of geotechnical engineering where the interaction of soil and water dominates. Although consolidation is used for estimating settlements, it has also played key roles in analyzing stability of slopes, design of piled foundations, laboratory tests, etc (Schiffman et al., 1984).

For applying the consolidation test after determining the swelling pressure, the loads,which are varying from 100 to 800 kPa, were applied to the specimen and data were gathered from settlement of the specimen under these loads odeometer. This procedure had repeated again as unloading to draw the void ratio-Log P curve to calculate the preconsolidation pressure, Cs and Cc.

With the help of data obtained from consolidation test and using Log-Time Method c_v and k (hydraulic conductivity) of can be found.

3.8 Triaxial Consolidated-Undrained Test (ASTM D4767-11)

The triaxial compression test system housed in the Istanbul technical University Geotechnical Laboratory comprised of many state-of-the-art pieces of equipment to permit a careful and high-precision CU test to be carried out by trained laboratory personnel. In this study both manual and automatic triaxial devices have been used. The important system components are listed below:

3.8.1 : Device's equipments

Vacuum Pump: This was used to pull air out of the soil specimen and de-air water.

Water Tank: This cylinder shaped tank was used to hold de-aired water.

Load Frame: This device pressed a loading piston downward against the soil test specimen to load it axially.

Test Cell: This cylinder shaped cell held the soil test specimen and pressurized water around it. The top plate allowed a loading piston to penetrate into the cell. The bottom

assembly connected pressure transducers and drainage/saturation lines to the soil specimen or chamber water.

Sensors:

a) Linear Position Sensor (LPS): This sensor measured the axial displacement of the soil specimen during the test.

b) Load Cell: This sensor measured the reaction force on the soil specimen as it is compressed.

c) Pore Pressure Transducer: This sensor measured the pore pressure within the soil specimen.

d) Cell Pressure Transducer: This sensor measured the confining pressure surrounding the soil specimen.

Controllers (In Automatic Device): This multi-function unit contained a vacuum regulator and pressure regulator. Two large box mounted on the panel held pressurized water and were connected to the cell water and soil specimen. It controlled the confining pressure and back pressure during testing. Also, the controllers have tubes, connecting it to a tap water and air pressure supply.

Others:

a) Network Module: This device regulates the flow of commands and data between the computer and the sensors on the load frame.

b) PC: A standard IBM-compatible PC ran a special software prepared by the manufacturer of the triaxial test system, so that the sensor readings acquisition and test management will be automatic once the soil specimen is conditioned in the test cell.

Figure 3.5 shows a photograph of Automatic and Manual triaxial test setup and the equipment used.

3.8.2 CU test procedure

The CU triaxial compression test procedure followed the guidelines set as ASTM Standard D-4767. The guidelines, however, were fairly general in their descriptions. Major efforts were made to translate some of the specifications outlined in the ASTM

method to practical steps applicable to the actual test equipment being used in the laboratory. The following list maps out the steps taken in running the CU test:

Step-1: Weigh the specimen on an electronic scale, so that the initial moist unit weight is known. Use a small amount of soil remaining inside the tube or the trimmed portion of the soil specimen to determine the initial (natural) moisture content of the soil.

Step-2: Place the soil specimen on the bottom platen attached to the base assembly of the test cell. Position the top platen on top of the soil specimen. Envelop the specimen fully with a thin rubber membrane. Seal the ends of the membrane using two rubbers O-rings. Assemble the test cell by placing the plexiglass cylinder cell wall and the top

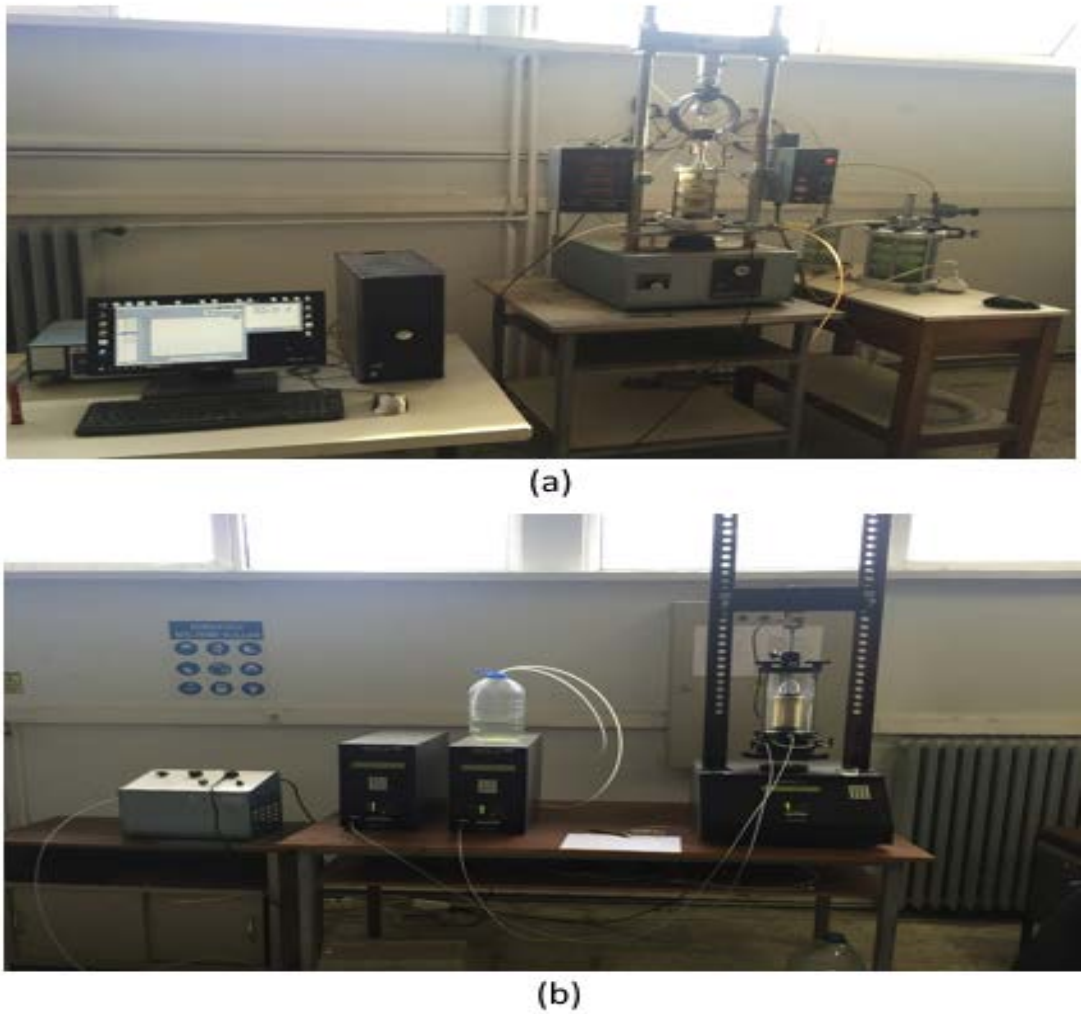


Figure 3.5 : Triaxial Test Setup: (a) Manuel Device, (b) Automatic Device.

assembly. Attach flexible tubing coming from the panel to the base assembly ports. Fill the space between the specimen and the cell wall with the de-aired water by

applying positive pressure to the water in the water tank. Observe that the water flows out of the tube connected to the top assembly.

Step-3: Initiate the saturation process by applying back pressure to the bottom of the specimen. This is done by setting the confining pressure (pressure applied to the chamber water) to 500 kPa and the water pressure going into the top and bottom of the specimen to 400 psi. Leave the specimen subjected to this state for a period of time until a B-value of 95% is reached. This is done by monitoring the pore water pressure reading frequently. B-value is calculated by dividing the change in the pore water pressure reading by the chamber pressure.

Step-4: Once the specimen is saturated, the consolidation process can be started. Increase the confining pressure so that the difference between the confining pressure and back pressure matches the desired effective consolidation stress (200kPa and 300 kPa). The effective consolidation pressure should be equal to or higher than the estimated overburden pressure that existed in the field. This is to assure that the soil specimen will not exhibit overconsolidated behaviors during the test. The specimen is left in this state for 24 hours.

Step-5: After consolidating the soil specimen, close off the drainage paths in and out of the specimen. Bring the loading piston down, so that its tip is in contact with the top platen. Go into the computer software and set the loading rate to the specified value (0.08 mm/min). Begin the loading process. During the test, the computer will record automatically all of the sensor readings frequently and update key plots on the computer screen. The actual test duration will depend on the loading rate and behaviors of the soil specimen (20% of Axial Strain). According to ASTM D-4767, the test is to be terminated at 15% axial strain, a 20% decrease in deviatoric stress, or 5% additional strain beyond the deviatoric stress peak.

Step-6: Shortly after the triaxial test, drain the water from the test cell. Disassemble the cell and carefully remove the soil specimen. Photograph and sketch the final conditions of the test specimen. Determine the final moisture content of the soil by placing the entire specimen in the laboratory oven.

3.9 Direct Shear Test (ASTM D3080M-11)

Direct shear testing (ASTM D3080) provides the shear strength properties of soils under conditions of drained loading, which is required for assessing the stability of earth slopes. It is commonly used to test cohesionless soils (i.e.; sands and gravels) because of the inherent difficulties in preparing specimens of cohesionless soil for triaxial strength testing. The ASTM D3080 standard can also be applied to test cohesive soils under conditions of drained loading to derive drained shear strength parameters. However, it is seldom used for testing cohesive soils, and its use is generally limited to cohesionless soils. The drained shear strength of cohesive soils is more commonly measured using triaxial shear strength testing (ASTM D4767).

The direct shear device is used to determine failure envelopes for soils. The device is not suitable for determination of stress-strain properties of soils.

Direct shear tests (6 cm x 6 cm x 2 cm) were conducted on different Clay-Nano Material base additives in order to evaluate the soil characteristics at different normal stress (ranging from 200 to 400 kPa). The tests were conducted at a constant shear rate. The direct shear tests were conducted on different mixtures samples to evaluate the soil shear strength parameters (cohesion “c” and friction angle “Φ”). The shear stress – shear strain curves for different mixtures were also determined.

The peak shear stress was obtained for all the tests conducted in this research. It is interesting to mention here that the peak shear stress values for different tests were reached at different shear strains.

During examination procedure we unexpectedly had noticed that Zycosil-Rc and Nano-Silica Powder-Rc mixtures showing different behavior in water when they totally get dried in the oven. Then we decided to design a new test:

1- Wet Condition: In this test, we put the samples directly in the shearing device after compacting in their optimum water content and curing 1 day to measure their shear strength parameters.

2- Dry Condition: In this test, we compacted the mixtures in their optimum water content and after 1 day curing time, put them in the oven. After they got dried we cut them and then put them in the direct shear test apparatus.

3.10 Materials

In this study we have chosen five different Nano-Materials and Nano-Polymers, which will be explain in this section.

3.10.1 Soil

Clayey soil was sampled from Ciftalan town in Istanbul/Turkey. This clay is taken from depth of the earth from a coal mine. These samples were found to be high plasticity clay. The properties of the soil are given in Table 3.1.

3.10.2 Nano-clay

Needed Nano-clay in this research is reformed (Montmorillonite) (Figure 3.6). Chemical combinations and physical features of Nano-clay are shown in Table 3.2. This product had purchased from "SIGMA-ALDRICH" company. Product code is: 682608-500G.

Figure 3.6 : Nano-Clay Powder.

Table 3.1 : Properties of Soil.

Properties	Results
Specific gravity	2.75
Fine-grained (%)	99
USCS classification	CH
Liquid limit (%)	60
Plastic limit (%)	31
Optimum moisture content (%)	28
Maximum dry density (kN/m^3)	14.5

Table 3.2 : Properties of Nano-Clay.

Properties	Details
Appearance (Color)	OffWhite to Beige
Appearance (Form)	Powder
Loss on Drying	<_ 6
Density	200 - 500 kg/m3
Average Particle Size	<_20 micron
Matrix	Montmonlonite clay base material
Contains	25-30 wt. % trimethyl stearyl

3.10.3 Nano-carbon fibers

The next Nano-Material that was used in this research is Nano-Carbon Fibers (Figure 3.7). Chemical combinations and physical features of Nano-clay are shown in Table 3.3. This product had purchased from "SIGMA-ALDRICH" company. Product code is 719811-25G.

Figure 3.7 : Nano-Carbon Fibers.

Table 3.3 : Properties of Nano-Carbon Fibers.

Properties	Details
Color	Black
assay	>98% carbon basis
form	platelets (conical) powder
D x L	100 mn x 20-200 pm
impurities	<14,000 ppm Iron content
average diameter	130 DM
pore size	0.12 cm^3/g average pore volume 893 A average pore diameter
surface area	average specific surface area
mp	3652-3697°C
density	1.9 gird. at 25 °C
bulk density	03-33 lb/cult

3.10.4 Nano-silica powder

The other Nano-Material used in this research is Nano-Silica Powder (SiO2) (Figure 3.8). Chemical combinations and physical features of Nano-clay are shown in Table 3.4. This product had purchased from “SIGMA-ALDRICH” company. Product code is 718483-100G.

Table 3.4 : Properties of Nano-Silica Powder.

Properties	Details
Appearance (Color)	White
Appearance (Form)	Nano-Powder
Surface Area	175 - 225 ra2/g
PH	3.7 - 43
Wider Ccotent	< 1.50 %
Purity	99.8% Based On Trace Metals Analysi:
Trace Metal Analysis	< 2500.0 ppm
Diameter	12 nm

Figure 3.8 : Nano-Silica Powder.

3.10.5 CBR PLUS

CBR PLUS Soil Stabilizer and Dust Suppressant product is intended for use in diluted form, primarily as a soil compaction aid and stabilizer in the construction and maintenance of unpaved roads and construction sites. This product can also be used as a dust suppressant. CBR PLUS Soil Stabilizer and Dust Suppressant concentrate is an anionic synthetic compound, which changes the hydrophilic characteristics of clay materials into hydrophobic characteristics (Figure 3.9).

Figure 3.9 : CBR PLUS.

3.10.6 Zycosil

Zycosil Multi-Surface Sealer is a clear, water soluble, concentrated, penetrating sealer based on organosilane technology. Deep penetration and reaction with the substrate provides long-term waterproofing for up to 20 years. Zycosil Multi-Surface Sealer protects surfaces from structural and aesthetic damage such as efflorescence, and structural freeze thaw. It also reduces the damaging effects of water penetration to steel (Figure 3.10).

Figure 3.10 : Zycosil (Zydex Industries Ltd.).

4. RESULTS AND DISCUSSION

In this study different and vast numbers of tests on selected soil with various admixtures were tested at soil mechanic laboratory of Istanbul Technical University. Five different mixtures prepared and several tests such as Sieve Analysis, Hydrometer Analysis, Specific Gravity, Atterberg Limits, Standard Proctor, Consolidation, Permeability, Direct Shear and Triaxial tests were applied. In this section, all results will be shown and explain.

In these tests, selected five different Nano-Materials and Nano-Polymers such as Nano-Clay (NC), Nano-Carbon Fibers (C), Nano-Silica powder (Si), CBR PLUS, and Zycosil had added to the Clay. Afterward, each test was conducted with four different percentage of each Nano-Materials.

4.1 Soil Characterisation

4.1.1 Sieve and hydrometer analysis of soil

At the first step of classifying, sieve and hydrometer analysis were conducted on the chosen soil type. Almost, 95% of the specimen passed from #200 sieve, which shows that the soil should classify as a subdivision of fine-grained soils. Grain-Size distribution curve of the sample obtained as it shown below in Figure 4.1. According to the graph, the soil consists of 48% silt and 52% clay.

4.1.2 Atterberg limits and specific gravity of soil

During this research, the sample's Liquid Limit, Plastic Limit, and Plastic Index have been determined.

Because the results obtained from Casagrande test highly depend on the accuracy of the researcher, each test were repeated three times, and the water content correspond to 25 blows has been chosen.

As G_s is one of the important factors in different tests for calculations, specific gravity test has been done, as well. Attereberg limits and specific gravity of soil, which was

taken from Ciftalankoy in Istanbul is shown in Table 4.1. The results show that the sample is CH.

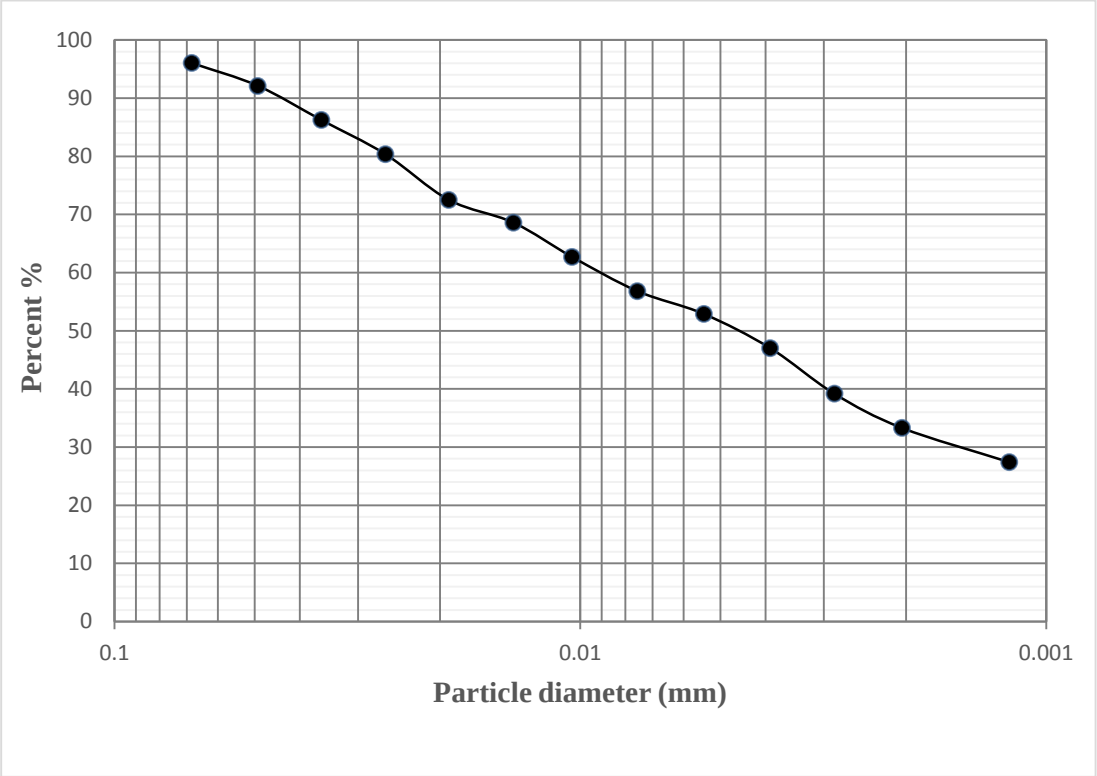


Figure 4.1 : Grain-size distribution of high Plasticity clay soil.

Table 4.1 : Atterberg Limits and Specific Gravity of Clay

Material	Lipid Limit	Plastic Limit	Plastic Index	Gs
Clay	60	31	29	2.75

4.2 Atterberg Limits of Mixed Soils

The result of Atterberg limit test done on soil samples wih different percentages of Nano-Materials and Nano-Polymers are presented in figure 4.2 and Table 4.2. In general, as shown in the figures adding additives to the clay has caused increasing liquid and plastic limits. Especially in Nano-Materials the increasing of plastic limits is more than liquid limit; consequently these changes cause to reduce plasticity index of soil(PI). On the other hand, as changes are so high, in soil with Zycosil Nano-Polymer all parameters increase dramatically and PI increases as well. Also as Nano-Additives percentages in soil increases, the role of changes improves as well. This effect cannot be used in the soil with high plasticity since this kind of soil exposes to

shrinkage after getting dried (Gallagher et al., 2007) and show high hydraulic conductivity which is harmful to some soil structures.

Nano-materials influence plasticity features of soil in three conditions:

1- The area of the large specific surface and superficial loads: superficial loads of Nano-Particles usually are connected with hydrated Cation. The particular large surface causes extensive intra-particle interactions (Zhang, 2007). Nano-Materials particles as compared to clay particle of micron size differentiate too much in a few feature like the area of specific surface, superficial loads and Nano-porosity.

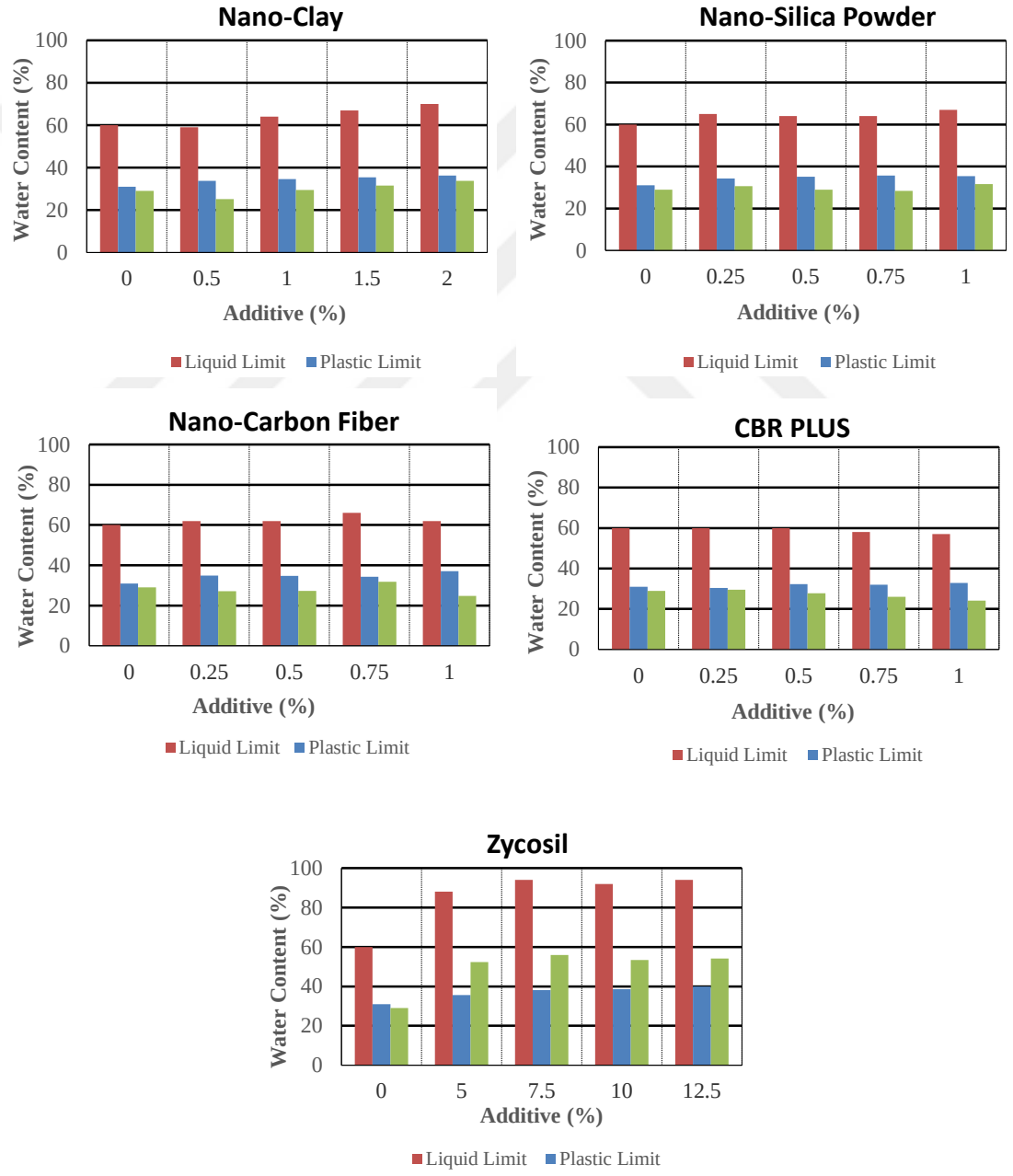


Figure 4.2 : Results of Atterberg Limit.

2- Nano-Porosity of intra-particle usually exist in Nano-holes of intraparticle due to absorbing and hydration within formation process of mineral water. Although this amount of water cannot cooperate or take part in. Superficial interaction of particles with particles so specially and prominently, it spoils within drying process in warming room; thus it can work and participate in significant Atterberg limits of porosity Nano-particles (Zhang, 2007).

3- Microstructure in compacted and mass form are firm and tight having some holes and filled up with water; they hardly ever scatter and fall while soil plastic tests are done for measuring Atterberg limits. Similar water existing inside intraparticle holes and laid water inside compacted structure in Atterberg limits cooperate and participate (Zhang, 2007).

Table 4.2 : Final Results of Atterberg Limits.

Material	Liquid Limit	Plastic Limit	Plastic Index
Clay	60	31	29
CBR PLUS 0.25%	60	30	30
CBR PLUS 0.5%	60	32	28
CBR PLUS 0.75%	58	32	26
CBR PLUS 1%	57	33	24
Nano-Carbon Fiber (0.25%)	62	35	27
Nano-Carbon Fiber (0.5%)	62	35	27
Nano-Carbon Fiber (0.75%)	66	34	32
Nano-Carbon Fiber (1%)	62	37	25
Nano-Clay 0.5%	59	34	25
Nano-Clay 1%	64	35	29
Nano-Clay 1.5%	67	35	32
Nano-Clay 2%	70	36	34
Nano-Silica Powder 0.25%	65	34	31
Nano-Silica Powder 0.5%	64	35	29
Nano-Silica Powder 0.75%	64	36	28
Nano-Silica Powder 1%	67	35	32
Zycosil 5%	88	36	52
Zycosil 7.5%	94	38	56
Zycosil 10%	92	39	53
Zycosil 12.5%	94	40	54

4.3 Harvard Miniature Compaction Test

Because Harvard Miniature Compaction compacts the sample with a different amount of energy in comparison with Standard Proctor device, the Harvard Miniature Compaction Apparatus was calibrated to get the same answer obtained from Standard Proctor Device. For calibration, at first, standard proctor test had been applied on the clay and then four different compaction tests with Harvard Compaction Equipment ,with different number of blows and heigh of fall, on the same soil were performed. Afterward, the number af blows and height of fall that compact the soil in the same rates in compare with standard proctor test had been chosen. Results of the standard Proctor and Harvard Miniature Compaction tests are shown in Table 4.3.

Table 4.3 : Results of experiments performed to calibrate the Harvard Miniature Equipment.

No.	Description	$\gamma_d(kN/m^3)$	w_{opt} (%)	Blows	Height(cm)
1	Standard Proctor Test	14.5	28	25	30
2	First Calibration	15	24	22	25
3	Second Calibration	14	29	14	18
4	Third Calibration	14.4	28	17	20
5	Forth Calibration	14.8	27	20	20

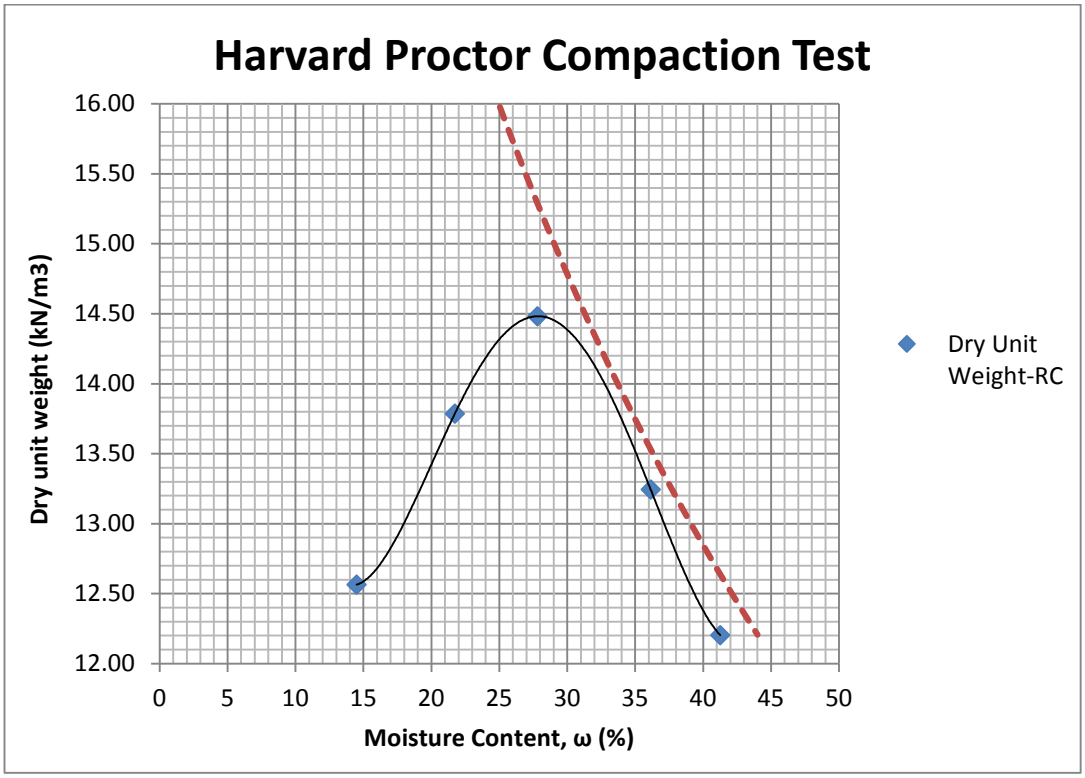


Figure 4.3 : Reference calibration's unconfined compression test results.

According to data obtained from all four experiments, the third calibration has been chosen as the reference calibration, and the corresponding curve is shown in Figure 4.3.

In this study, compaction test has been applied on 25 samples with four and six different percentage of five different additives, and for each sample, the dry unit weight has been determined at the optimum water content. Results are shown in Table 4.4.

Results in Table 4.4 show that additives do not affect optimum water content and dry unit weight of clay in considerable degree, except when large amounts of additives are used.

Table 4.4 : Results of Harvard Miniature Compaction Device.

Material	γd (kN/m3)	Wopt (%)
Clay	14.5	28
CBR PLUS 0.25%	14.3	28
CBR PLUS 0.5%	14.5	28
CBR PLUS 0.75%	14.6	28
CBR PLUS 1%	14.4	28
Nano-Carbon Fiber (0.05%)	14.3	27
Nano-Carbon Fiber (0.1%)	14.3	28
Nano-Carbon Fiber (0.25%)	14.4	29
Nano-Carbon Fiber (0.5%)	14.2	29
Nano-Carbon Fiber (0.75%)	14.1	28
Nano-Carbon Fiber (1%)	13.9	28
Nano-Clay 0.5%	14.3	28
Nano-Clay 1%	14.3	28
Nano-Clay 1.5%	14.3	28
Nano-Clay 2%	14.2	28
Nano-Silica Powder 0.05%	14.1	28
Nano-Silica Powder 0.1%	14.4	28
Nano-Silica Powder 0.25%	14.2	28
Nano-Silica Powder 0.5%	14.3	29
Nano-Silica Powder 0.75%	14.3	30
Nano-Silica Powder 1%	13.8	31
Zycosil 5%	14.3	28
Zycosil 7.5%	14.3	27
Zycosil 10%	14.2	26
Zycosil 12.5%	14.1	25

4.4 Unconfined Compression Test

For this research, it was aimed to evaluate the effectiveness of the stabilizers mixed with soil at the optimum moisture content. Low to high dosage rates were used for soils at optimum moisture content. Overall, around 300 specimens corresponding to each moisture content and dosage rate were created, cured, and subjected to the compressive strength testing. This section presents the results of unconfined compressive strength testing conducted on mixtures of CiftalanKoy Clay with Nano-Materials and Nano-Polymers at their optimum moisture contents. The data points represent the unconfined compressive strength for the samples at varying curing times, which are 1, 7, and 28 days. A summary of all unconfined compressive strengths for different mixed samples treated at optimum moisture content can be found in Table 4.5, Table 4.6, Table 4.7, Table 4.8, and Table 4.9.

Table 4.5 : Unconfined Compression Strengths for Clay-Si Mixed Samples.

Curing Time	Material	Percentage	Unconfined Strength (kPa)
1 Day	Clay	0	309.76
	SI	0.05	317.50
	SI	0.1	319.05
	SI	0.25	328.04
	SI	0.5	336.09
	SI	0.75	319.40
	SI	1	288.08
Curing Time	Material	Percentage	Unconfined Strength (kPa)
7 days	Clay	0	309.76
	SI	0.05	317.69
	SI	0.1	319.98
	SI	0.25	334.54
	SI	0.5	340.74
	SI	0.75	323.08
	SI	1	291.17
Curing Time	Material	Percentage	Unconfined Strength (kPa)
28 Days	Clay	0	309.76
	SI	0.05	317.81
	SI	0.1	321.84
	SI	0.25	337.02
	SI	0.5	343.12
	SI	0.75	325.25
	SI	1	292.72

Table 4.6 : Unconfined Compression Strengths for Clay-Nc Mixed Samples.

Curing Time	Material	Percentage	Unconfined Strength (kPa)
1 Day	Clay	0	309.76
	Nc	0.5	310.10
	Nc	1	346.93
	Nc	1.5	381.00
	Nc	2	331.44
Curing Time	Material	Percentage	Unconfined Strength (kPa)
7 days	Clay	0	309.76
	Nc	0.5	348.33
	Nc	1	340.74
	Nc	1.5	387.20
	Nc	2	332.99
Curing Time	Material	Percentage	Unconfined Strength (kPa)
28 Days	Clay	0	309.76
	Nc	0.5	314.41
	Nc	1	341.36
	Nc	1.5	389.68
	Nc	2	333.61

Table 4.7 : Unconfined Compression Strengths for Clay-C Mixed Samples.

Curing Time	Material	Percentage	Unconfined Strength (kPa)
1 Day	Clay	0	309.76
	C	0.05	312.55
	C	0.1	330.05
	C	0.25	349.61
	C	0.5	334.54
	C	0.75	312.24
	C	1	298.92
Curing Time	Material	Percentage	Unconfined Strength (kPa)
7 days	Clay	0	309.76
	C	0.05	312.86
	C	0.1	332.88
	C	0.25	352.64
	C	0.5	336.09
	C	0.75	312.55
	C	1	288.08
Curing Time	Material	Percentage	Unconfined Strength (kPa)
28 Days	Clay	0	309.76
	C	0.05	315.96
	C	0.1	336.96
	C	0.25	354.68
	C	0.5	340.12
	C	0.75	316.57
	C	1	294.27

Table 4.8 : Unconfined Compression Strengths for Clay-CBR PLUS Mixed Samples.

Curing Time	Material	Percentage	Unconfined Strength (kPa)
1 Day	Clay	0	309.76
	CBR PLUS	0.25	312.86
	CBR PLUS	0.5	314.41
	CBR PLUS	0.75	319.05
	CBR PLUS	1	315.96
Curing Time	Material	Percentage	Unconfined Strength (kPa)
7 days	Clay	0	309.76
	CBR PLUS	0.25	313.79
	CBR PLUS	0.5	315.34
	CBR PLUS	0.75	320.60
	CBR PLUS	1	316.57
Curing Time	Material	Percentage	Unconfined Strength (kPa)
28 Days	Clay	0	309.76
	CBR PLUS	0.25	322.15
	CBR PLUS	0.5	315.65
	CBR PLUS	0.75	321.22
	CBR PLUS	1	317.50

Table 4.9 : Unconfined Compression Strengths for Clay-ZY Mixed Samples.

Curing Time	Material	Percentage	Unconfined Strength (kPa)
1 Day	Clay	0	309.76
	ZY	5	393.70
	ZY	7.5	347.93
	ZY	10	303.57
	ZY	12.5	291.13
Curing Time	Material	Percentage	Unconfined Strength (kPa)
7 days	Clay	0	309.76
	ZY	5	403.07
	ZY	7.5	348.68
	ZY	10	302.15
	ZY	12	291.55
Curing Time	Material	Percentage	Unconfined Strength (kPa)
28 Days	Clay	0	309.76
	ZY	5	408.88
	ZY	7.4	348.56
	ZY	10	301.87
	ZY	12.5	297.71

Figure 4.4, Figure 4.5, and Figure 4.6 show the results in a chart with different curing times for different soil mixtures.

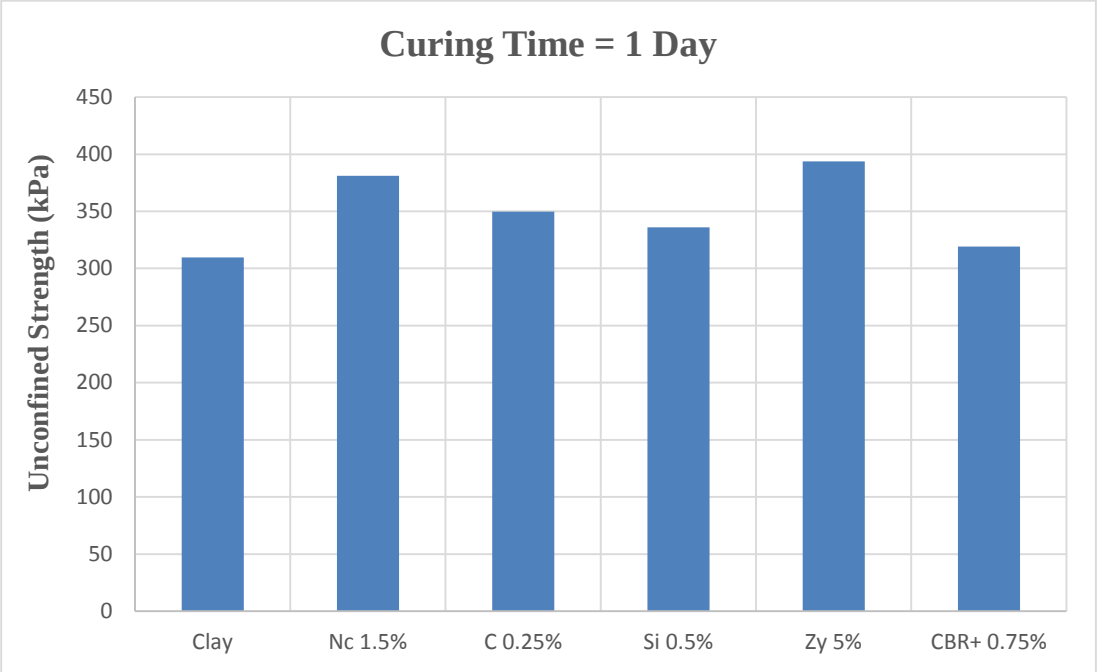


Figure 4.4 : Maximum Unconfined Compression Strengths for Mixed Samples – 1 Day Curing.

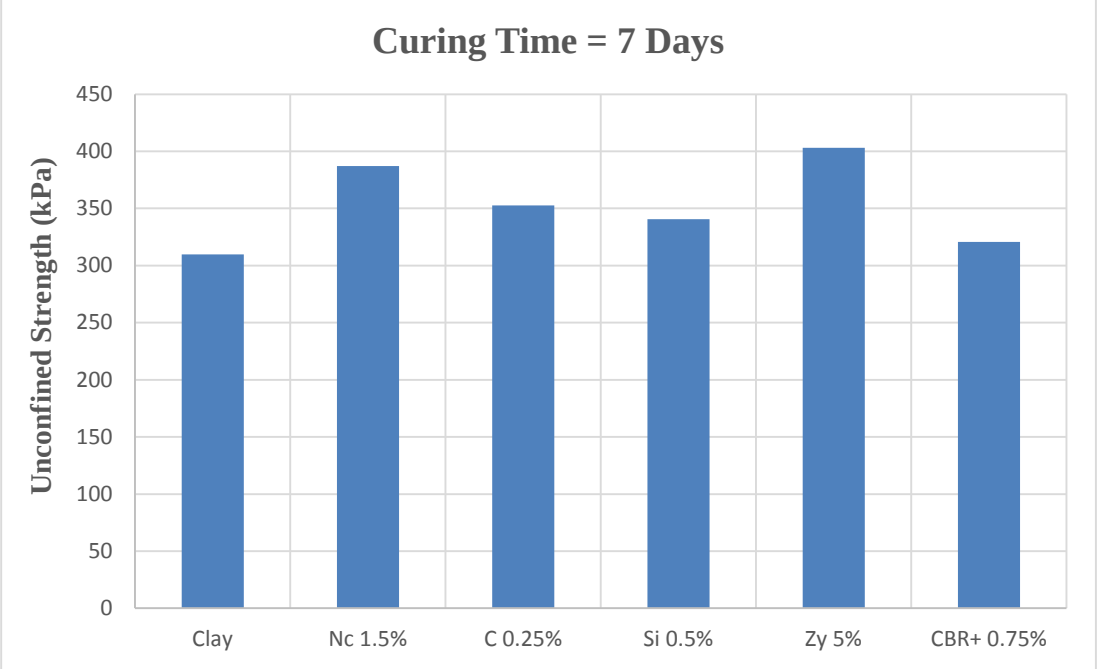


Figure 4.5 : Maximum Unconfined Compression Strengths for Mixed Samples – 7 Days Curing.

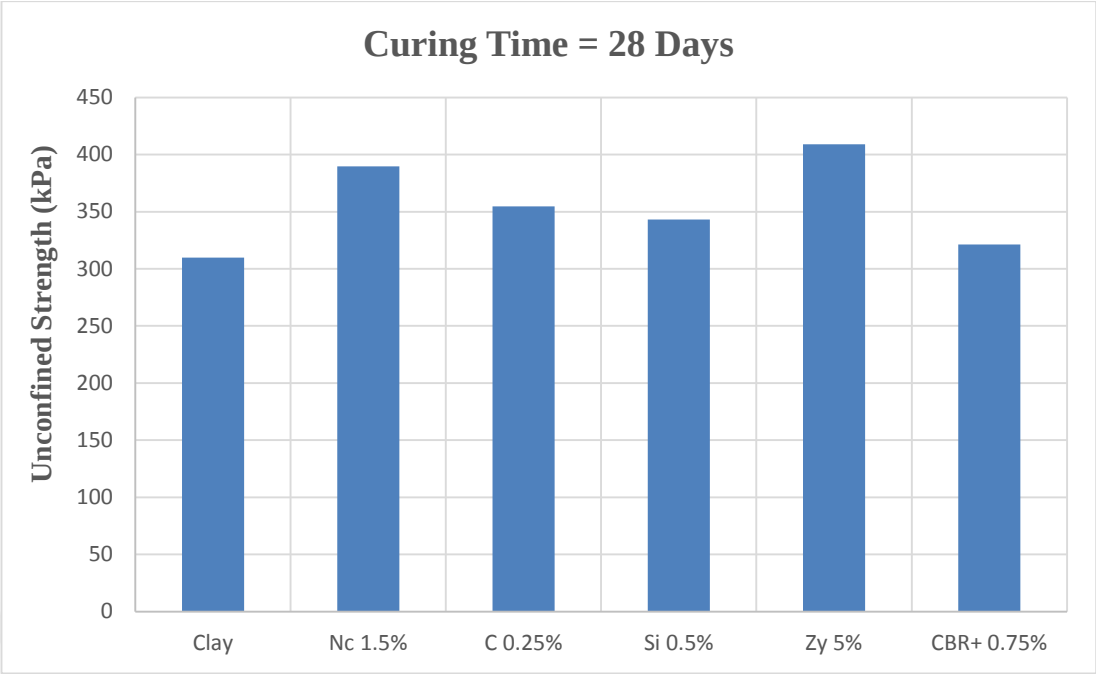


Figure 4.6 : Maximum Unconfined Compression Strengths for Mixed Samples – 28 Days Curing.

For Nano-Materials base treatment, specimens treated with 5% Zycosil achieved the greatest strengths for all curing times, followed by 1.5% Nano-Clay, 0.25% Nano-Carbon Fibers, 0.5% Nano-silica Powder and 0.75% CBR PLUS. Specimens treated with 5% Zycosil achieved a 28-day compressive strength of 409 kPa, resulting in a strength increase of 132% from the untreated strength of 310 kPa. Treatment with 1.5% Nano-Clay produced results, which is attaining a 28-day strength of 390 kPa, equating to a strength increase of 126%. The compressive strength achieved using 0.25% Nano-Carbon Fibers was 354 kPa, resulting in a 115% strength increase. Treatment with 0.5% Nano-silica Powder had a weak effect which increased the strength to 343 kPa, means around 111% increment in unconfined strength. Treatment with 0.75% CBR PLUS was the least effective of the Nano-Polymer tested, resulting in a compressive strength of 321 kPa and a strength increase of only 104%.

Curing time did not have an extreme impact on the strength of the Nano-Material base treated samples. The strength of the specimens tested at a curing time of one day achieved 90% to 95% of the 28-day strength, and reached 92% to 96% of the 28-days strength at seven days.

After this, with respect of results obtained from unconfined compression test, because of vast amount of materials and tests, experiments continued with the percentages of Nano-Materials and Nano-Polymers, which had showed the maximum effect.

4.5 Soaked CBR Test

The effect of soaking on the CBR value of a mixed clay was studied here. Twelve CBR samples were prepared at 98% relative standard AASHTO compaction at their corresponding water content. One CBR sample was prepared for each sample for soaking period of 10 days. These samples were prepared and compacted mechanically in the laboratory. Swelling degrees measured for 4 and 10 days. At the end of the 10^{th} day, samples were tested for measuring load rate of penetration. Results are summarized in Table 4.10, Table 4.11, Table 4.12, Table4.13, Figure 4.7, and Figure 4.8.

Table 4.10 : Swelling Rates of Soaked CBR.

Material	Swelling Rate After 4 Days	Swelling Rate After 10 days
Clay	8.0%	8.0%
Zycosil (12.5%)	7.9%	8.5%
Zycosil (10%)	7.2%	7.9%
Zycosil (7.5%)	7.2%	8.0%
Zycosil (5%)	6.8%	8.2%
CBR PLUS (1%)	5.7%	7.4%
CBR PLUS (0.75%)	5.9%	8.1%
CBR PLUS (0.5%)	6.5%	8.9%
CBR PLUS (0.25%)	7.2%	9.1%
Nano-Clay (1.5%)	6.4%	8.3%
Nano-Carbon Fiber (0.25%)	5.3%	7.5%
Nano-Silica Powder (0.5%)	6.7%	8.0%

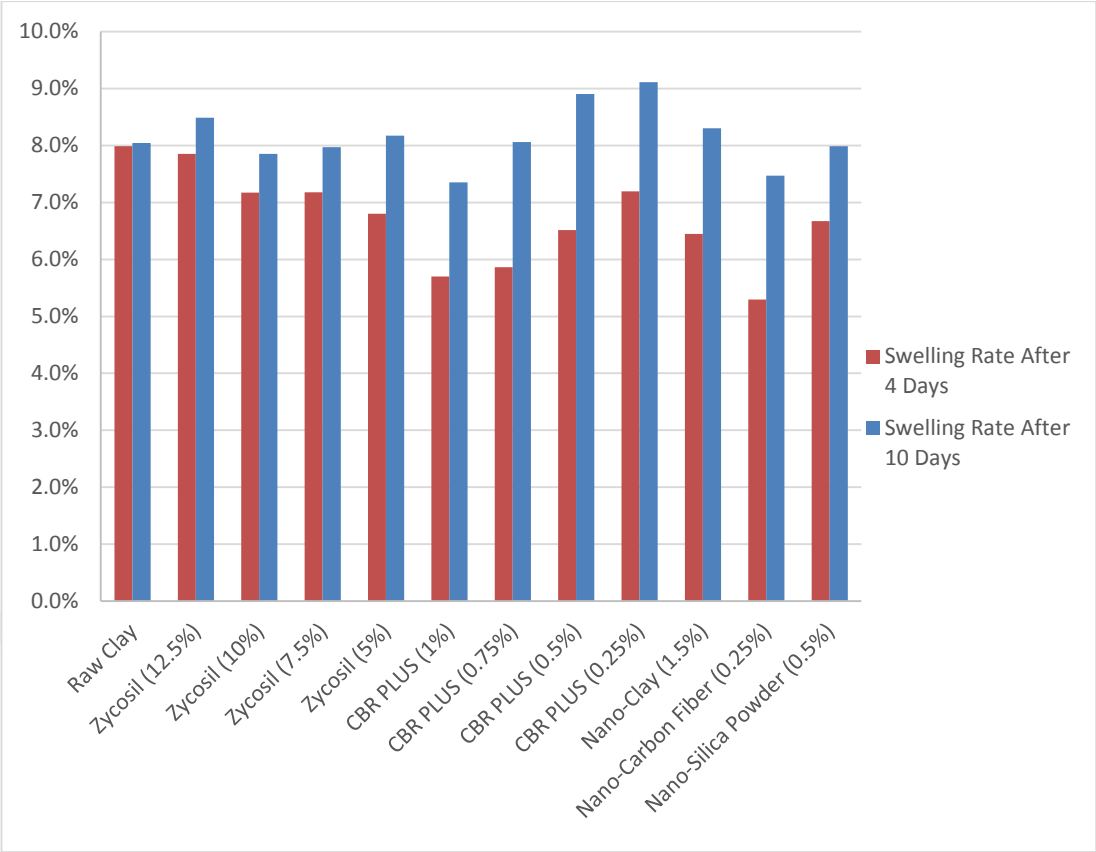


Figure 4.7 : Swelling Rates of Soaked CBR.

Table 4.11 : Penetration Load Rate of Soaked CBR.

Material	Percentage	CBR Value @2.5 mm (%)	CBR Value @5 mm (%)
Clay	0.00%	0.2	0.8
Zycosil	12.50%	1.4	2.9
Zycosil	10.00%	1.6	3.0
Zycosil	7.50%	1.4	2.6
Zycosil	5.00%	1.6	2.8
CBR PLUS	1.00%	0.8	1.2
CBR PLUS	0.75%	1.0	1.5
CBR PLUS	0.50%	0.7	1.3
CBR PLUS	0.25%	0.9	1.3
Nano-Clay	1.50%	0.6	1.5
Nano-Carbon Fiber	0.25%	0.9	1.5
Nano-Silica Powder	0.50%	1.1	1.6

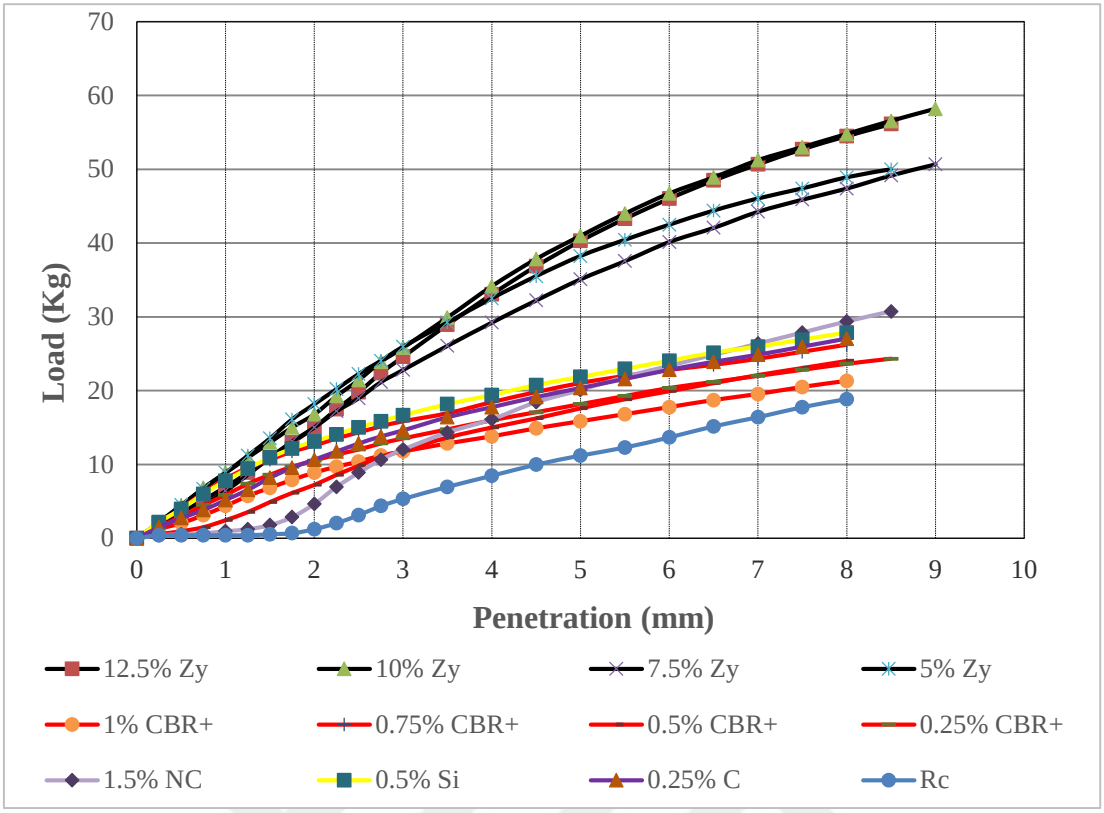


Figure 4.8 : Penetration Rate of Soaked CBR.

Table 4.12 : Materials With The Maximum Effect on Penetration Load Rate of Soaked CBR.

CBR Value	Material	Percentage
Max CBR Value@2.5 mm	Zycosil 10%	1.60%
Max CBR Value@5 mm	Zycosil 10%	3%

Table 4.13 : Materials With The Maximum Effect on swelling Rate of Soaked CBR.

Swelling Rate	Material	Percentage
Min Swelling for 4 Days	Nano-Carbon Fiber (0.25%)	5.3%
Min. Swelling for 10 Days	CBR PLUS (1%)	7.4%

For swelling: with respect to Table 4.10 and figure 4.7, 0.25% Nano-Carbon fibers surprisingly has the best effect on preventing the clay from swelling. It decreases the swelling about 45% in 4 days and 8% in 10 days, but it also extends the days to reach the maximum swelling, because as it shown in data normal clay finishes its swelling in 4 days. The next more effective material on swelling was 1% CBR PLUS which has decreased the amount of swelling by 30% and 9% in 4 days and 10 days respectively. Zycosil was the next effective material but it was not so much noticeable like others. Other materials do not have considerable effect on swelling rate of clay. But the

common effects among Nano-Material based additive was extending the time to reach the final amount of swelling.

For CBR Value: Zycosil has the best effect on amount of CBR value. Zycosil increases it around eight times for 2.5 mm penetration and four times for 5 mm penetration, with additive amount of 5 percent and 10 percent respectively. Actually, the hydrophobic nature of Zycosil cause this results which prevent clays from absorbing water and getting loose. Other Nano-Material based additive do not have considrable effect on penetration load rate.

4.6 Consolidation and Permeability Test

Constant rate of deformation consolidation and permeability tests were performed on 12 remolded samples compacted with Harvard miniature compaction device at optimum water content, which are mixture of clay with different Nano-Material base additives. The test matrix was developed to investigate the following major aspects:

1- Determination of effects of Nano-Materials and Nano-Polymers on preconsolidation and free-swelling pressure of clay.

2- Determination of effects of Nano-Materials and Nano-Polymers on Cc-compression index of clay.

3- Variability of c_v - coefficient of consolidation and permeability in presence of different Nano-Material base additives.

4.6.1 Consolidation results

The preconsolidation stresses were evaluated from the e-Log P curves in accordance with the Casagrande procedure (Casagrande, 1936). As a reference, consolidation curve of clay is shown in Figure 4.9. For clay preconsolidation pressure is about 220 kPa and the swelling pressure is 110 kPa. Nano-Materials and Nano-Polymers do not make any especial change on pre-consolidation pressure, but as in Soaked CBR test, it had been observed that swelling pressure could highly be under effect of these Nano-Material base admixtures, as well as Cc-compression index. Summary of these results are shown in Table 4.14 for swelling pressure and in Table 4.15 for Cc-compression index under $\Delta P = 400$ kPa.

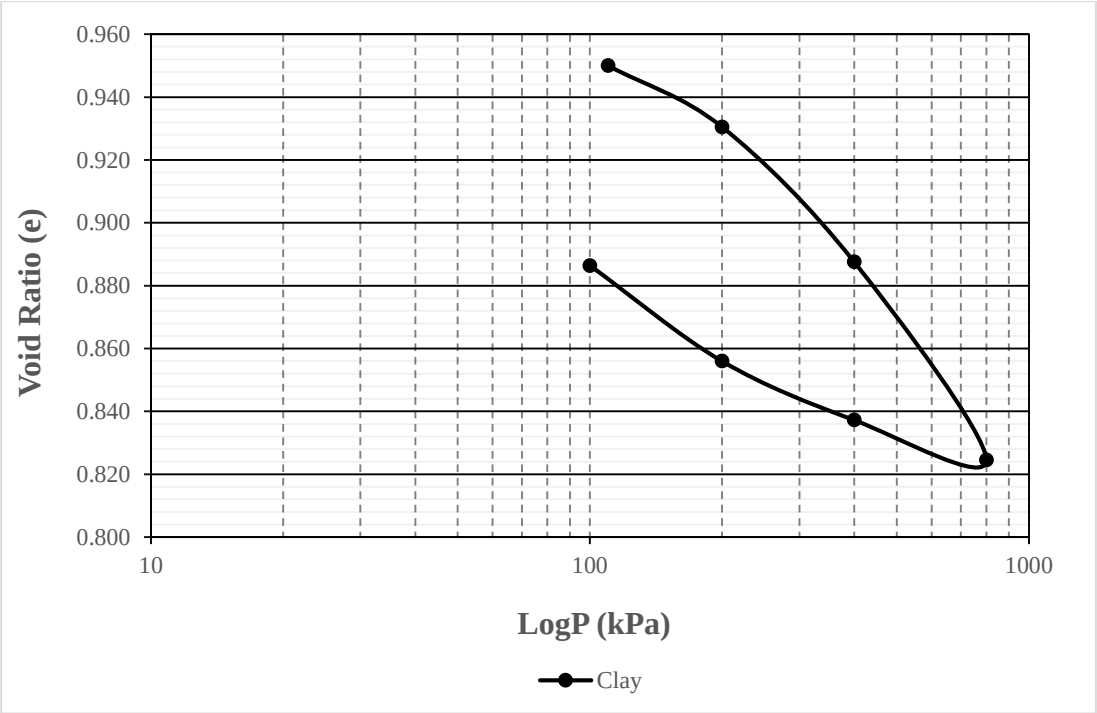


Figure 4.9 : e-Log σ' curve of Clay.

Table 4.14 : Swelling Pressure.

Material	Swelling Pressure (kPa)
Clay	110
CBR PLUS 0.25%	75
CBR PLUS 0.5%	75
CBR PLUS 0.75%	70
CBR PLUS 1%	100
Nano-Carbon Fiber (0.25%)	72
Nano-Clay 1.5%	100
Nano-Silica Powder 0.5%	100
Zycosil 5%	85
Zycosil 7.5%	85
Zycosil 10%	85
Zycosil 12.5%	85

Table 4.15 : Compression Index (Cc) under ΔP = 400 kPa.

Material	Compression Index (Cc)
Clay	0.21
CBR PLUS 0.25%	0.21
CBR PLUS 0.5%	0.21
CBR PLUS 0.75%	0.20
CBR PLUS 1%	0.18
Nano-Carbon Fiber (0.25%)	0.21
Nano-Clay 1.5%	0.20
Nano-Silica Powder 0.5%	0.19
Zycosil 5%	0.19
Zycosil 7.5%	0.22
Zycosil 10%	0.23
Zycosil 12.5%	0.26

As it is obvious from results and base on Table 4.15, neither Nano-Materials nor Nano-Polymers cannot have any special positive effect on consolidation of clay because compression index of the soil, which is needed to calculate the amount of consolidation, does not change noticeably. On the other hand, regarding swelling pressure Nano-Carbon Fibers and CBR PLUS could reduce it around 35%, which is really considerable for only 0.25% and 1% of additive, respectively. Zycosil reduces the swelling pressure around 20%, but according to Table 4.15, it slightly increases the amount of settlement at the same time.

4.6.2 Permeability results

Permeability determinations were made at different stages of consolidation during each test to define the variation of deformation with void time under different loads. The coefficients of consolidation, c_v (m^2/s), determined from the logarithm-of-time method proposed by Casagrande and Fadum (1940) for ΔP=200 kPa and ΔP=400 kPa. Also, Hydraulic Conductivity (k (m/s)) has calculated using coefficients of consolidation. The results of these calculations are shown in Table 4.16, Table 4.17, Figure 4.10, and Figure 4.11.

Table 4.16 : c_v and k under ΔP=200 kPa.

Material	Clay	CBR PLUS 0.75%	Nano-Clay 1.5%	Carbon 0.25%	Silica 0.5%	Zycosil 5%	Zycosil 10 %
t50 (min)	3	2.7	4	4	3.2	4.5	2.5
Hdr (cm)	1	1	1	1	1	1	1
Cv (m^2/s)	1.10E-07	1.22E-07	8.22E-08	8.22E-08	1.03E-07	7.30E-08	1.31E-07
mv (m^2/kg)	1.10E-06	1.00E-06	9.00E-07	1.00E-06	8.00E-07	8.00E-07	1.00E-06
k (m/s)	1.20E-10	1.22E-10	7.40E-11	8.22E-11	8.24E-11	5.84E-11	1.31E-10

Table 4.17 : c_v and k under ΔP=400 kPa.

Material	Clay	CBR PLUS 0.75%	Nano-Clay 1.5%	Carbon 0.25%	Silica 0.5%	Zycosil 5%	Zycosil 10 %
t50 (min)	3.1	2.3	3.4	3	3.4	2.8	1.4
Hdr (cm)	1	1	1	1	1	1	1
Cv (m^2/s)	1.06E-07	1.43E-07	9.65E-08	1.10E-07	9.65E-08	1.17E-07	2.35E-07
mv (m^2/kg)	8.00E-07	8.00E-07	8.00E-07	8.00E-07	7.00E-07	7.00E-07	9.00E-07
k (m/s)	8.48E-11	1.14E-10	7.72E-11	8.76E-11	6.76E-11	8.19E-11	2.12E-10

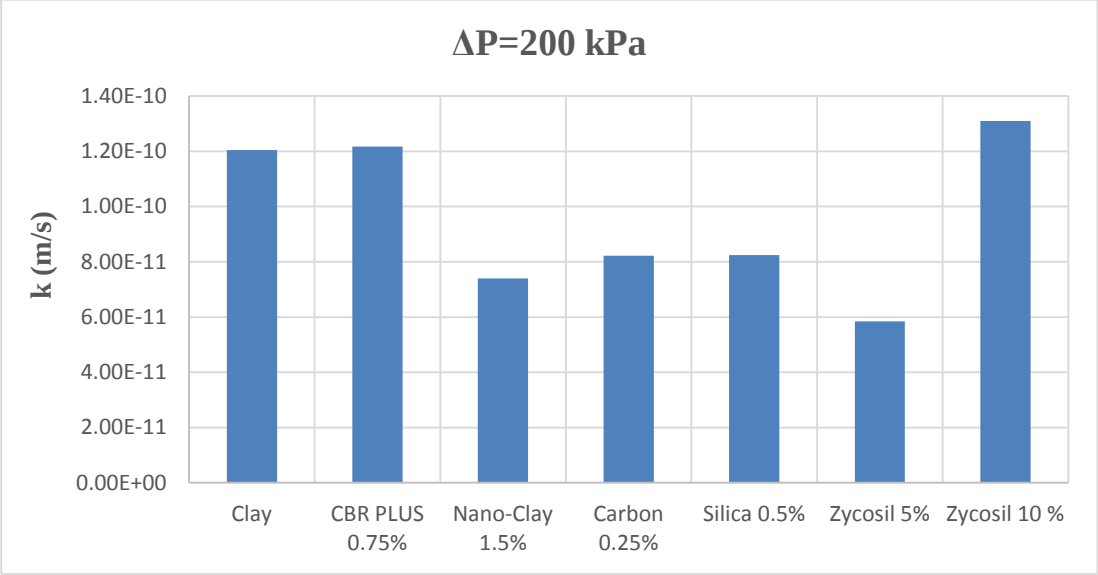


Figure 4.10 : Hydraulic Conductivity (k (m/s)) under ΔP=200 kPa.

From data gained from 5% of Zycosil has a huge effect on hydraulic conductivity and reduces it around 50% under ΔP=200 kPa, but on the other hand when the percentages of Zycosil increases to 10% hydraulic conductivity of mixture increase suddenly and reach to 110% of clay. The next material was Nano-Clay which decreases the permeability about 35%.Other Nano-Materials except 0.75% CBR PLUS decrease the permeability around 30%. For ΔP=400 kPa, again 10% of Zycosil dramatically increase the permeability around 240%, and Nano-Silica Powder showed the best effect with reduces the permeability around 20%.

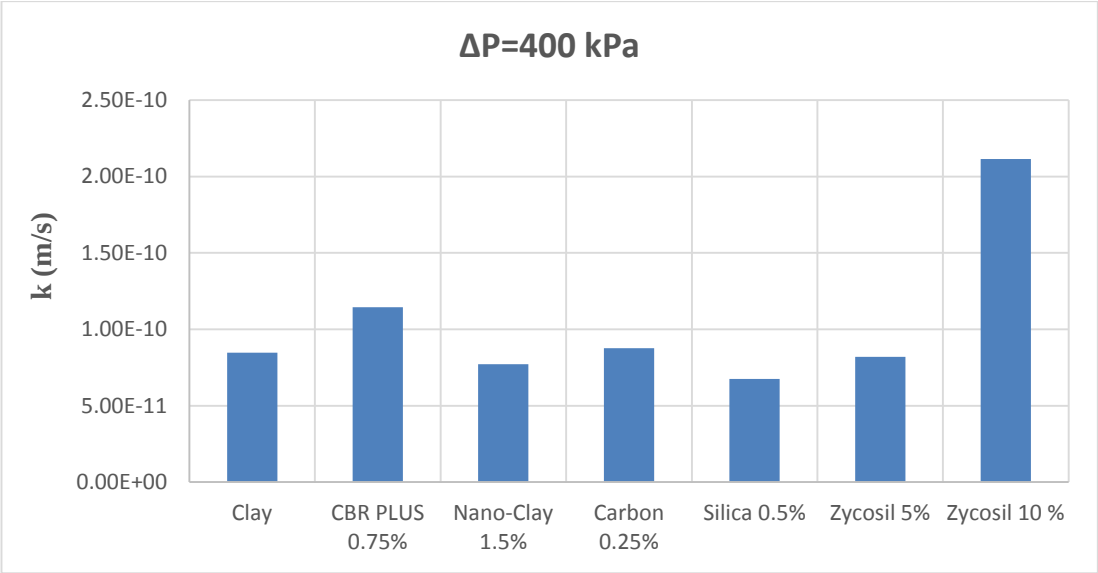
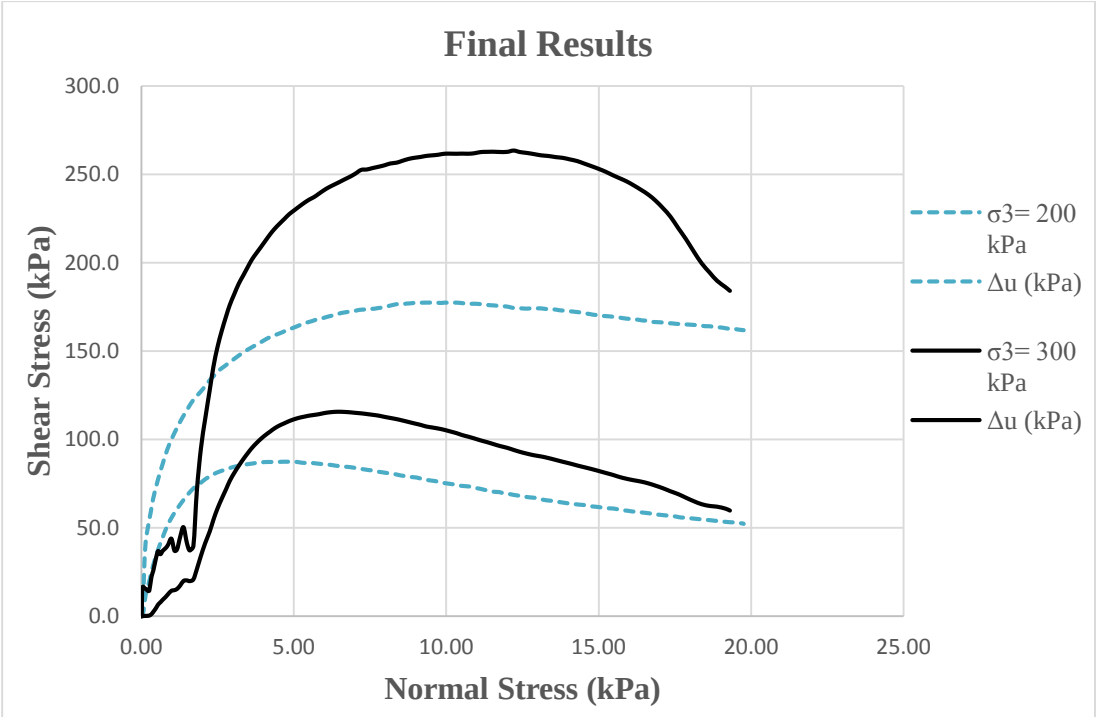


Figure 4.11 : Hydraulic Conductivity (k (m/s)) under ΔP=400 kPa.

Like ΔP=200 kPa, other Nano-Materials except 0.75% CBR PLUS decrease the permeability around 10-15%. Actually, during preparation of samples it was obvious that permeability will rise with increasment in percentage of Zycosil, due to chemical reactions among clay's particles, water and Zycosil the amount of voids in sample were increasing, which has proved before by reducing in amount of dry unit weight of clay.

4.7 Consolidate-Undrained Triaxial Test

This section contains data plots from consolidated-undrained triaxial compression test results values of strain (20 %) and two different confining stresses (200 and 300 kPa), which has been applied on Clay, 5% of Zycosil-Rc, 0.25% Nano-Carbon Fibers-Rc, 0.5% Nano-Silica-Rc, 0.75% CBR PLUS-Rc and 1.5% Nano-Clay-Rc mixtures. Figures 4.12 - 4.17 were drawn from data acquired from the data acquisition system attached to the triaxial system. These figures show load per unit area and pore water pressure plotted against unit strain. Clearly, Figures include the stress-strain trend at different values of confining pressure. In case of 200 and 300 kPa of all-around pressure, positive pore water pressure was developed, which indicates that the clay specimens under these certain circumstances were normally consolidated and this behavior is expected for such kind of clays. Results are summarized in Table.4.18.



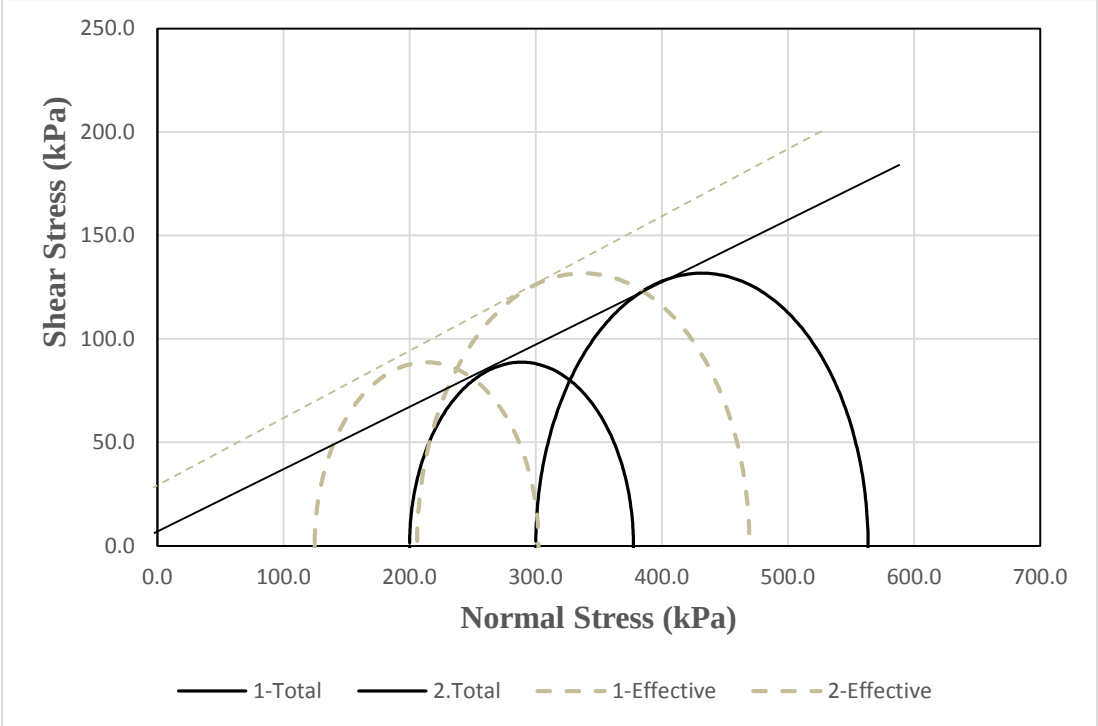
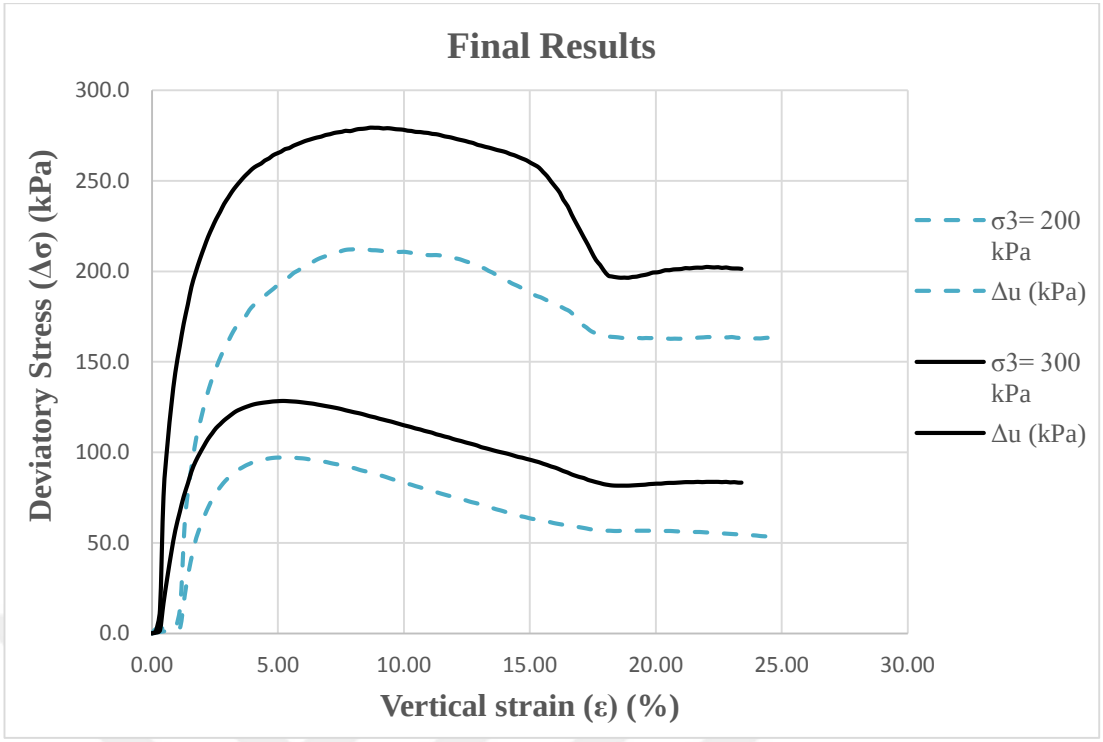


Figure 4.12 : Results Obtained from CU Test – Clay.



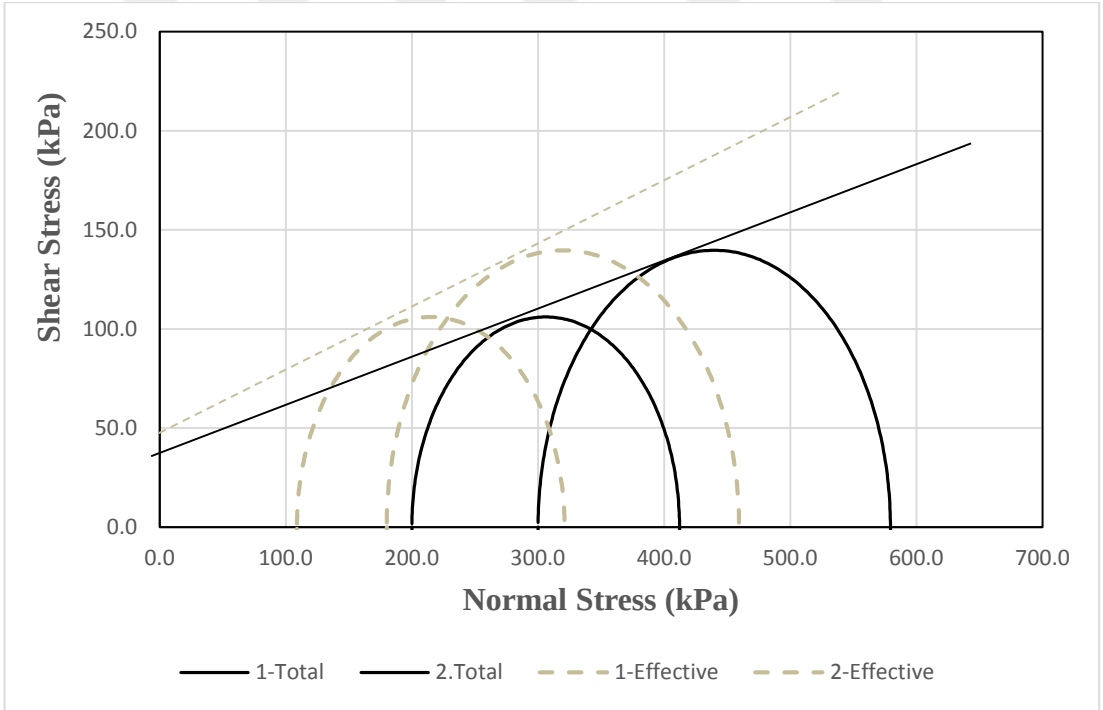
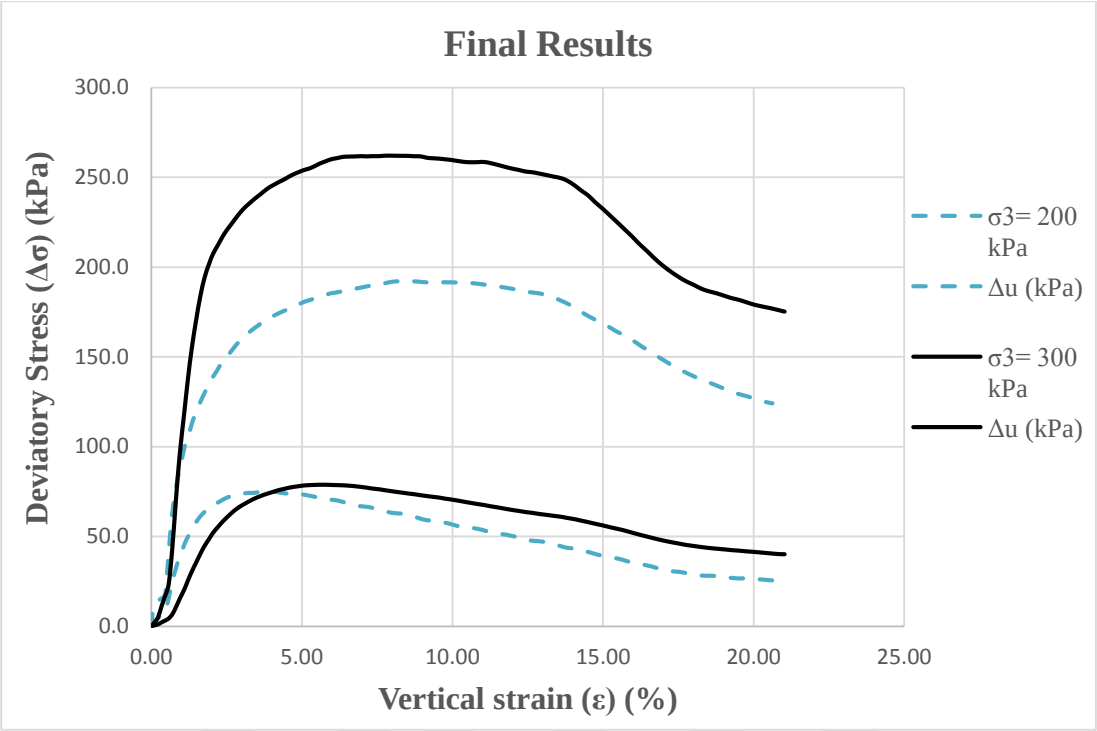


Figure 4.13 : Results Obtained from CU Test – 5% Zycosil-Clay Mixture.



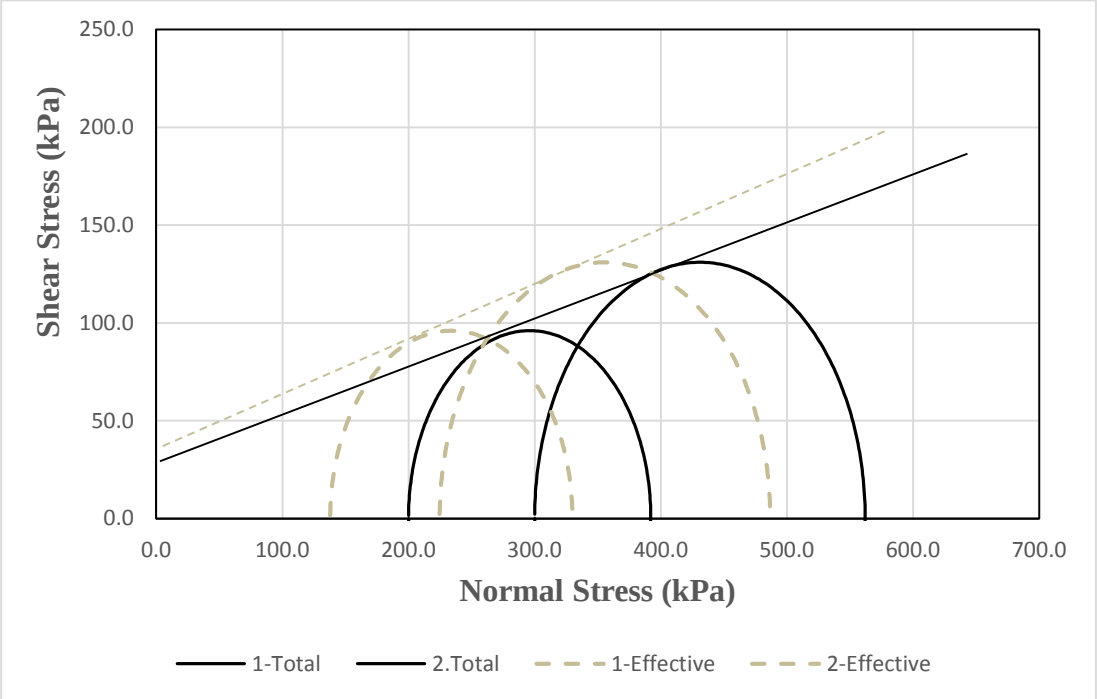
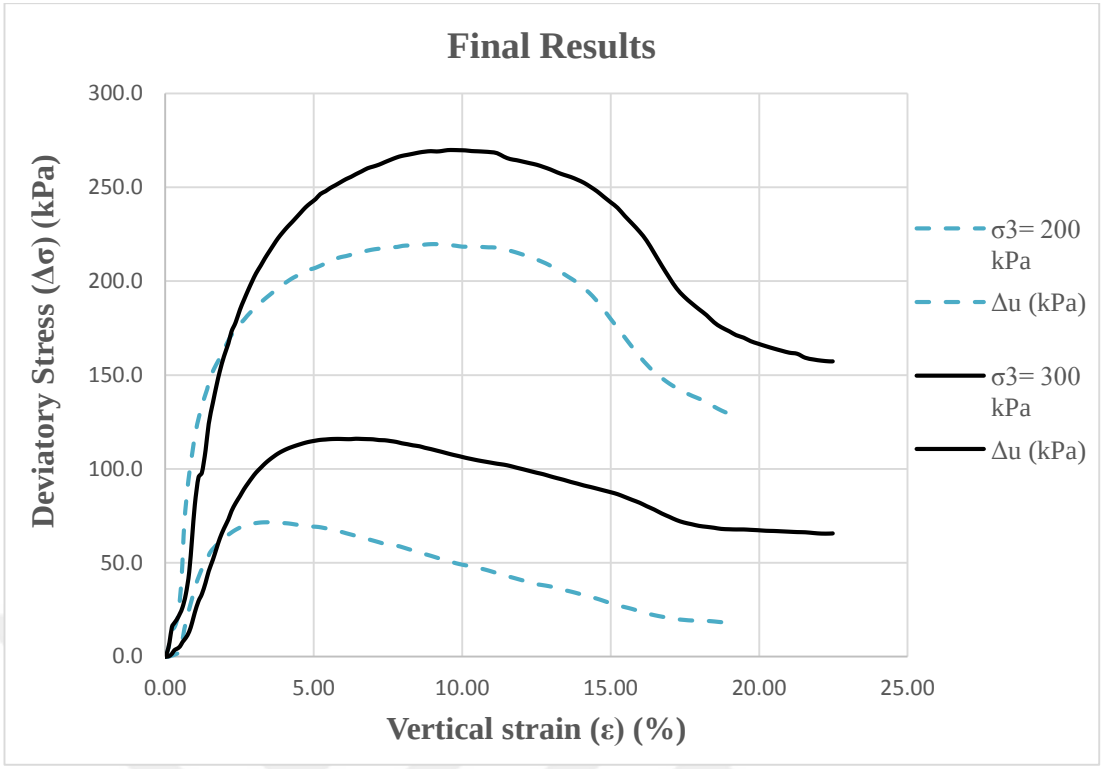


Figure 4.14 : Results Obtained from CU Test – 0.25% Nano-Carbon Fibers - Clay Mixture.



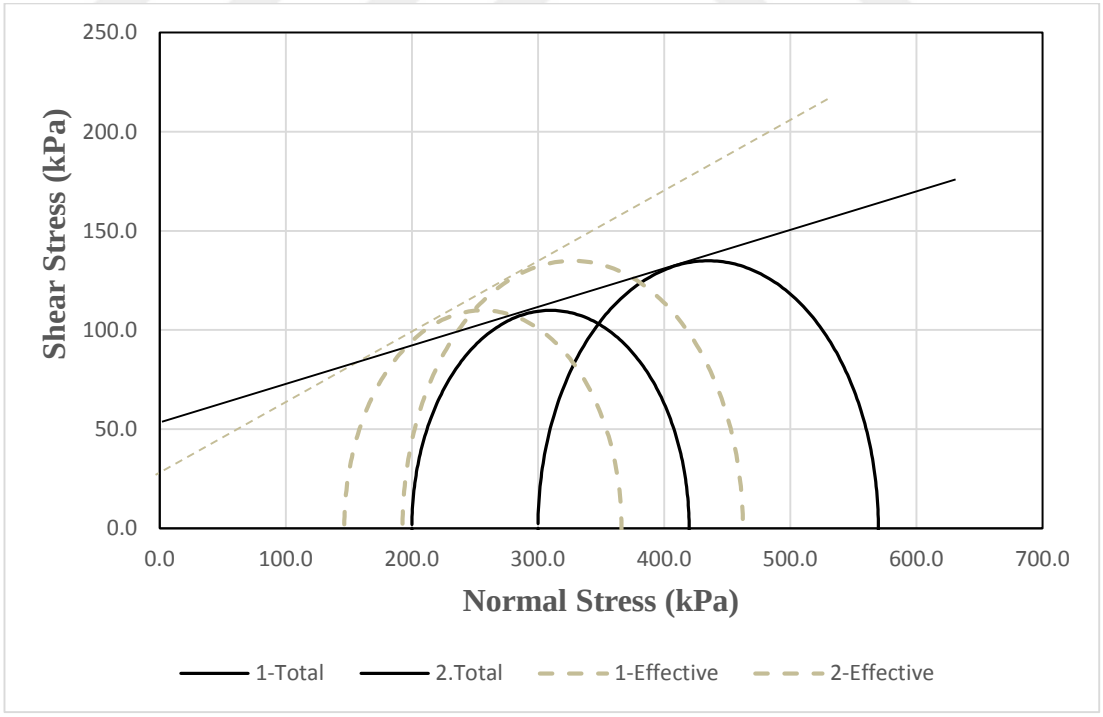
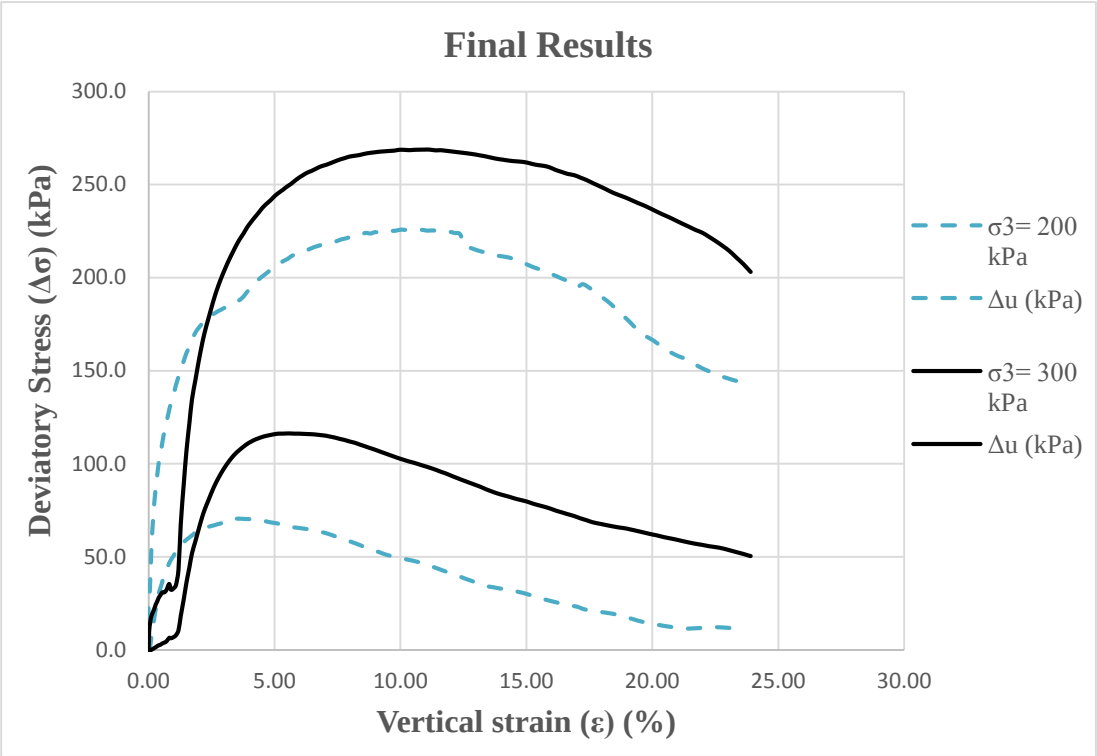


Figure 4.15 : Results Obtained from CU Test – 0.5% Nano-Silica Powder - Clay Mixture.



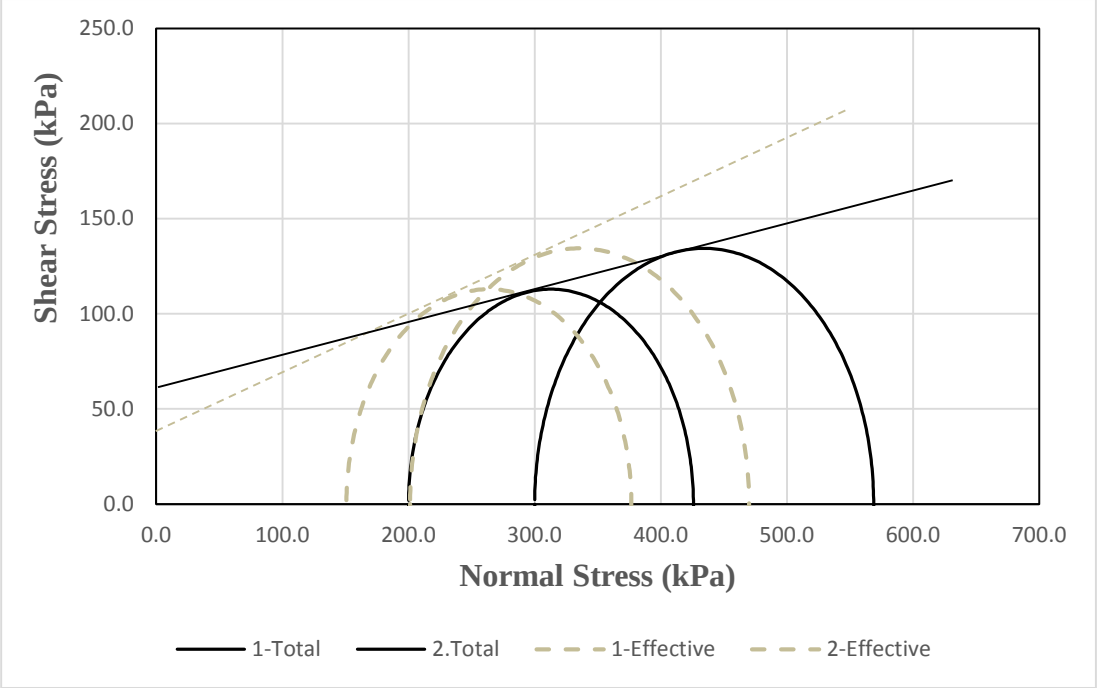


Figure 4.16 : Results Obtained from CU Test – 0.75% CBR PLUS - Clay Mixture.

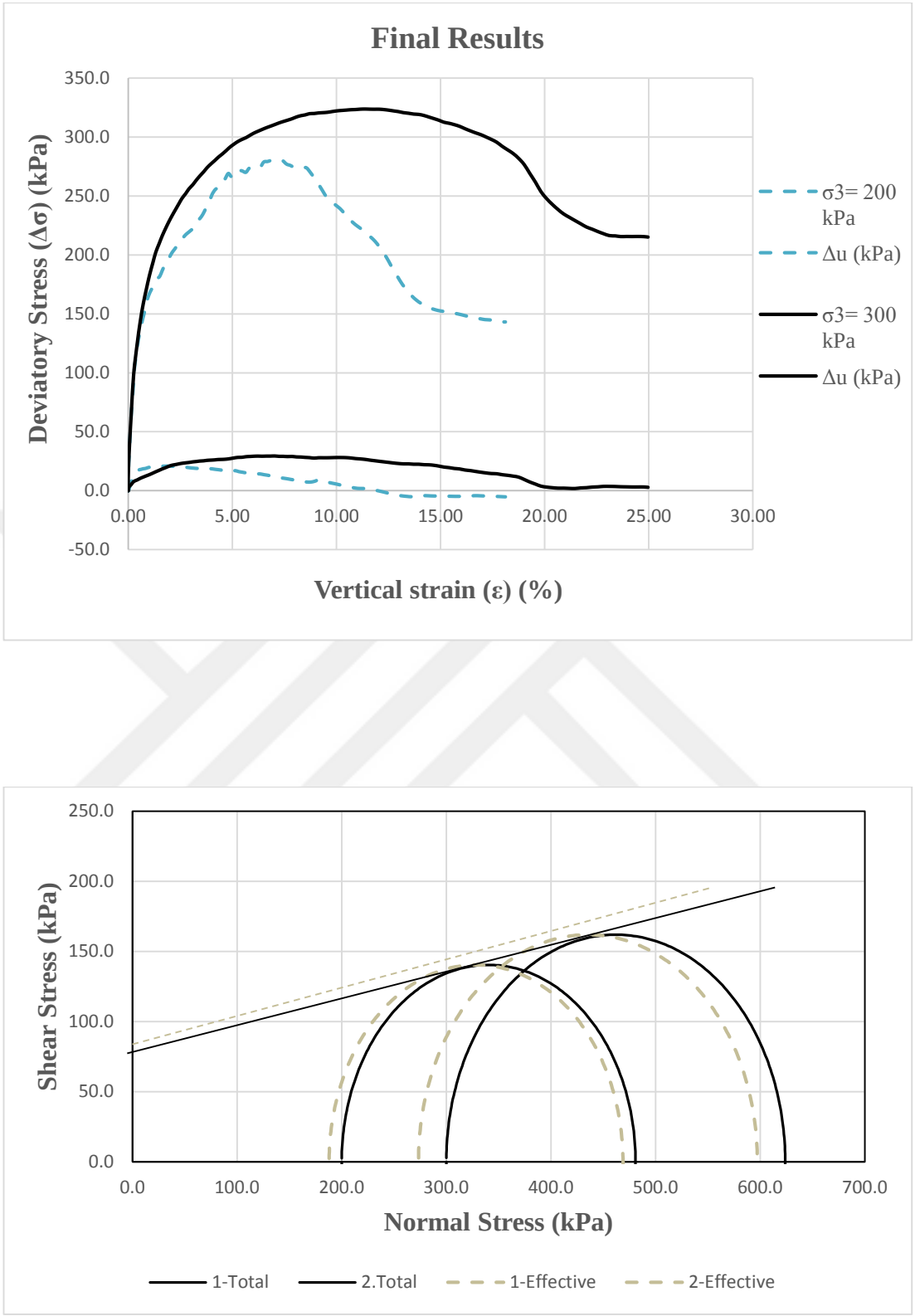


Figure 4.17 : Results Obtained from CU Test – 1.5% Nano-Clay - Clay Mixture.

Table 4.18 : Summarized Results Obtained From CU Test.

Material	Φ	c (kPa)	Φ’	c' (kPa)
Clay	13	35	19	20
Nano-Carbon Fiber (0.25%)	15	22	17	30
Nano-Clay (1.5%)	10	80	11	78
Nano-Silica Powder (0.5%)	11	50	21	21
CBR PLUS (0.75%)	10	58	18	36
Zycosil (5%)	14	32	18	42

From figures 4.12-4.17 and data from Table 4.18, it is obvious that although the amount of additives is so small, they can make huge effects on cohesion and internal friction angle.

4.8 Direct Shear Test

During the tests it was obsereved that the behavior of dried samples kept in water are very different in compare with the condition that they are compacted in their optimum water content. This interesting behavior was more obvious in Zycosil mixed and nano-Silica Powder mixed samples. In Figure 4.18, these differences are visible. Thus, direct shear tests were applied to examine the differences. For examination, at first, all samples were compacted in their optimum water content with compaction rate of 98-99% and then divided them in two groups. Group one was named in “wet condition” which direct shear test applied on them after they were compacted, and group two which named “dried condition” which direct shear test applied on them after they were compacted and dried in the oven for 24h in 50 degree centigrade.

This section contains data plots from direct shear test results values of strain (20 %) and 3 different normal stresses (200, 300, and 400 kPa), which has applied on wet and dried Clay, 5% of Zycosil-Rc, 10% of Zycosil-Rc, and 0.5% Nano-Silica-Rc, mixtures. Figures 4.19 - 4.26 shows how cohesion and internal angle friction of samples changes after being dried. Results are summarized in Table.4.19.

These data can be interpreted in three ways:

1- In wet condition 5% of Zycosil can dramatically increase the cohesion of clay, on the other hand 0.5% Nano-Silica Powder decrease the amount of cohesion but not have very strong effect like Zycosil.

2- In dry condition, 5% of Zycosil dramatically decrease the cohesion of clay and increase the internal friction angle.

3- Comparing each samples' wet and dry conditions, cohesion in highly increase after getting dried, except 5% of Zycosil-Clay mixtures, which the cohesion is intensely decreased.

Table 4.19 : Summarized Data Obtained from Direct Shear Test for Both Wet and Dried Conditions.

Material	Wet Condition		Dried Condition	
	Φ	c (kPa)	Φ'	c' (kPa)
Clay	16	21	15	75
Nano-Silica Powder (0.5%)	21	9	19	68
Zycosil (5%)	16	77	23	18
Zycosil (10%)	22	26	18	55

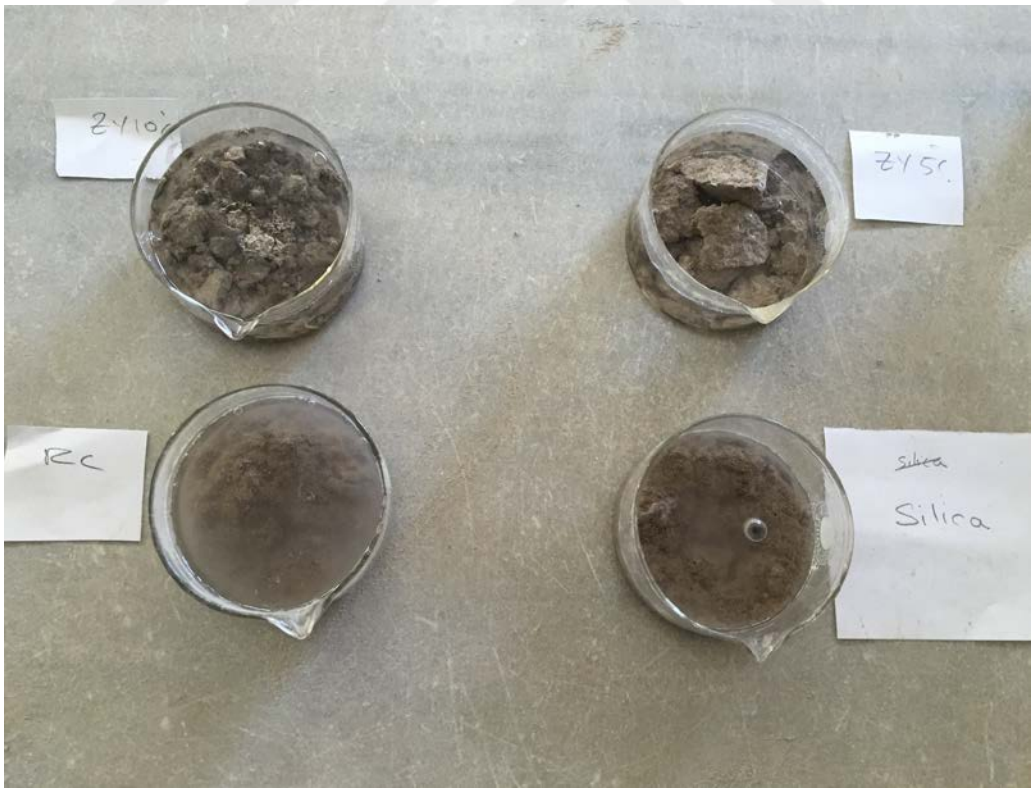


Figure 4.18 : Puting Samples in the water after getting dried.

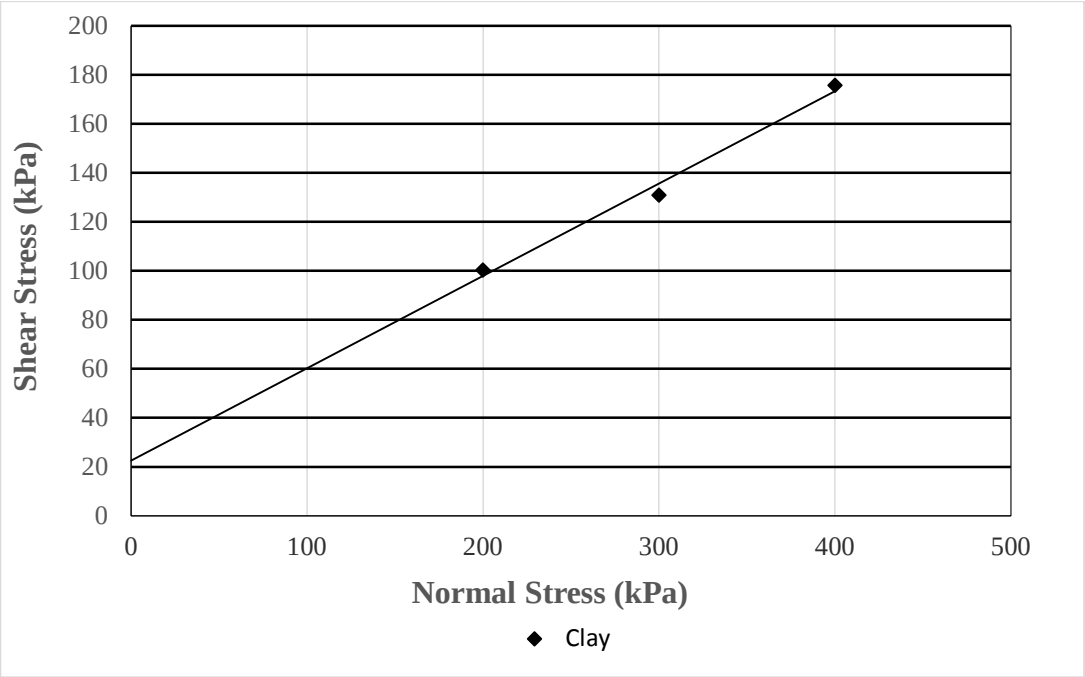


Figure 4.19 : Results of Direct Shear Test – Clay - Wet Condition.

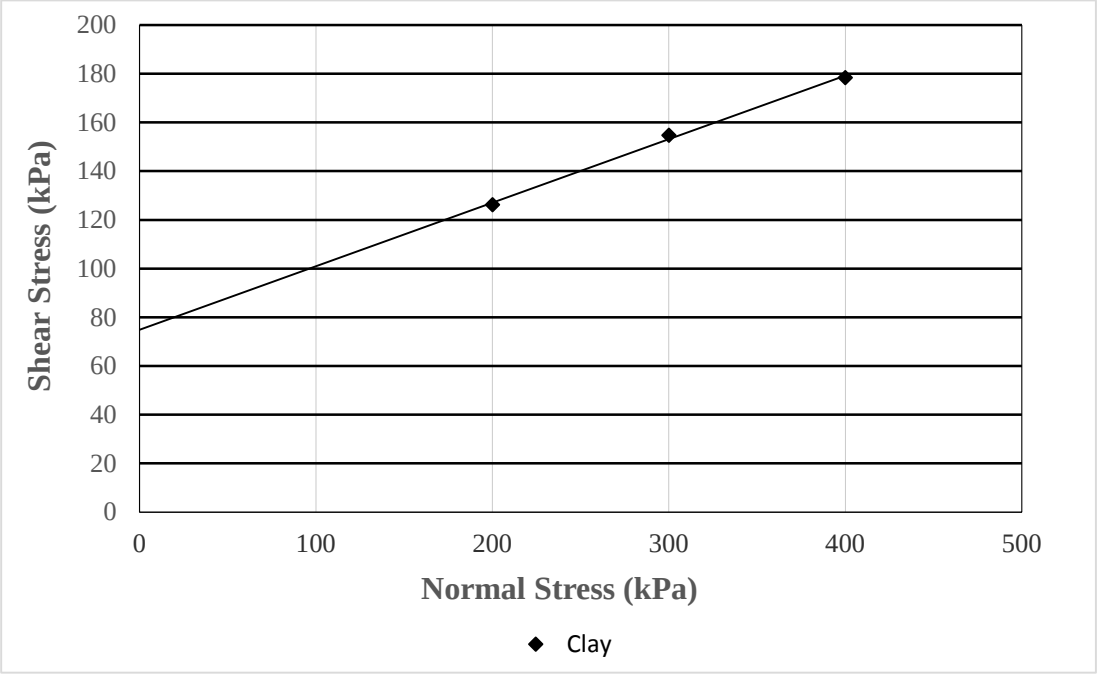


Figure 4.20 : Results of Direct Shear Test – Clay- Dried Condition.

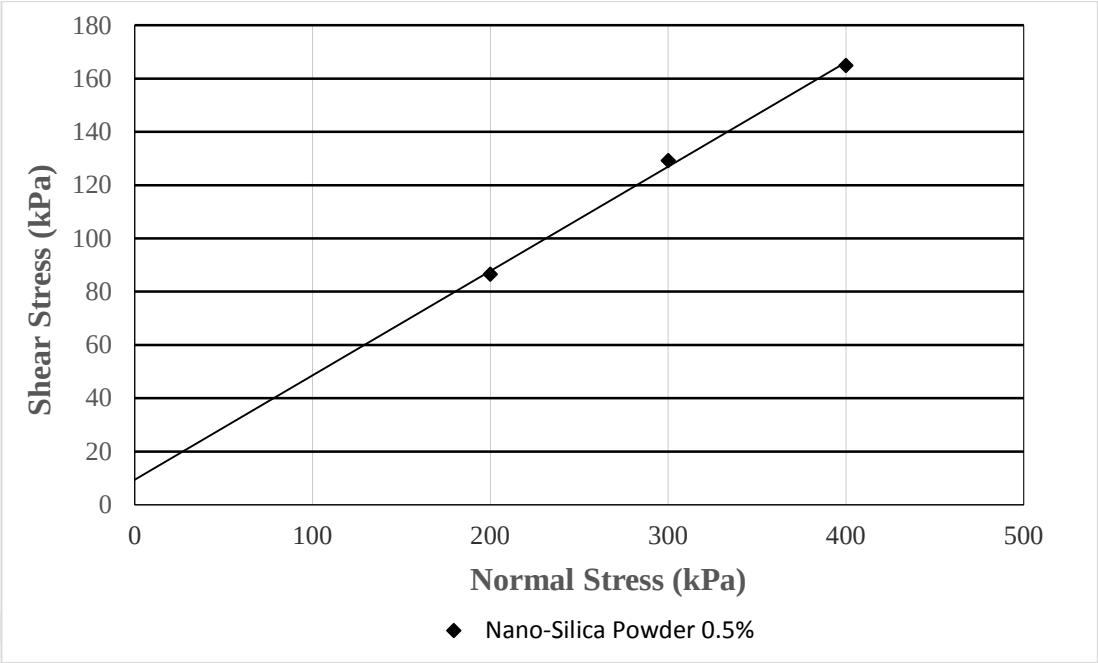


Figure 4.21 : Results of Direct Shear Test – 0.5% Nano-Silica Powder-Clay - Wet Condition.

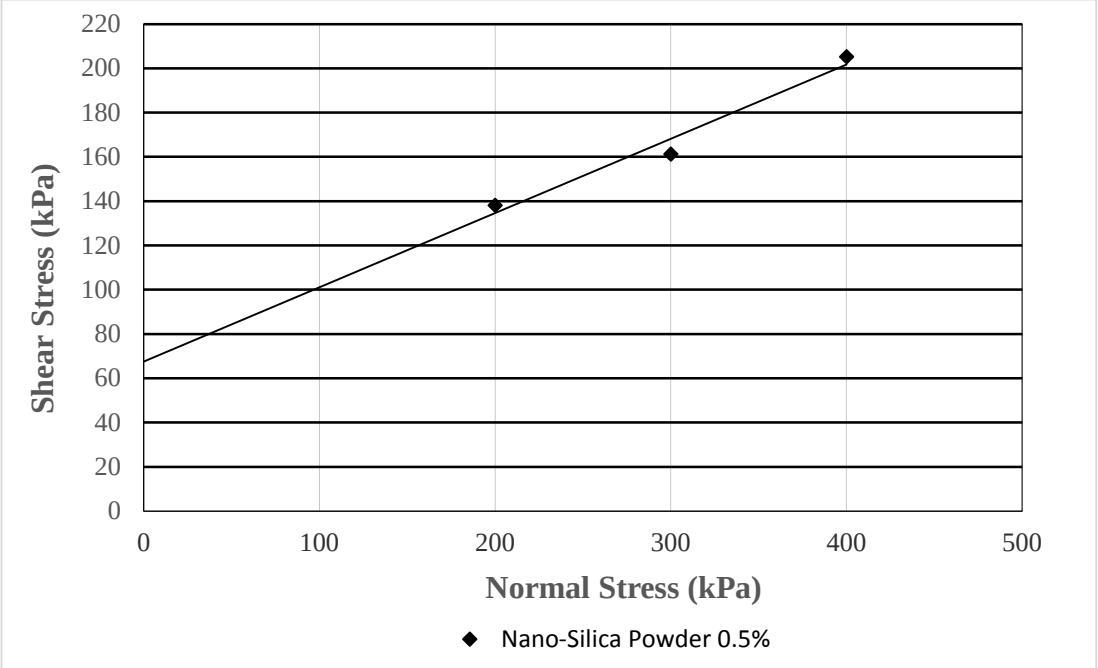


Figure 4.22 : Results of Direct Shear Test – 0.5% Nano-Silica Powder-Clay - Dried Condition.

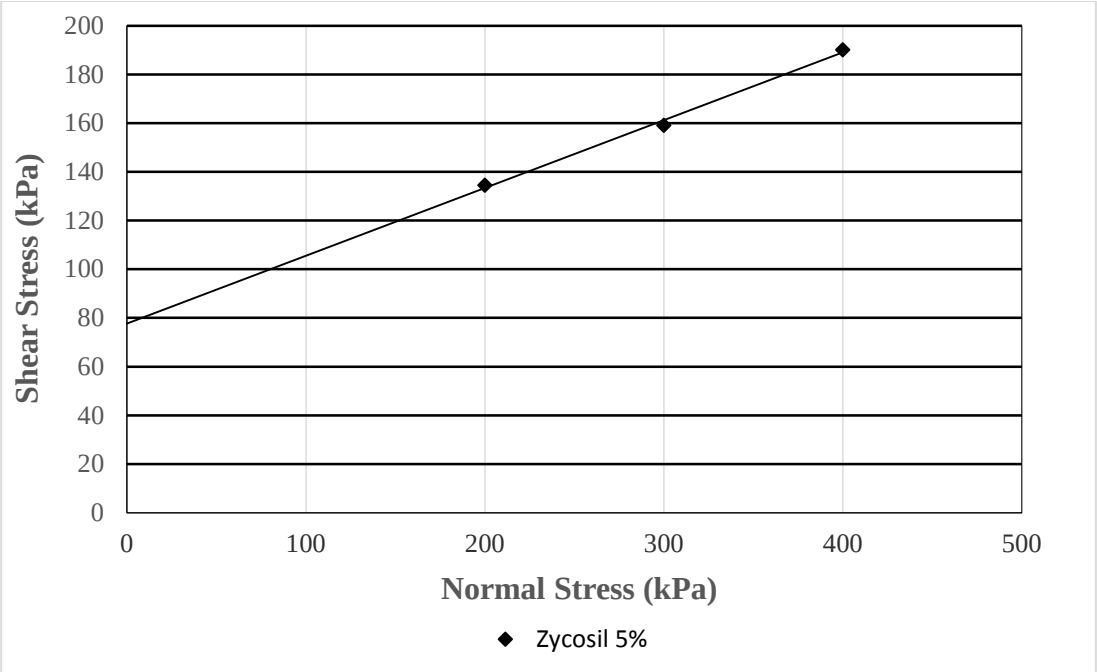


Figure 4.23 : Results of Direct Shear Test – 5% Zycosil-Clay - Wet Condition.

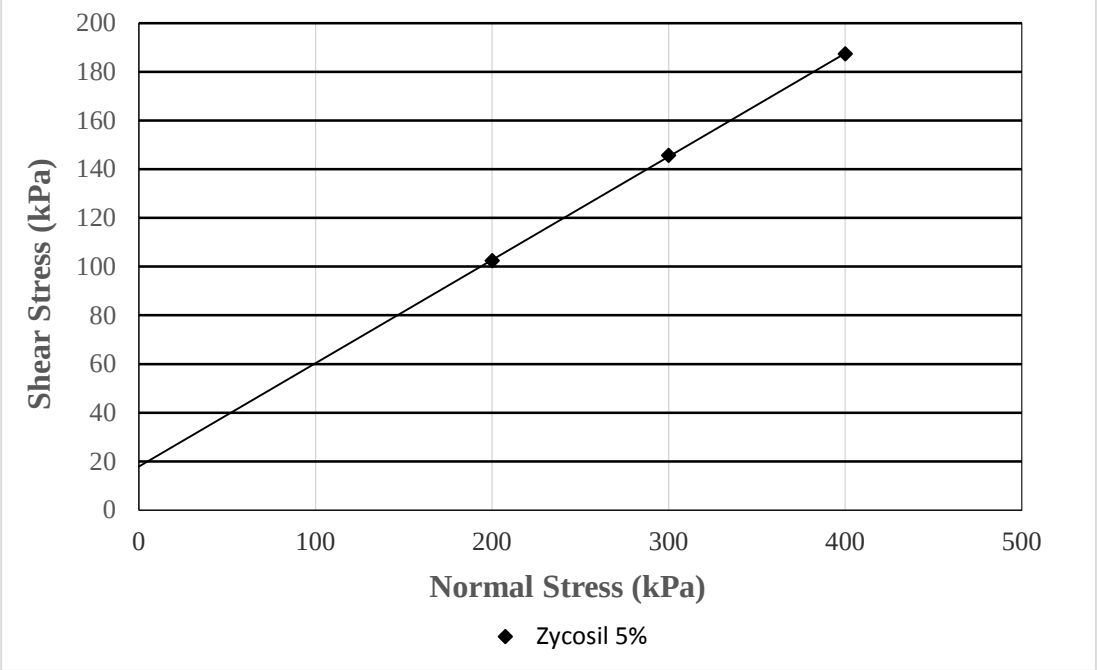


Figure 4.24 : Results of Direct Shear Test – 5% Zycosil-Clay - Dried Condition.

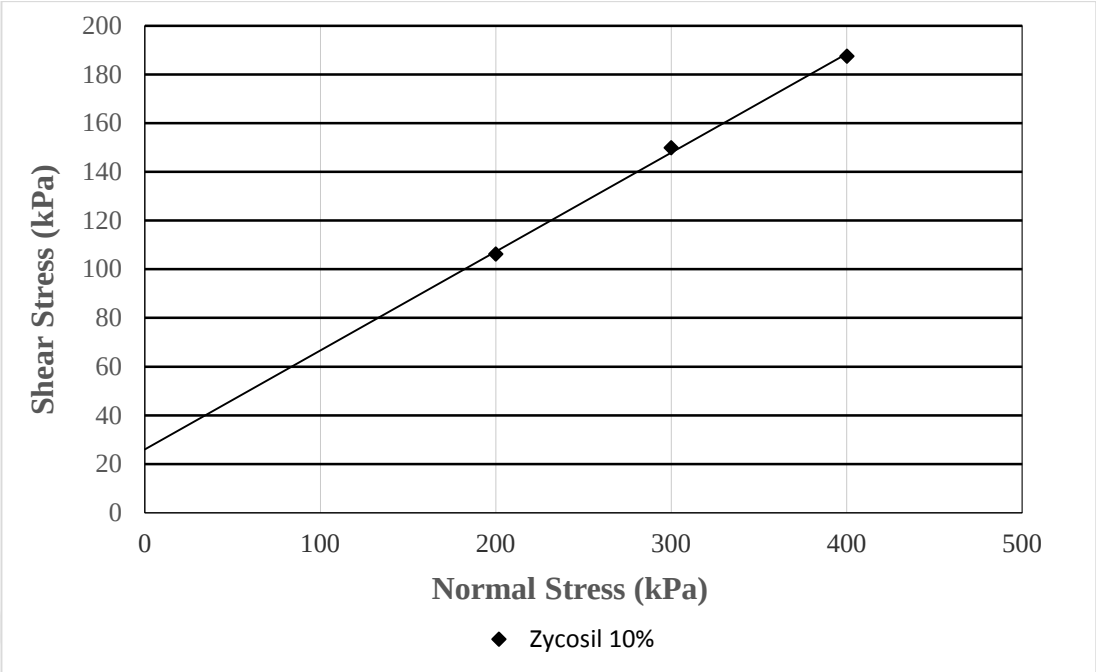


Figure 4.25 : Results of Direct Shear Test – 10% Zycosil-Clay - Wried Condition.

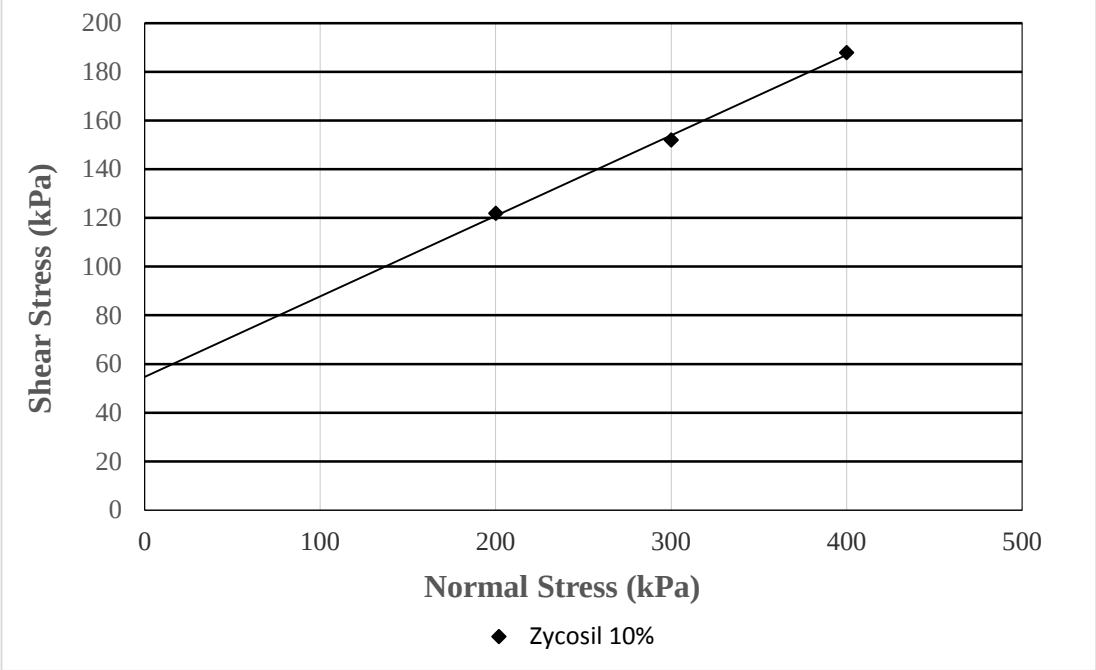


Figure 4.26 : Results of Direct Shear Test – 10% Zycosil-Clay - Dried Condition.

5. CONCLUSION

The purpose of this study was to find new materials for stabilization of clays, which can be useable in unpaved roads or road's subgrade. The clay used in this study was CH clay, which was collected from Ciftalankoy.

Lime and cement stabilization are traditional methods of clay stabilization but they are not eco-friendly. Thus, it has been tried to find some materials that are more eco-friendly, and small amount of them can have huge effect. Nano-Materials and Nano-Material based Polymers are the materials that could satisfy this goal. Therefore, with the help of other research that had been done before, five materials such as, Nano-Clay, Nano-Silica Powder, Nano-Carbon Fibers, CBR PLUS, and Zycosil had been chosen and several tests such as, Atterberg limits, unconfined compression test, Soaked CBR test, consolidation and permeability tests, and triaxial tests had been applied to investigate Nano-Materials and Nano-Polymers effects on clays.

It is also necessary to mention that during conducting tests, accidentally, it was noticed that the treated clays have different behavior after getting dried and placed in water. The different behavior such as resistance against dissolving in water in Zycosil-Clay mixture, or getting more loose in water in Nano-Silica-Clay mixture was noticed. Consequently, by conducting direct shear test, behavior of treated clays in wet and dried conditions were studied.

According to test results, the following outcomes can be summarized:

- Except Zycosil, which increase the liquid limit, plastic limit, and plastic index around 50%, 22%,and 86% respectively, addition of Nano-Materials does not have noticeable change on Atterberg limits of clay.
- In general, additives that used in this thesis did not have any special effects on unit weight and optimum water content of the clay.
- For unconfined compression test, additives mixed with clay at different percentages were kept for 1, 7, and 28 days as curing time. Additive-Mixed samples got 70-75% of their maximum strength on 7^{th} day, and at the end of

28th day, they had reached their maximum strength. The results showed that Zycosil 5% had the best effect among other materials, which increased the strength from 310kPa to 409kPa (about 32%) after 28th day of curing. Nano-Clay 1.5%, Nano-Carbon Fibers 0.25%, Nano-Silica Powder 0.5%, and CBR PLUS 0.75% had increased the strength around 25%, 14%, 10%, and 4%, respectively. However, it should be noted that the maximum unconfined compressive strength of samples decreased by increasing the percentage of additives.

- Swelling rate and CBR value were measured by conducting the soaked California Bearing Ratio (CBR) test. Normal Clay reached its maximum swelling rate (8%) in only 4 days, but other mixtures' rate fixed after 10 days. Therefore, swelling rate measured in two different days, 4th and 10th days; on 10th day, almost all mixtures reach the maximum rate (8%), but on the 4th day Nano-Carbon fibers 0.25% and CBR PLUS 1% – Clay mixture showed the best performance and their swelling rate was around 5.3% and 5.7%, respectively.
- Effect of additives could not be measured properly, because normal clay had already very low values, which were 0.2% and 0.8% at 2.5mm and 5mm of penetration, respectively. However, even under these conditions Zycosil 10% was the most effective additive that could increase the CBR value to 1.6% and 3% at 2.5mm and 5mm of penetration, which still consider as poor soil group.
- Additives did not have any considerable effect on consolidation of clay, but as we noticed in CBR test, Nano-Carbon fibers and CBR PLUS could decrease the swelling pressure from 100kPa to 72kPa and 75kPa, respectively. In addition, Zycosil could reduce it to 85kPa.
- Under 200kPa pressure, 5% of Zycosil had the best effect on hydraulic conductivity of clay, which was able to decrease it around 50% (1.20×10^{-10}m/s to 5.84×10^{-11}m/s). Nevertheless, 10% of Zycosil increased the hydraulic conductivity to 1.31×10^{-10}m/s, which means increase in Zycosil more than 5% can reverse the process. Regarding Nano-Materials, generally, they could decrease the permeability around 30-40%, which prove that their Nano-Scale size can fill the porous and prevent water

penetration). It should be mentioned that 0.75 % of CBR PLUS did not change soil's permeability.

- Under 400kPa pressure, 0.5% of Nano-Silica powder had better performance, and could decrease the hydraulic conductivity around 20% (from 8.48×10^{-11}m/s to 6.76×10^{-11}m/s). 1.5% of Nano-Clay could decrease the permeability around 10%, but 0.25% of Nano-Carbon fibers and 5% of Zycosil could not make any difference. However, under this pressure 10% of Zycosil increase the hydraulic conductivity by 250%. Moreover, 0.75% of CBR PLUS increased the permeability up to 35%.
- Results obtained from CU test improve that adding Nano-Materials and Nano-Material base Polymers can dramatically increase the cohesion, and decrease the internal friction of soil. These results are summarized in Table 4.18.
- By supplying direct shear test on dried and wet clay, it revealed that behavior of soil could change after getting dried and put in the water again. This procedure raises the cohesion of the clay, but does not change the internal friction angle significantly. However, results shows that additive-mixed clay are more sensitive and can demonstrate uneven performance that is more extensive in both cohesion and internal friction angle aspects which are summarized in Table 4.19.

During this work interesting results were gotten which means that only small amount of Nano-Materials ,with the help of their huge specific surface area, can affect the clay as well as Nano-Polymers. Although, the price of these materials is quite high, but with advances in the technology producing these materials will be much more easy and the final cost will be cheaper than now. This thesis is trying to give this insight to other researches that how Nano-Materials can affect clayey soils, for their furthere investigations.

It shoud be mentioned that this study only involved only some of the nano materilas, and there are many numbers of materials and mixtures that other researches can work on such as mixing Nano-Materials with lime, cement, etc., or using other types of nano and nano based materials.

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Zydex Industries Ltd. Zycosil Multi Surface Sealer Data Sheet.

APPENDICES

APPENDIX A: Results of direct shear test

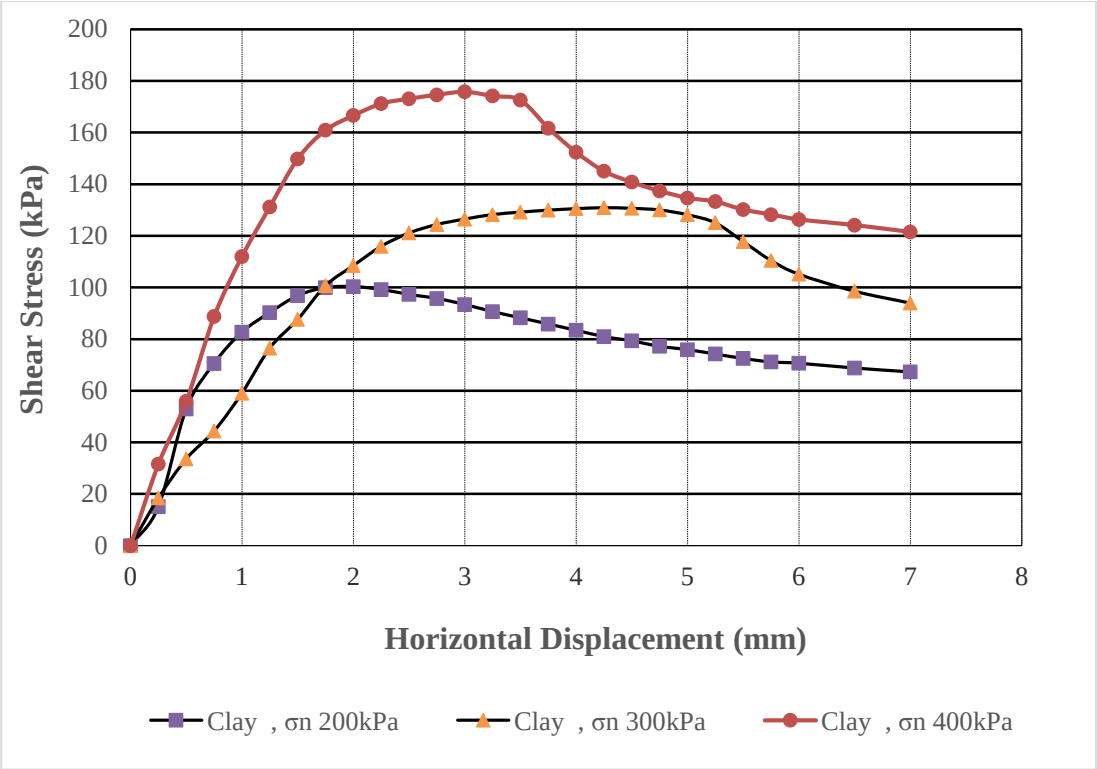

Figure A.1 : Normally compacted clay's shear stress-displacement curves.

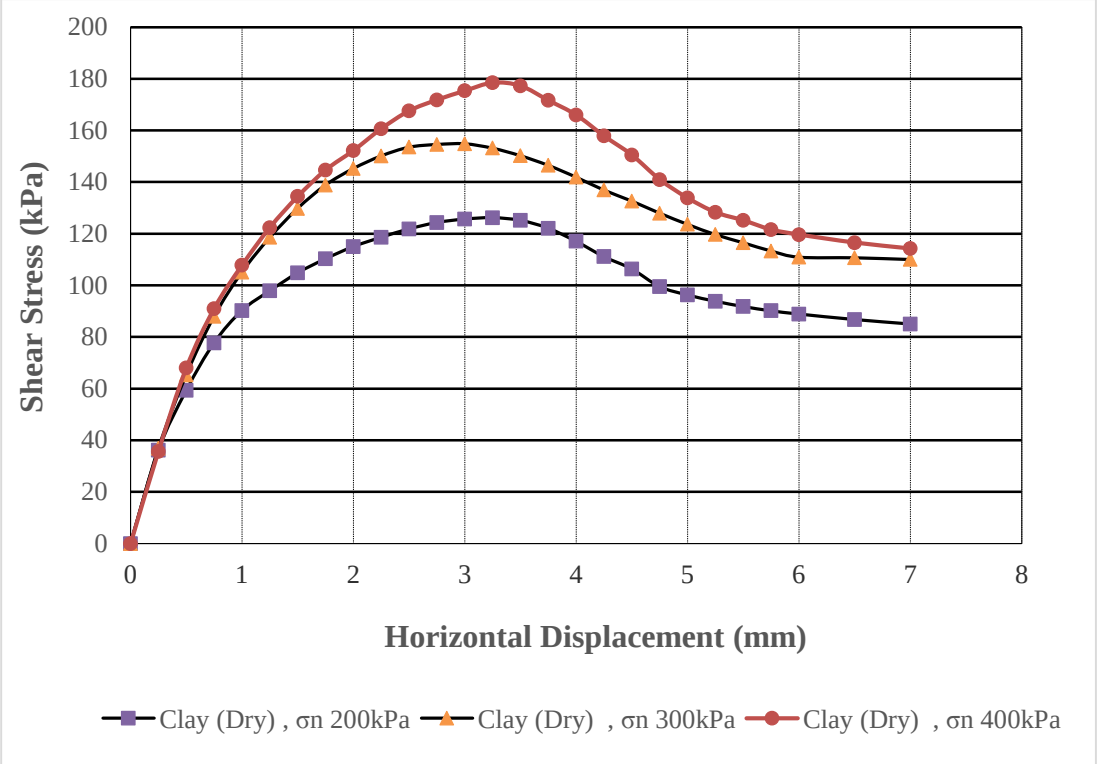

Figure A.2 : Normally compacted but dried clay's shear stress-displacement curves.

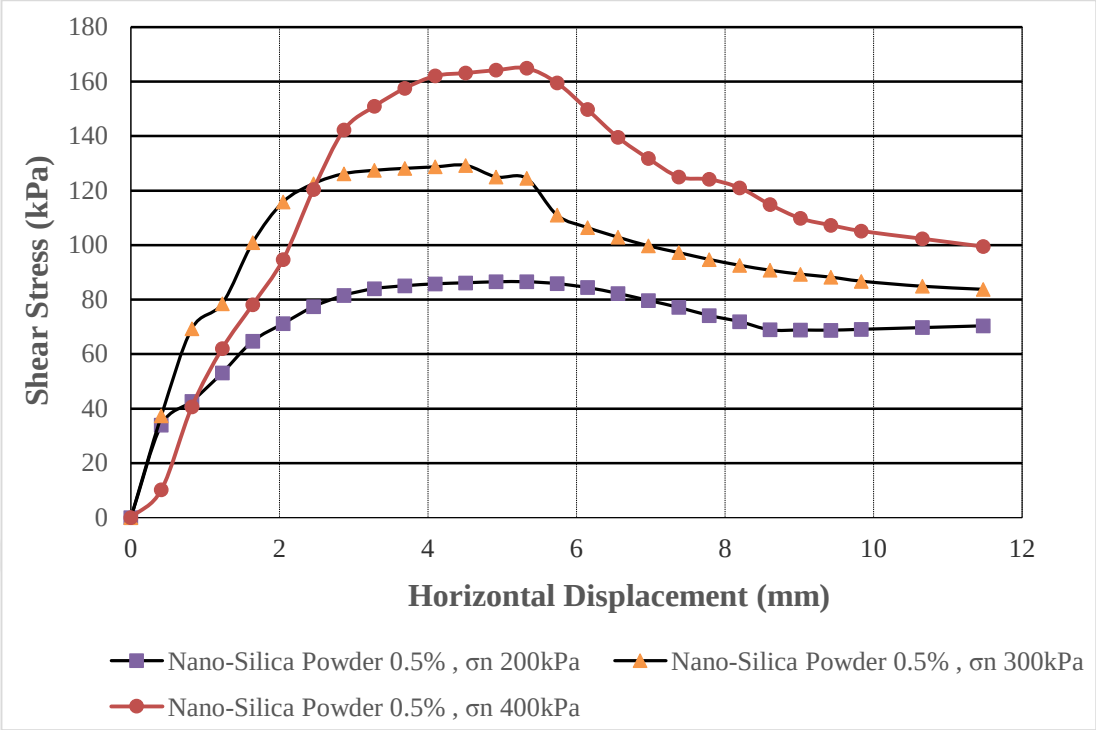

Figure A.3 : Normally compacted 0.5% Nano-Silica Powder-Clay shear stress-displacement curves.

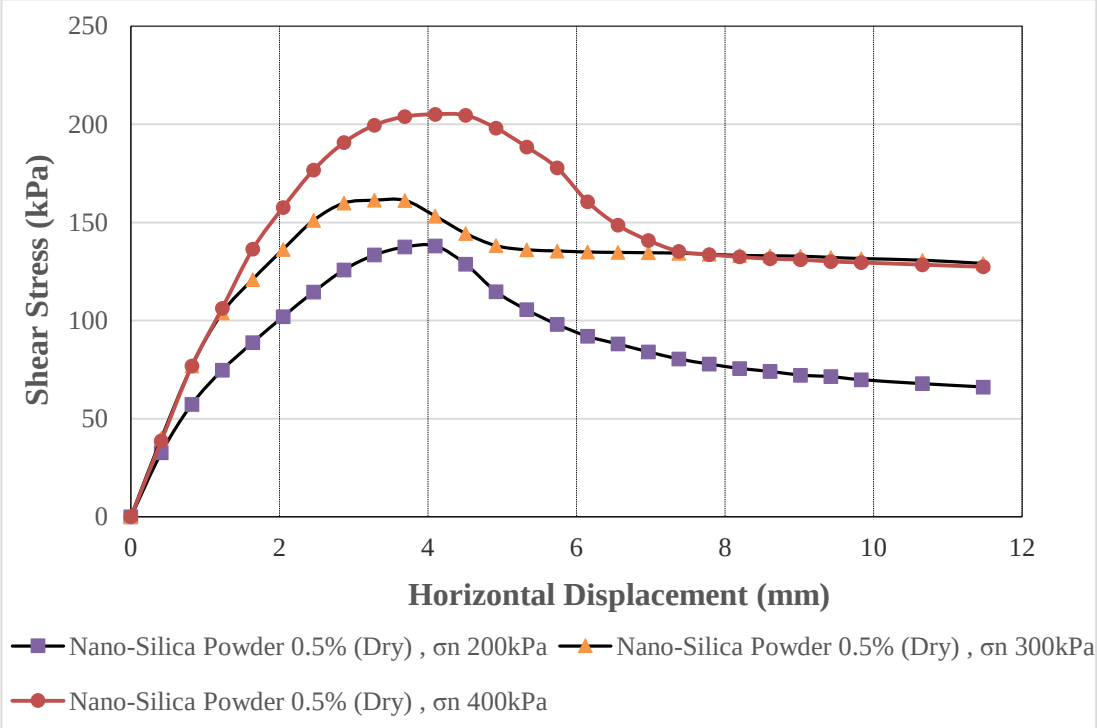

Figure A.4 : Normally compacted but dried 0.5% Nano-Silica Powder-Clay shear stress-displacement curves.

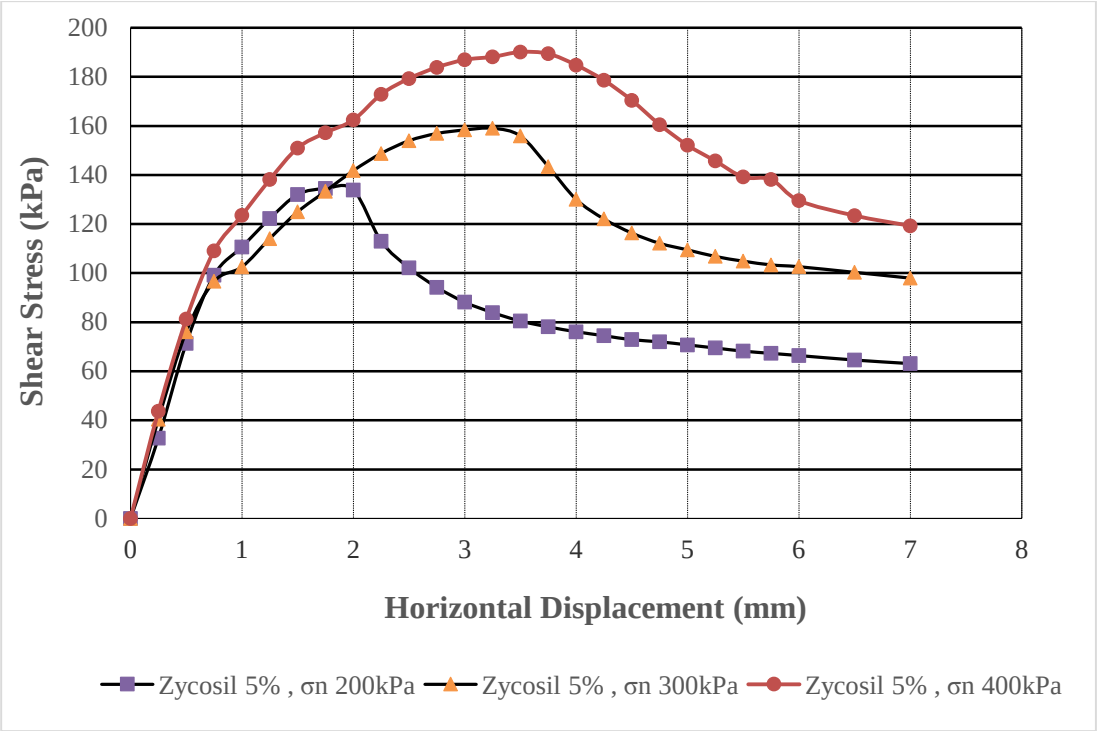

Figure A.5 : Normally compacted 5% Zycosil-Clay shear stress-displacement curves.

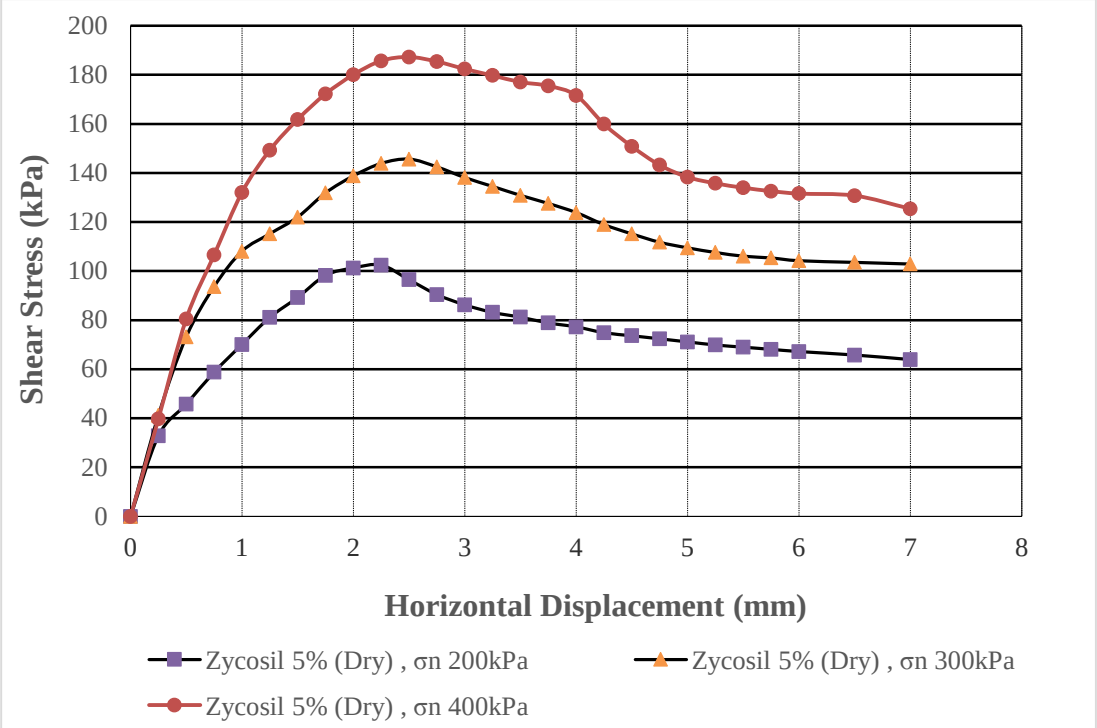

Figure A.6 : Normally compacted but dried 5% Zycosil-Clay shear stress-displacement curves.

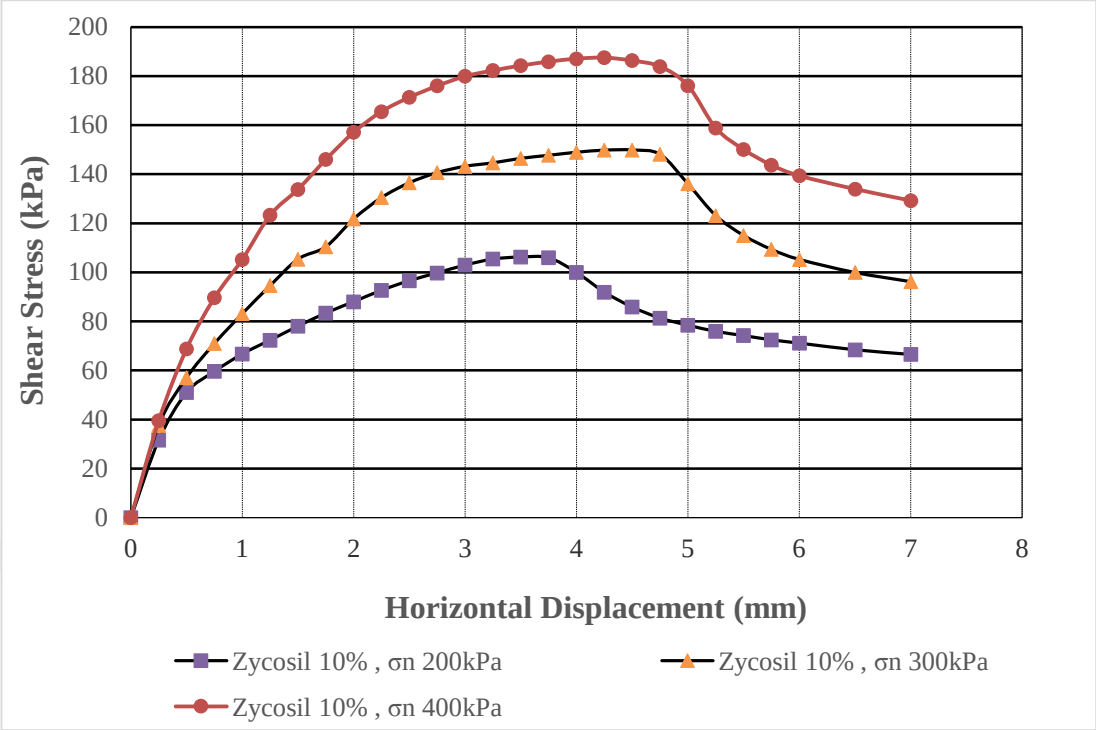

Figure A.7 : Normally compacted 10% Zycosil-Clay shear stress-displacement curves.

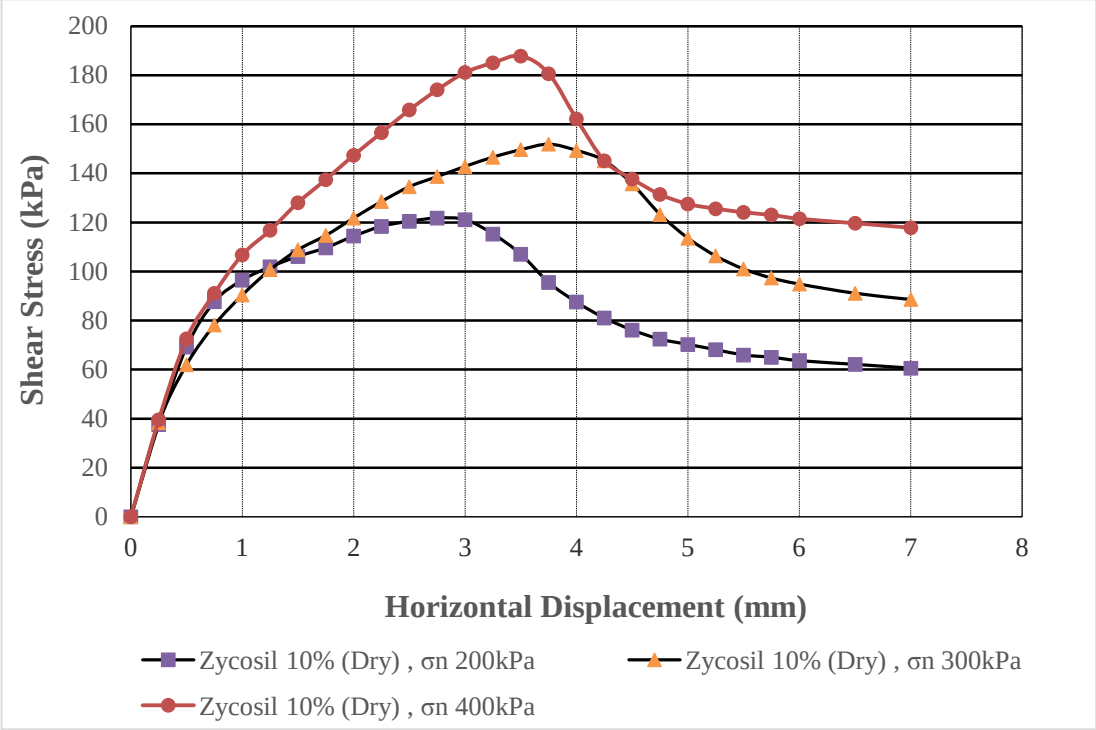

Figure A.8 : Normally compacted but dried 10% Zycosil-Clay shear stress-displacement curves.

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