MAY 2019 Ph.D. in Civil Engineering AKAR BAKHTIYAR ABDULLAH

HASAN KALYONCU UNIVERSITY
GRADUATE SCHOOL OF
NATURAL AND APPLIED SCIENCES

INVESTIGATING GEOTECHNICAL PROPERTIES OF SOILS USING GROUND GRANULATED BLAST FURNACE SLAG, FLY ASH, AND LIGNOSULFONATES

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AKAR BAKHTIYAR ABDULLAH

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Investigating geotechnical properties of soils using Ground Granulated Blast Furnace Slag, Fly Ash, and Lignosulfonates

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Prof. Dr. Mehmet KARPUZCU

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

Akar Bakhtiyar ABDULLAH

ABSTRACT

INVESTIGATING GEOTECHNICAL PROPERTIES OF SOILS USING GROUND GRANULATED BLAST FURNACE SLAG, FLY ASH, AND LIGNOSULFONATES

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Ph.D. in Civil Engineering
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A number of ways exist to measure the level of soil improvement after administering a ground improvement application, but stabilisation is the one most frequently used. The term “soil stabilisation” describes the changing of a natural soil for construction or engineering purposes. Stabilisation will often have the purpose of making soils stronger by incorporating waste materials. Traditional chemical stabilisers, though, may not find ready acceptance in engineering construction because quicklime and cement are not renewable, their production involves consumption of large amounts of energy, they release considerable volumes of greenhouse gases and the brittleness of the resulting stabilised soils can affect structures’ stability. This study presents the effect of using unary, binary and ternary blends of Ground Granulated Blast Furnace Slag (S), Fly Ash (FA) and Lignosulfonate (LS) as the stabilizer to investigate soil properties. For this, two soil mixture groups were designed depending on soil types with changing binder contents. disposal materials were used in partial substitution of soil1 and soils2 at 0%, 6%, 8%, and 10% by weight. The results show that among different way of replacements, soils containing 8% of disposal materials exhibited the best results of California Bearing Ratio (CBR), Unconfined Compressive Strength (UCS), atterberg limits, and permeability coefficient at 28 days. The stabilizer soils containing ternary binder materials (S, FA, and LS) gave better results than binary then unary replacements.

Keywords: Ground Granulated Blast Furnace Slag, Fly ash, Lignosulfonate, California Bearing Ratio, Unconfined Compressive Strength, Atterberg limits, Permeability

ÖZET

GRANÜLE YÜKSEK FIRIN CÜRUFU, UÇUCU KÜL VE LIGNOSÜLFONATLAR KULLANARAK TOPRAKLARIN JEOTEKNIK ÖZELLIKLERININ ARAŞTIRILMASI

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Bir zemin iyileştirme uygulaması yapıldıktan sonra zemin iyileştirme seviyesini ölçmek için bir takım yollar vardır, ancak stabilizasyon en sık kullanılandır. “Toprak stabilizasyonu” terimi, inşaat veya mühendislik amaçlı doğal bir toprağın değişimini tanımlar. Stabilizasyon, çoğu zaman atık malzemeleri dahil ederek toprakları daha güçlü hale getirme amacına sahip olacaktır. Geleneksel kimyasal stabilizatörler, hızlı kireç ve çimento yenilenebilir olmadığından, üretimleri büyük miktarda enerji tüketimini gerektirdiğinden, önemli miktarda sera gazı salgıladığından ve elde edilen stabilize edilmiş toprakların kırılganlığının yapıları etkileyebileceğinden, mühendislik yapımında hazır kabul görememektedir ' istikrar. Bu çalışma, toprak özelliklerini araştırmak için Öğütülmüş Granül Yüksek Fırın Cürufu (S), Uçucu kül (FA) ve lignosülfonatın (LS) birleşik, ikili ve üçlü karışımlarının kullanılmasının etkisini göstermektedir. Bunun için değişen bağlayıcı içerikli toprak tiplerine bağlı olarak iki karışım grubu tasarlanmıştır. Katki maddeler, toprağın 1 ve 2 ye degişik oranlarda % 0,% 6,% 8 ve% 10'da kullanılmıştır. Sonuçlar, farklı değişim şekilleri arasında, katkı maddelerin % 8'ini içeren toprakların en iyi Kaliforniya Dayanım testi, Serbest basınç dayanımı, atterberg sınırları ve 28 gün geçirgenlik katsayısı sonuçlarını gösterdiğini göstermektedir. Üçlü bağlayıcı malzemeleri içeren dengeleyici topraklar, ikili ve birleşik değiştirmelerden daha iyi sonuçlar verdi.

Anahtar Kelimeler: Öğütülmüş Fırın Cürufu, uçucu kül, Kaliforniya Taşıma Oranı, Kapalı basınç direnci, Atterberg sınırları, geçirgenlik. .

To all members of my sweet family

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LIST OF SYMBOLS/ ABREVIATIONS

S	Ground Granulated Blast Furnace Slag
FA	Fly ash
LS	Lignosulfonate
CBR	California Bearing Ratio
UCS	Unconfined Compressive Strength
BTS	Brazilian tensile strength
Ep	Resilient Modulus
DCP	Dynamic Cone Penetrometer
Vs	Shear-Wave Velocity
G_0	Small-Strain Shear Modulus
SEM	Scanning Electron Microscope
C	Carbon
O	Oxygen
Na	Sodium
PI	Plasticity Index
PL	Plastic Limits
LL	Liquid Limit
wc/c	Clay/Cement ratio
C/S	Ratio of cement-to-soil
CCR	Calcium Carbide Residue
GBFS	Granulated Blast Furnace Slag
BOFS	Basic Oxygen Furnace Slag
LFS	Ladle Furnace Slag

CSEB	Compressed Stabilized Earth Blocks
USC	Unified Soil Classification
ASTM	American Society for Testing and Materials
BET	Brunauer-Emmett-Teller
womc	Optimum Moisture Content
rdm	Maximum Dry Density
k	Permeability Coefficients
LCC	Lime Cement Column
ACI	American Concrete Institute
QC	Quality Control
QA	Quality Assurance
CDF	Confined Disposal Facilities
msa	million standard axles
IRC	Indian Roads Congress
ASTM	American Society for Testing and Materials

CHAPTER 1

INTRODUCTION

1.1 General

Problems associated with the settlement and bearing capacity increased when the Geotechnical projects have to be built on the low shear strength and soft weak soils. Stabilization of soils is an economic, lasting method and a simple way to prevent above-mentioned issues which dispersive soils replaced with suitable materials (Ouhadi and Goodarzi, 2006). The stabilization process denotes to the treatment, including chemical reactions that improve the shear strength, controlling shrinking and swelling, reducing permeability, and increasing bearing capacity of soil (EPA, 2000; Cuisinier et al., 2011; Arulrajah et al., 2012). For decades, many verified materials like Portland cement and lime have been used to attain acceptable surface quality of construction projects, even though they are presently reflected to environmental problems and high economic costs (Horpibulsuk et al., 2010; Consoli et al., 2007). Nevertheless, these traditional chemical additives are not continuously accepted in civil works (Harichane et al., 2012; Abdi, 2011). Usually, when lime or cement stabilized, an alkaline attack on steel structures and concrete close to the stabilized soils imposes due to significantly increasing of the pH value of soils (da Rocha et al., 2014; Chen et al., 2014; Indraratna et al., 2008). To overcome these problems, economical compound materials can be used for the improvement of soils that eligible of ionic exchanges, flocculation of clay and activating light pozzolanic activity (Manso et al., 2013). Utilization of the byproduct materials has been intensely encouraged for the soil stabilization as the desires of sustainable development (Zhang et al., 2017). The most renowned are Ground Granulated Blast Furnace Slag (S), Fly Ash (FA) and lignosulfonate (LS) (Makikyro and Miikikyrii, 2001; Kolias et al., 2005; Zhang et al., 2015; Cai et al., 2016).

Ground Granulated Blast Furnace Slag resulting from manufacture of iron in the furnace, is a granular, non-crystalline, and fine after being rapidly cooled down by water and then ground (Pal et al., 2003; Jin et al., 2015). Ground Granulated Blast

Furnace Slag is formed by the combination of iron ore with limestone flux, which mainly contains of manganese, aluminum, calcium, magnesium, and silicates in different constituents (Zhang et al., 2011). On the other hand, Fly ash play an important role in improving the properties of soils due to favorable properties such as pozzolanic properties, low unit weight, low compressibility, high shear strength, and insensitive to the moisture variations (Dahale et al., 2017). Fly asf has chemical composition like high percentage of silica (60–65%), alumina (25–30%), magnetite, and Fe_2O_3 (6–15%) enables for the synthesis of zeolite, alum, and precipitated silica (Querol et al., 2002; Iyer and Scott, 2001). In contrast with the other traditional chemical admixtures, LS is non-toxic by-product that displayed favorable aspects for stabilizing both non-cohesive and cohesive soils (Zhang et al., 2015; Cai et al., 2016; Indraratna et al., 2008).

The use of traditional admixtures in stabilization of different soil types is well familiar with the literature, nevertheless, stabilization via by-product materials increasingly used recently (Chen et al., 2015; Dahale et al., 2017; Nath, 2018; Al-Malack et al., 2016). Wild et al. (Wild et al., 1999) improved swelling of clay using ground granulated blast furnace slag incorporated with an amount of lime. Obuzor et al. (Obuzor et al., 2011) endorsed the favorable effects of GBFS on the durability, strength and the behavior of volume change of the clay. Kumar and Sharma (Phani Kumar and Sharma, 2004) enhanced swelling characteristics, soil plasticity, permeability and shear strength of the stabilized soil through fly asf replacement. Cokca (Cokca, 2001) decreased potential swelling and plasticity when they investigated stabilization of soil using different volumes of fly asf type C. Vinod et al. (Vinod et al., 2010) and Cai et al. (Cai et al., 2016) treated stability of soil structure by coating and bonding soil particles using lignin based cementation materials. Alazigha et al. (Alazigha et al., 2016) improved swelling and durability of the expansive clay stabilization by optimizing LS content.

1.2 Research Significance

Although there have been several researches on the characteristics and properties of stabilized soils using byproduct materials, these were mostly unary (individually) used. Studies to investigate the performances of binary and ternary stabilized soils have been noticed to be quite limited. Based on these, an effort has been made to establish the viability of using by-product stabilized soils, accordingly; the main objective of the present study is to address the potential use and effectiveness utilizing unary, binary, and ternary effects of various dosages of S, FA, and LS for the stabilization of two types of soils. Un stabilized soils were selected as a control stabilizer for comparison purpose. The assessment will be entirely based on analyzing and assessing developments in the stabilized soil types and determining the engineering properties such as chemical composition of the materials, CBR, UCS, Atterberg limits tests, and permeability. Thus, using ternary recycled materials for soil stabilization is unique and will open new doors for entering different shapes of byproduct materials in Geotechnical engineering.

1.3 Outline of the Thesis

In the present Ph.D. thesis, the total work is demonstrated by five chapters;

Chapter 1 is a general introduction and the significance of the thesis

Chapter 2 is the literature reviews on the compounds as well as other previous studies

Chapter 3 this chapter covers highlights of the experimental test. The properties of all materials used, mix design, mixing techniques, the preparation of test samples, and testing procedures are stated.

Chapter 4 contains the experimental results and discussion of the CBR, UCS, Atterberg limits tests, and permeability. In addition to stabilization effects on environmental and economy.

Chapter 5. is a key to the main conclusions from the results of this study.

CHAPTER 2

LITERATURE REVIEW AND BACKGROUND

Before beginning a geotechnical project, a feasibility study should be carried out on the site. Before beginning the design process, it is customary to survey the site to gain knowledge of the subsoil's characteristics and the survey should take into account:

- The intended structure's function and design load.
- The type of foundation that should be put in place.
- The subsoil's bearing capacity.

Traditionally, it was the bearing capacity that influenced decisions on site selection. If it turned out not to be good, options were:

- To abandon the site.
- To replace the existing soil.
- To change the design in order to meet the site's limitations.

There was a considerable increase in the number of sites being abandoned because the bearing capacity of the soil was inadequate. The result was that usable land became more scarce and the demand for natural resources increased. Sites considered prone to liquefaction as well as sites covered by organic soils and soft clay were particularly affected, as were contaminated land and land that had suffered a landslide. The fact is, though, that for the majority of geotechnical projects finding a construction site that, unmodified, meets design requirements is unlikely and so the current practice has evolved of modifying problem soils' engineering properties and it is now possible to raise the performance of organic soils, soft clays and other problem soils to the level required by civil engineering standards.

2.1. Soil stabilisation

Before engineers can begin the process of treating the soil, they need to know the quantity of stabilisers that will be needed and tests are carried out in the laboratory to show how much mineral admixture will be required if stabilisation is to be perfect. Among its other benefits, stabilisation mitigates mining waste contamination, combining at the same time an increase in bearing capacity and a reduction in compressibility. Chemical stabilisation is often used and has been shown to work well in improving the engineering properties of several soil types. Fly ash, slag, gypsum, alum, lime and cement are all in customary use and problem soils' stiffness, compressibility and strength have all shown improvement under this treatment (Du et al., 2016; Horpibulsk et al., 2011; Puppala et al., 2004; Lo and Wardani, 2002).

The purpose of soil stabilisation is to improve the strength of the soil and increase its resistance to water softening by bonding soil particles, waterproofing them, or both (Sherwood, 1993). The available technology usually makes it possible to solve practical problems with a structural solution. When water is drained from wet soil, the soil becomes stronger and drainage and compaction are the simplest ways of stabilising soil. The main alternative is improving particle size gradation and binders added to a weak soil can bring further improvement. The methods so far described can all be allocated into either the mechanical or the chemical stabilisation category.

2.1.1. Mechanical stabilisation

Mechanical stabilisation involves the physical alteration of soil particles' nature and usual methods can include compaction or induced vibration or the incorporation of nailing and barriers. However, this review is not concerned with mechanical stabilisation and these methods will not be discussed in detail.

2.1.2. Chemical stabilisation

Chemical stabilisation works by a chemical reaction between the cementitious material that is the stabilising material and pozzolanic in the soil. This review is primarily concerned with chemical stabilisation and from this point on soil stabilisation will be taken to mean chemical stabilisation. The process involves the stabilisation of unbound materials through the addition of cementitious materials which may include bitumen, fly ash, lime and cement alone or in combination. When stabilised, the soil will have reduced compressibility, reduced permeability and increased strength (Keller, 2011). Stabilisation can take place in situ or off-site. It should be noted, though, that stabilisation is not some magical process capable of improving every property of every soil (Ingles and Metcalf, 1972) and the decision on the actual stabilisation process will be governed by the properties that the engineers most wish to improve. Chief among these are likely to be strength, stability, compressibility, permeability and durability (Ingles and Metcalf, 1972; Sherwood, 1993; Stab, 2002).

2.2. Components of stabilisation

Binder materials are used as stabilising agents to improve the geotechnical properties of weak soils. The properties in question include strength, compressibility, permeability and durability and the technology's components comprise: soils and the minerals in soils; and cementitious materials including binders and stabilising agents.

2.2.1. Soils

The most frequent stabilisation sites are those with soft soils including silt, organic soils and clayey peat where tests have shown a need to improve the engineering properties. Sherwood (Sherwood, 1993) describes fine grained granular materials as easiest to stabilise because their surface area is large in relation to their particle diameter. Clay soils contain particles with long, flat shapes which provide a large surface area, while silt can have significant sensitivity to small changes in the moisture level and can be difficult to stabilise (Sherwood, 1993). Peat and organic soils combine a water content that can be as high as 2000% with high porosity and organic content. A peat soil can have a consistency anywhere between muddy and fibrous and peat deposits are usually not deep but in the worst case they can extend several metres below the surface (Pousette et al., 1999; Cortellazzo and Cola, 1999; Åhnberg and Holm, 2017). The high exchange capacity of organic soils can impede hydration by not releasing calcium ions freed as part of the process of hydrating calcium aluminate and calcium silicate in the cement and in such cases stabilisation will rely on choosing the correct binder and the correct amount to add (Hebib and Farrell, 1999; Lahtinen et al., 1999; Åhnberg and Holm, 2017)

2.2.2. Stabilising agents

Stabilising agents may be primary hydraulic binders or secondary non-hydraulic binders; in either case, when water or pozzolanic materials are present they form cementitious materials by reaction with water. The commonly used binders are:

- cement
- lime
- fly ash
- blast furnace slag
- Lignin

2.2.2.1. Cement

Calcination is the name given to the process of heating limestone (calcium carbonate) to 1450 °C in a kiln with clay or another material with the aim of producing cement. Calcination liberates a carbon dioxide molecule from the calcium carbonate to produce calcium oxide (quicklime). A chemical combination between the calcium oxide and other materials in the mix forms clinker, a hard material containing calcium silicates and other cementitious compounds. Grinding the clinker to powder with gypsum forms the commonest kind of cement which is Ordinarily Portland Cement (OPC).

In 2010, worldwide production of hydraulic cement amounted to 3,300 million tons, more than half which was produced by the three largest producers: China (1800 million tons), (India (220 million tons) and the USA (63.5 million tons) (Survey, 8 October 2011). The division of the global capacity showed a similar ratio (Edwards, Epsom, UK, 2010). 2011 and 2012 saw continuing increases in consumption reaching 3585 Mt in 2011 and 3736 Mt in 2012, although the 8.3% growth rate in 2011 was slower. It fell again in 2012 to 4.2%.

Those are global growth rates; the Chinese experience between 2010 and 2012 differed substantially from that in Europe and North America as a result of the sovereign debt crisis that caused a recession in the region with consumption falling in 2010 by 1.9% to 445 Mt. Although it came back to 4.9% in 2011, 2012 saw another fall, this time by 1.1%. Demand in other parts of the world held strong in 2010, aided by consumption in Asian, African and Latin American countries and the 2010 figure of 1020 Mt more than offset European and North American reductions. Globally, annual consumption grew by 7.4% in 2010, by 5.1% in 2011 and by 4.3% in 2012. The end of 2012 saw worldwide production facilities including grinding and integrated facilities standing at 5,673, of which 3,900 were in China with 1,773 elsewhere in the world. There was in 2012 a global capacity for cement production amounting to 5,245 Mt with China responsible for 2,950 Mt and the rest of the world 2,295 Mt (Hargreaves, March 2013).

Cement was the first binding agent used in the soil stabilisation technology that took shape in the 1960s and, because it stabilises the soil on its own, is a primary stabiliser (Sherwood, 1993; Stab, 2002). When cement is used on its own, the reaction is not dependent of minerals being present in the soil and as it acts by reaction with water it can be used in any soil (Stab, 2002) which explains its use in stabilising a wide variety of soils. Cement types available on the market include OPC, blast furnace cement, sulphate resistant cement and cement high in alumina and which is used is usually decided by the kind of soil being stabilised and the target final strength. Hydration is at the heart of the cement reaction. It starts at the moment that cement and water are mixed, together with whatever materials are needed to reach the target hardness. It hardens and sets, enclosing soil as if it were a glue but without changing the structure of the soil (Stab, 2002). The hydration reaction is slow and begins at the surface of the cement grains – the grains may never be hydrated at their centre (Sherwood, 1993). A complex reaction involving a number of chemical reactions that have not yet been isolated (MacLaren and White, 2003) it is nevertheless known to be affected by:

- Foreign material and impurities in the soil
- The water to cement ratio
- The temperature at which the cement is cured
- The presence of any additives
- The specific surface of the mixture.

Taken all together, these factors have varying impacts on the strength of the stabilised soil and need to be taken into account when formulating a mix in order to achieve the target strength. Two calcium silicates, C_3S and C_2S, influence OPC's strength development (Stab, 2002; Falciglia et al., 2014). Calcium hydroxide is also produced when Portland cement is hydrated and reacts with pozzolanic materials present in the stabilised soil with the result that further cementitious material is produced (Sherwood, 1993).

2.2.2.2. Lime

Lime increases the strength of stabilised soil economically through cation exchange capacity with no pozzolanic reaction (Sherwood, 1993). By causing the particles to flocculate, lime changes clay from its normal plate-like structure to needles that form metalline structures by interlocking with each other. A clay soil stabilised with lime not only dries but increases its resistance to changes in the water content (Rogers and Glendinning, 1996). There can also be a pozzolanic reaction when pozzolanic materials react with lime in the presence of water to produce cementitious compounds (Sherwood, 1993; Stab, 2002). The effect can be brought about by either quicklime, CaO or hydrated lime, Ca $(OH)_2$ and slurry lime, too, will work on a dry soil though it may need water for compaction (Hicks, 2002). Quicklime, however, is the form of lime most often used because of its advantages over hydrated lime (Rogers and Glendinning, 1996):

- Quicklime has a higher free lime content per unit of mass
- Quicklime is denser so that it requires less space for storage and involves less dust
- Heat generated not only leads to a faster increase in strength but also reduces the moisture content to a greater extent

Quicklime, when added to a wet soil, will immediately absorb up to 32% of its own weight of water to form hydrated lime. Heat generated in this reaction reduces the water content and increases the plastic limit of the soil through evaporation (Stab, 2002; Sherwood, 1993). This effect is illustrated in Figure 2.1. Adding 2% of lime to a soil with 35% moisture content and a 25% plastic limit increases the plastic limits to 40% so that instead of being 10% less than the moisture content it is now 5% in excess of it (Sherwood, 1993). The same study showed cation exchange with calcium ions to be responsible for the initial plasticity reduction – the water affinity for clay is greater with calcium ions which replace hydrogen and sodium cations. Even in soils in which the clay is saturated with calcium ions, as in calcareous soils among others, the addition of lime raises the pH and so increases the exchange capacity. Lime has an ability similar to that of cement to react with minerals in wet clay, increasing the pH and thereby also increasing the solubility of aluminous and siliceous compounds which then react with calcium in the formation of calcium silica and calcium alumina hydrates. These are cementitious products of a type similar to those found in cement paste. Natural pozzolanic materials containing alumina and silica have demonstrated a strong potential to react with lime; such materials include clay minerals, PFA (pulverised fly ash) and blast furnace slag. Stabilisation with lime is most often used in geotechnical and environmental applications including the encapsulation of contaminants and, in cohesive wet soils, backfill rendering. Other suitable applications include the stabilisation of slopes, capping highways, and foundation improvement using lime piles and columns of soil to which lime has been added as a stabiliser (Ingles and Metcalf, 1972). On the other hand, the presence of sulphur organic materials can impede lime stabilisation because gypsum and other sulphates have a negative effect on soil strength when the presence of lime causes them to swell.

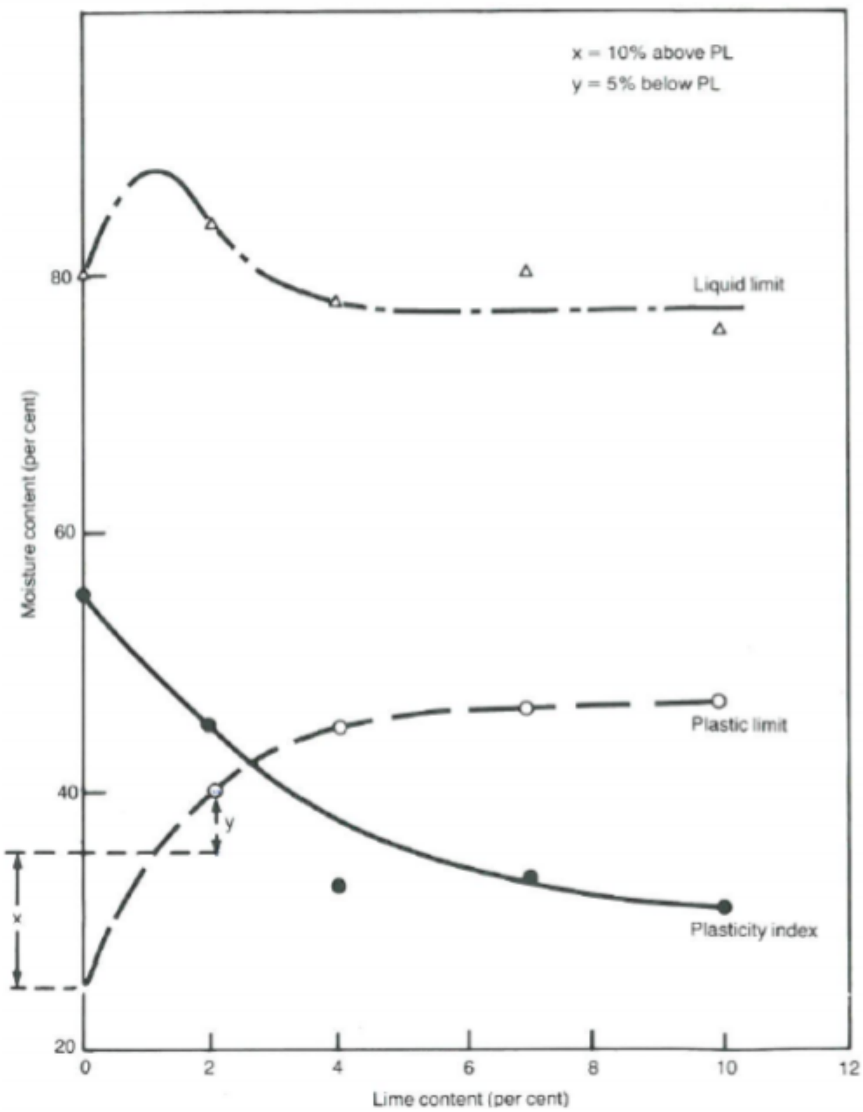


Figure 2.1: Effect of the addition of the lime on plasticity properties of London clay (Sherwood, 1993)

2.2.2.3. Fly-Ash

Electricity generation using coal produces fly as a by-product. Fly ash has limited cementitious properties when compared with cement and lime, and fly ash is used mostly as a secondary binder – secondary binders cannot achieve the required effects alone but can improve strength in soft soils by forming cementitious compounds when mixed with a small amount of activator. Fly ash is readily available, cheap and environmentally friendly. Two main classes of fly ash exist: class C and class F (Bhuvaneshwari et al., 2005). Burning sub- bituminous coal produces Class C fly ash which has high cementing properties because of its high free CaO content. Burning lignite produces Class C with the highest CaO at more than 30% and this confers good self-cementing properties (Beeghly, 2003). Burning anthracite or bituminous coal produces Class F fly ash which has lower self-cementing properties because of the restricted amount of free CaO available to flocculate clay particles. Class F fly ash therefore requires the addition of cement, lime or some other activator. Soil treated with fly ash has a lower potential to swell as a result of mechanical bonding rather than ion exchange with clay minerals (Mackiewicz and Ferguson, 2005). Soil stabilised with fly ash is subject to a number of limitations (White et al., 2005):

- It may be necessary to dewater to bring down moisture content in the soil to be stabilised.
- Curing a soil/fly ash mixture at sub-zero temperatures and then soaking it with water makes it very liable to slaking and strength loss.
- Sulphur in the fly ash can leave expansive minerals in the soil.
- There may be a reduction in durability and long-term strength.

Figure 2.2: A road reclaimer mixes soil with moist conditioned fly ash (Beeghly, 2003).

2.2.2.4. Blast furnace slags

Pig iron production produces blast furnace slags as a by-product. While their chemical composition is not unlike that of cement, blast furnace slags are not themselves cementitious compounds, but their latent hydraulic properties can be brought out when an alkaline material such as lime is added (Sherwood, 1993; Åhnberg and Holm, 2017). According to (Sherwood, 1993), these slags may be available in three forms depending on how they were cooled:

1. Air-cooled slag

Slag may be left in the open to cooled slowly after leaving the blast furnace, in which case crystallised slag will be produced suitable for crushing to use as an advocate.

2. Granulated (merit 5000) or Pelletised slag

Hot slag can be quenched, or cooled suddenly with water or air, to produce vitrified slag. Quenching with water produces granulated blast furnace slag or Merit 5000 (commonly used in Sweden) while pelletised slag is produced by quenching with air.

3. Expanded slag

When hot slag cools, it produces steam and, under certain conditions, expanded slag can be produced.

2.2.2.5. Lignin

As used in this study, lignin is a by-product of paper manufacturing. It is pozzolanic, is a powder yellow – brown in colour, has a distinctive smell and insoluble in deionised water. It does not biodegrade and is capable of maintaining the ductile behaviour of a stabilised soil while at the same time increasing strength and stiffness (Karol, 2003; Chen, 2004; Vinod et al., 2010). Globally, the paper making industry produces more than 50 million tons each year [39]. It is very cheap and compares very well for cost with other stabilisers.

2.2.2.6. Pozzolanas

A pozzolana it is an aluminous or siliceous material with no particular cementitious value but, finely divided and in the presence of moisture, will react chemically with chemical hydroxide to produce cementitious compounds (ASTM 595). No heat is required for this reaction to take place. Pozzolanic clay minerals include: illite, kaolinite, mica, and montmorillonite. Artificial pozzolanas exist and include ashes produced when clays, shales, and siliceous rocks containing pozzolanas heated. Burning a plant leaves in the ashes the silica that the plant absorbed from the soil as a nutrient; this is a pozzolana. Excellent pozzolanic materials rich in silica can be produced from the ash left when bagasse, rice husks and rice straw are burned (Sherwood, 1993).

2.3. Factors affecting the strength of stabilised soil

The strength of a stabilised soil can be negatively affected by the presence of organic matter, carbon dioxide, sulphates and sulphides (Sherwood, 1993).

2.3.1 Organic matter

The top layers of most soils contain generous amounts of organic matter and, in soils that are well drained, this material can be as deep as 1½ metres (Sherwood, 1993). This organic matter lowers the pH value when it reacts with hydration products including calcium hydroxide ($Ca(OH)_2$) and a reduction in pH can slow down hydration and make it difficult (and sometimes impossible) to compact the stabilised soil effectively.

2.3.2 Sulphates

Stabilising soil rich in sulphates with a calcium-based stabiliser will cause the stabilised soil in the presence of excess moisture to form ettringite (calcium sulphoaluminate) and/or thamausite. Either of these takes up a volume greater than the combined volume of the reactants but it may be necessary to introduce further excess water while mixing in order to dissolve sulphate if the reaction is to take place (Sherwood, 1993).

2.3.3 Sulphides

A number of waste materials and industrial by-products contain sulphides in the form of iron pyrites (FeS_2) which, as it oxidises, produce sulphuric acid. In the presence of calcium carbonate, the acid can react as follows to produce either gypsum or hydrated calcium sulphate:

i. $2FeS_2 + 2H_2O + 7O_2 = 2FeSO_4 + 2H_2SO_4$

ii. $CaCO_3 + H_2SO_4 + H_2O = CaSO_4.2\,H_2O + CO_2$

In the presence of excess water, this hydrated sulphite can attack the stabilised material in exactly the same way as sulphate (Sherwood, 1993). Natural soils, though, may also contain gypsum (Little and Nair, 2009).

2.3.4 Compaction

The density of the soil can be significantly impacted by the addition of binder and the stabilised mixture's maximum dry density will be lower than that of unstabilised soil for any given degree of compaction. The optimum moisture content will increase when binder content is increased (Sherwood, 1993). When cement has been used as the stabiliser, hydration will start as soon as the cement is in contact with water and the process resulting from hydration makes the soil mix harder and necessitates immediate compaction of the soil mix; if compaction is delayed, the hardening of the stabilised soil may mean that additional compaction is required resulting in broken bonds and lost strength. This is most likely to happen in stabilised clay soils (Figure 2.1) because of alterations in the plasticity of the clay (Sherwood, 1993). Compared with stabilisation using cement, delaying compaction after lime stabilisation can be advantageous because a soil stabilised with lime needs a mellowing period to give the lime time to diffuse throughout the soil for maximum effect on plasticity. When the mellowing period ends, lime-stabilised soils can be remixed before a final compaction which results in greater strength (Sherwood, 1993).

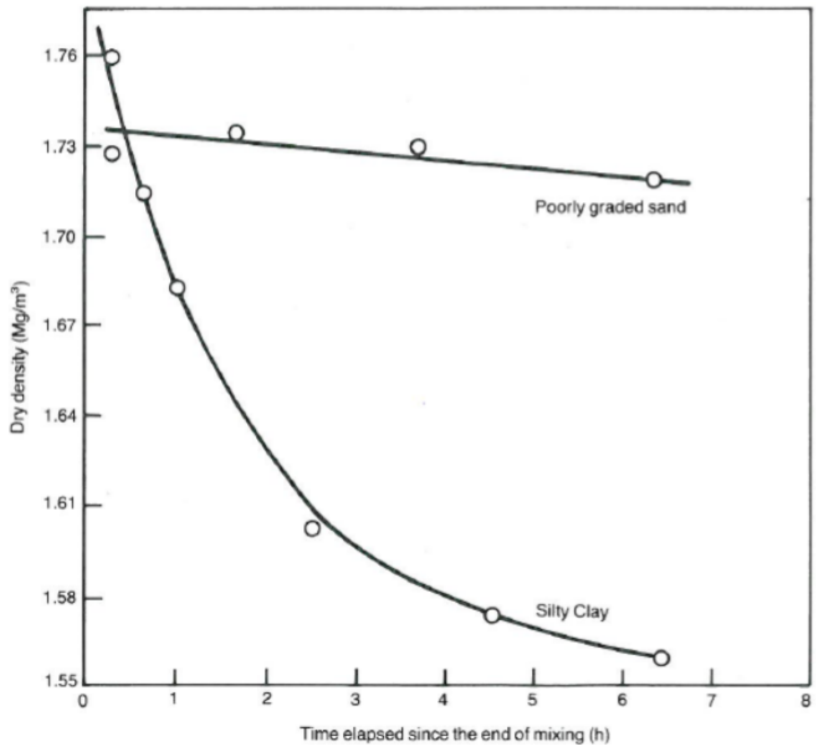


Figure 2.3: Dry density versus time elapsed since the end of mixing of two material stabilized with 10% cement (Sherwood, 1993).

2.3.5 Moisture content

The moisture content of stabilised soil must be sufficient not only for hydration but also so that the soil can be efficiently compacted. Fully hydrated cement absorbs up to 20% of water (Sherwood, 1993) and, for quicklime, the figure is 32% (Rogers and Glendinning, 1996; Sherwood, 1993). Insufficient moisture in the soil means that the soil and the binders are in competition for it and hydration can be retarded and the final strength diminished when organic soils, clay, peat and other soils with high water affinity do not find enough water in the soil for their purposes.

2.3.6 Temperature

Pozzolanic reactions are affected by the temperature and on-site temperatures vary continuously throughout the day. Pozzolanic reactions between binders and particles of soil slow down when temperatures are low, reducing the strength of the stabilised soil. In cold regions, it may be best to delay stabilisation until a warm season (Sherwood, 1993; Maher et al., 2004).

2.3.7 Freeze-thaw and dry-wet effect

Stabilised soils react badly to a cycle of freeze and thaw and it may be necessary to protect them from frost damage (Maher et al., 2004; Al Tabbaa and Perera, 2006). Chemical reactions with the binder in a stabilised soil govern shrinkage, and cement-stabilised soils can be subject to frequent wet and dry cycles simply as a result of the temperature changing through the day. Stress introduced in this way and was stabilised soil will require that the soil be protected (Maher et al., 2004).

2.4. Stabilisation methods

2.4.1. In-situ Stabilisation

Soils can be stabilised in-situ or ex-situ (that is, off-site). In-situ stabilisation involves the addition of stabilising agents to the soil where it is to be used. Contaminated soils and foundations, whether deep or shallow, can be improved in this way and the mix should be formulated after an assessment of the engineering properties of the stabilised soil and of the improved ground to calculate the dimensions of the improved ground in terms of how much settlement and what degree of stability the planned structure will need for its support (Keller, 2011). Lime or cement, wet or dry, as well as other cementitious materials, can be injected into the soil. The soil’s condition and moisture content, combined with how effective the intended binders are and the type of construction to be carried out, will decide whether the binders should be mixed dry or wet. In-situ stabilisation will be deep mixing or mass stabilisation according to the depth at which the soil is to be treated.

2.4.1.1. Deep mixing method

In deep mixing, the soil is stabilised at some depth by injecting into the ground a wet or dry binder which is then mixed with the soil already there using a rotary or mechanical mixing tool (Stab, 2002). As shown in Figure 2.4, the pattern produced may be a block pattern, a single pattern, a panel pattern or a stabilised grid pattern (Stab, 2002); in each case, the purpose is the production of a mass of stabilised soil able to interact with the soil that is already there. The purpose is not the production of a mass that has been stabilised stiffly and is capable of carrying the design load unassisted. The stabilised soil should not have sufficient stiffness or strength to inhibit load distribution an effective interaction with the natural soil (Stab, 2002). The objective is to distribute the design load in such a way that is shared by the stabilised soil and the natural soil.

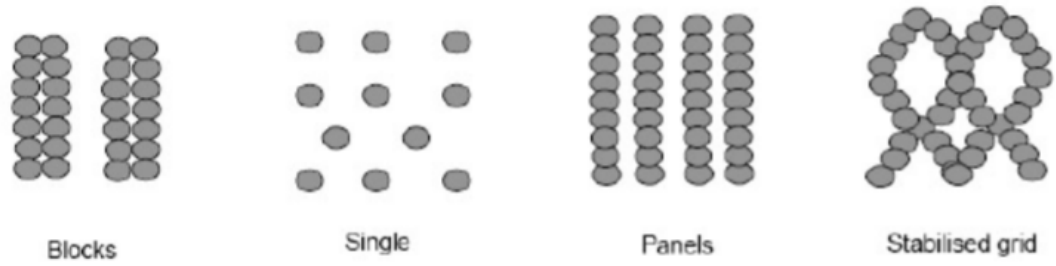


Figure 2.4: Typical patterns of deep soil mixing (Stab, 2002)

1. Wet mixing

In wet deep mixing, the binder is converted into a slurry to be injected into the soil by nozzles at the soil auger's tip (Massarsch and Topolnicki, 2005). The mixing tool is a combination of drilling rod, transverse beams, and a drill end to which is attached a head. The conditions for the nature of the application may call for modification of the method. For example, TRD (Trench cutting Re-mixing deep method) requires no open trench and builds a continuous cut-off wall. The crawler-mounted mixing tool resembles a chainsaw and blends a cementitious binder into the natural soil to build what amounts to a soil/cement wall. Injection ports in a rotating chain of teeth that both cut and scratch deliver the grout to the area under treatment. Maximum depth the wall can be 45 metres and the width may be between 0.5 m and 0.9 m. Permeability between 1 x 10-6 and 1 x 10-8 cm/s will make this very effective in excluding groundwater. The FMI machine resembles the TRD machine cutting blades rotate on its cutting arm or trencher which is dragged through the soil by the power unit and can incline up to 80 degrees (Stocker and Seidel, 2005). Once again, instead of being excavated the soil is mixed with a binder in the form of slurry administered through the cutting arm's vents (Figure 2.5).

Figure 2.5: The FMI and TRD–Trenching machine for construction of deep walls (Massarsch and Topolnicki, 2005)

Figure 2.6: Parts of wet mixing tool showing injection of slurry into the soil (Porbaha et al., 2005)

2. Dry Mixing

Dry mixing (DM) has the advantage of being quiet and clean, producing little vibration and no soil requiring disposal. It is used extensively in northern Europe and Japan and works by injecting dry binders into the moist soil and mixing the two thoroughly (Figure 2.13). The soil is premixed by a tool designed for the purpose while penetrating the soil to the required depth. Dry binder is injected simultaneously with withdrawal of the mixing tool and then mixed with the premixed soil leaving a column of moist soil mix. This method is known as LCC (Lime Cement Column) in Scandinavia and particularly in Sweden. It is known as Trevimix in Italy and as DJM (dry jet mixing) in Japan (Yasui et al., 2005).

Figure 2.7: Bauer cutter soil mixing (Fiorotto et al., 2005)

A typical DM machine consists of a drill motor and an installation rig, both of which are track-mounted. Binder is pumped by compressed air through the hose into the mixing shaft which delivers it into the ground (Figure 2.13). There is no need to convert the powdery binder into a slurry before injecting it by compressed air into the ground. A cavity is created in the soil by rotating the blade and then filled with binders. The machine will most efficient if kept as far as possible within its operational radius (Stab, 2002). The principle and the mixing blade detail are presented in Figures 2.9 and 2.10 (Stab, 2002; Yasui et al., 2005). The compressed binder, once it has been thoroughly mixed with the native soil, becomes a hard column of which the maximum depth is 40 metres and the maximum diameter 1.5 metres (Stab, 2002). Penetration of the soil can cause vibration which may reduce the strength of a sensitive soil, making it necessary for some binder to be injected into the ground during the process of penetration. Changing the amount of binder can adjust the strength of the stabilised soil column which may be anywhere on a scale from high to low and greatly improved ratio can be achieved by overlapped mixing or interlocked columns Other uses for this method are embankment stabilisation, slope protection, foundation improvement, and the mitigation of liquefaction (Yasui et al., 2005). This method's effectiveness will be governed by how much moisture is in the soil and it is not advisable for sandy soils with less than 30% water content (Nozu, 2005).

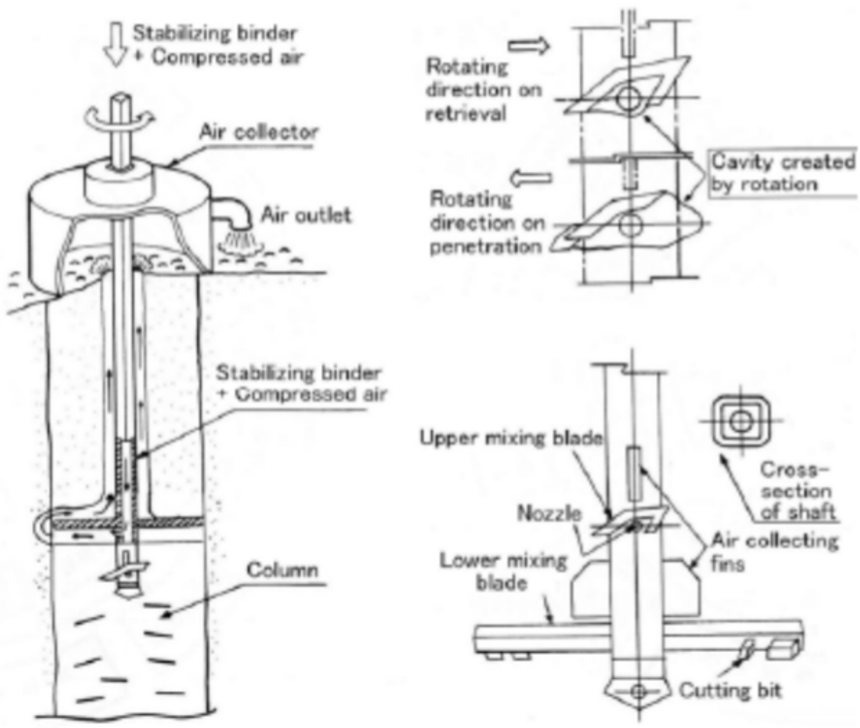


Figure 2.8: Schematic diagram of construction principle and structure of mixing blade (Yasui et al., 2005)

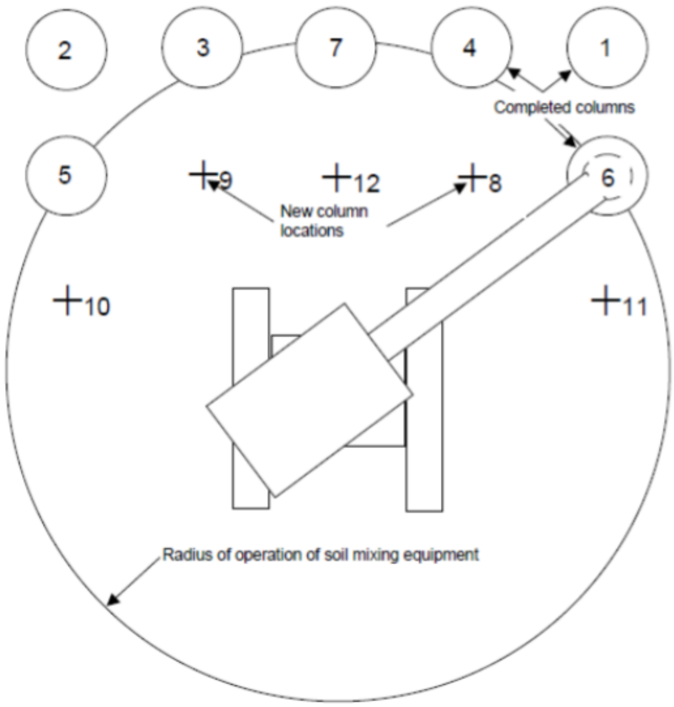


Figure 2.9: Sequence of operation for deep soil mixed columns (Stab, 2002)


Figure 2.10: Nordic dry mixing “standard” tool (Larsson, 2005)


Figure 2.11: Nordic modified dry mixing tool (Larsson, 2005)

Figure 2.12: Injection of dry binder into the soil from the mixing tool (Keller, 2011)

2.4.1.2. Quality Control and Quality Assurance

For deep mixing to work properly, automatic QC (quality control) and QA (quality assurance) are essential and this is taking care of by monitoring instruments in the mixing machine and binder feed to control where the column is positioned, to ensure that mixing is proportional, to dictate how much binder is used, and to control the speed of penetration and withdrawal (Stocker and Seidel, 2005; Yasui et al., 2005) (Figure 2.14).

Figure 2.13: Online quality control/quality assurance of construction parameters (Stocker and Seidel, 2005)

2.4.1.3. Applications

Deep mixing methods, whether for geotechnical or environmental purposes, may fall into either of two main categories: structural and non-structural. Included among non-structural methods are: ground cut-off and/or dewatering walls, containing contamination, and secondary containment. Included non-structural purposes are: foundations, both deep and shallow, retaining walls and talents, cut stabilisation and open excavation. (Porbaha et al., 2005) defined six areas for deep mixing applications: hydraulic barriers, retaining walls, foundation supports, excavation supports, liquefaction and seismic mitigation, and environmental remediation. Deep mixing applications in foundation engineering can be for storage tanks, dome silos, heavy machinery, highway embankments and rail systems. Figures 2.15 and 2.17 show both deep and shallow foundations.

Figure 2.14: Railroad bridge supported by deep mixing column at San Francisco International Airport (Porbaha et al., 2005)

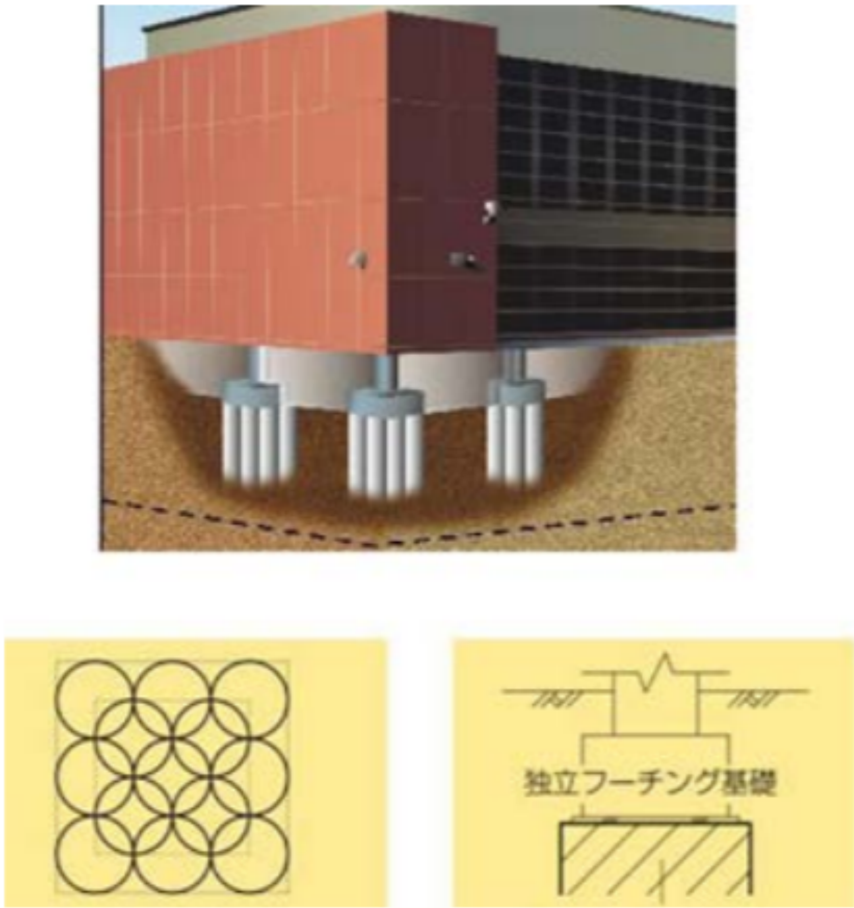


Figure 2.15: Application of deep mixing in building foundation (Nozu, 2005)

Figure 2.16: Foundation for A2 motorway bridge near Katowice (Nozu, 2005)

1. **Hydraulic barrier support systems**

Deep mixing in hydraulic structures to install a cut-off wall facilitates flood and seepage control and the installation of piping. The use of TRD to construct a groundwater barrier is shown in Figure 2.18. The site is the Herbert Hoover Dike in the USA.

Figure 2.17: Top: TRD equipment at work, bottom: Inspection of the exposed TRD wall at Herbert Hoover Dike surrounding Lake Okeechobee in south-eastern Florida (Inc., 2012)

2. Retaining wall systems

Free-standing walls can also be built using deep mixing, as Figure 2.19 shows. The wall may be there for soil retention and the commonest uses of retaining walls are in water bulkheads, ports and harbours, secant walls and open excavations.

Figure 2.18: Reinforced deep mixing retaining wall (Porbaha et al., 2005)

a. Excavation support systems

Deep mixing is to be preferred when building supports for open excavations as well as underground construction including excavation and braced excavation, trenches for railway tracks and cut and cover tunnels (Figure 2.20).

b. Seismic and Liquefaction mitigation systems

Foundations can be seismically retrofitted using deep mixing, which also comes into play to mitigate lateral spreading and liquefaction in riverbanks and culvert foundations. Dune stabilisation and levee strengthening are another common DM applications (Figures 2.24 and Figure 2.25) (Porbaha et al., 2005; Yasui et al., 2005). The main stabilisation objective here will be to reduce pore water pressures, increase the shear strength of soils with a propensity to liquefy (Stab, 2002) and to minimise the propagation of waves in infrastructure superstructures and substructures (Figures 2.21, 2.22 and 2.23 (Holm et al., 2002)).

Figure 2.19: Top: Trench excavation railroad for an Alameda corridor project. bottom: Structural cut-off wall during construction of new facility at Harvard University, Cambridge (Porbaha et al., 2005)

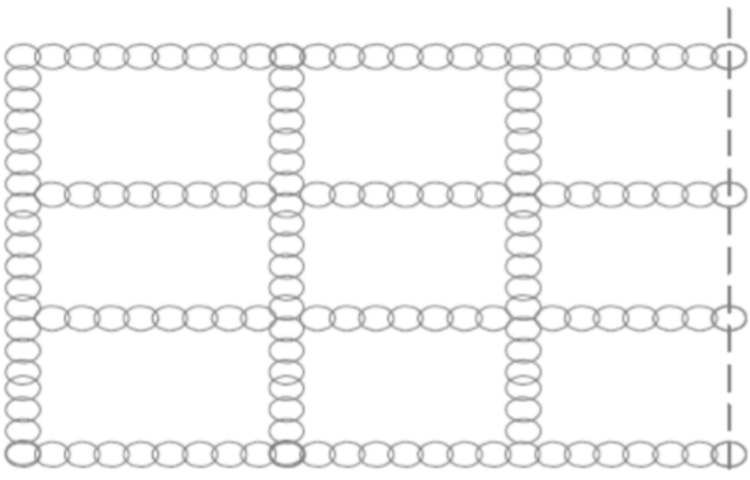


Figure 2.20: Example of panel pattern in liquefaction mitigation (Stab, 2002)

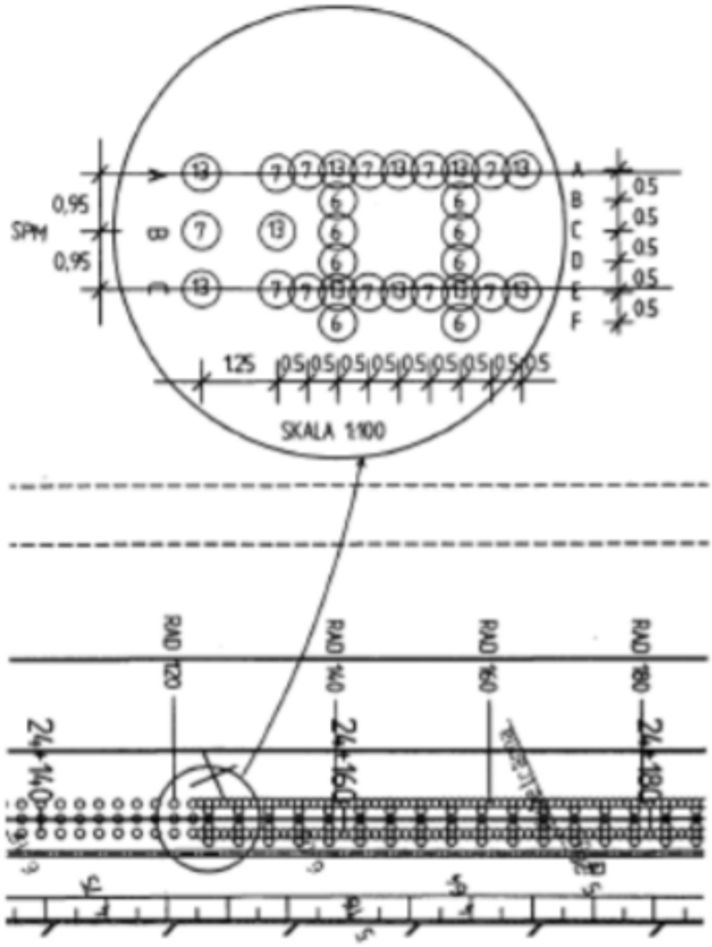


Figure 2.21: Panel installation pattern for vibration mitigation caused by high speed train at the Ledsgård, Gothenburg, Sweden (Holm et al., 2002)

Figure 2.22: Vibration mitigation using dry deep mixing method; column installation in progress while commuter train passing (Holm et al., 2002)

Figure 2.23: Liquefaction mitigation along river bank at Napa Yacht club, California (Porbaha et al., 2005)

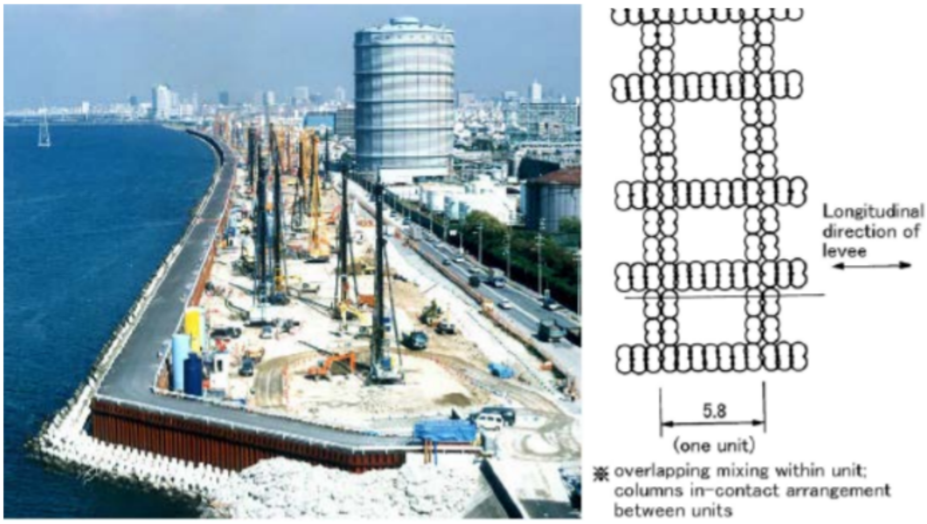


Figure 2.24: Application of DJM and resulting columns at Yodogawa river embankment in Japan (Yasui et al., 2005)

2.4.1.4. Mass Stabilisation

Mass stabilisation may be deep or shallow and is intended to stabilise soft soil to a specific depth (Figure 2.26). The technique is fairly new and is indicated when high moisture content soils are to be stabilised. Contaminated sediments, silts, clays and contaminated sediments are all suitable for this treatment (Yasui et al., 2005; Inc., 2012) which is particularly cost-effective in cases of high water content or where high volumes of contaminants are present. It is well suited for deposits of contaminated dredged sediment, organic soils and waste sludge (Keller, 2011) and has advantages in such cases over removal and replacement, which have been the alternatives in traditional use.

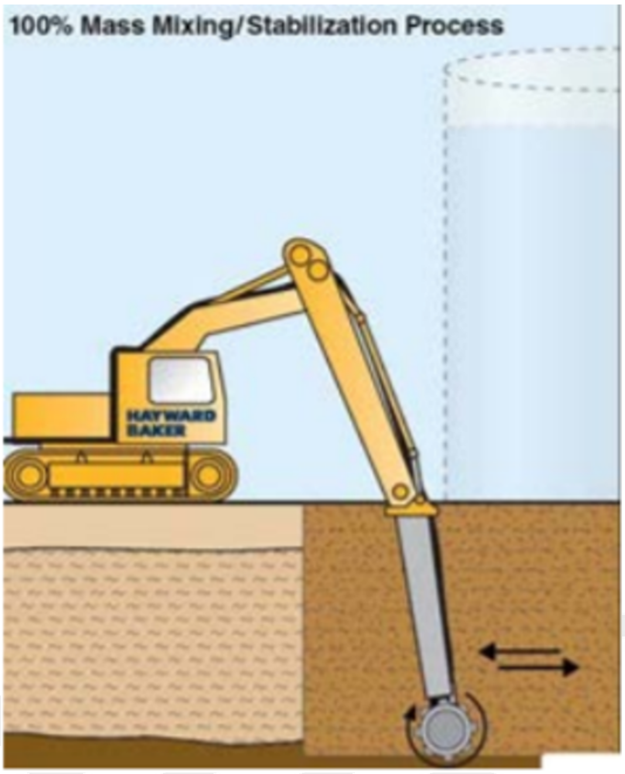
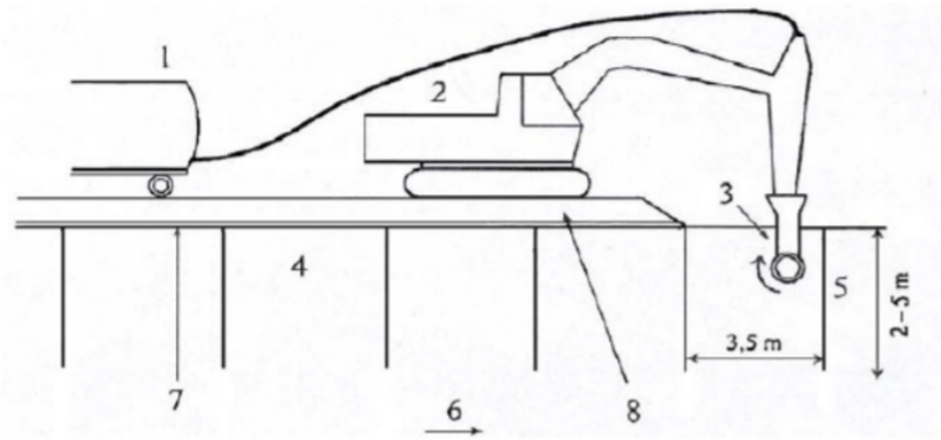


Figure 2.25: Mass mixing stabilization (Inc., 2012)

Figure 2.28 shows one way of carrying out the operation, with binder injected through a rotating auger or mixing head, while Figure 2.29 shows the soil mass being blended by a mixing tool mounted on an excavator with shuttles to deliver the binder pneumatically to the mixing tool head. The rotating mixer moves vertically and horizontally as it mixes the soil block. The diameter will usually be between 600 mm and 800 mm and the speed of rotation between 80 and 100 rpm. Normal practice is to stabilise the soil in a block, the size of which will define the machine's operating range which will normally be between 8 and 10 m^2 in plan and between 1.5 and 3 m in depth. Output is from 200 to 300 m^3 of stabilised soft soil in each shift (Figure 2.27). Typical rates of binder application are 200 to 400 kg/m^3 (Inc., 2012).



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

Figure 2.26: Schematic diagram of mass stabilisation (Massarsch and Topolnicki, 2005; Stab, 2002)

In Nordic countries, binder is typically applied at between 150 and 250 kg/m^3, and the target shear strength is 50 kPa (Massarsch and Topolnicki, 2005). Development means that it is now possible to use rapid cement as the binder and this has been applied in the stabilisation of contaminated dredged material at Port Hamina and the Helsinki, Finland shoreline on which embankments are formed to create a new area before dredged contaminated dredged material is deposited between them (Inc., 2012). A geotextile is placed on the stabilised mass before it sets; placing a granular base course on the geotextile compresses the stabilised mass and forces out air pockets that have formed during mixing (Massarsch and Topolnicki, 2005). According to EuroSoilStab, (Stab, 2002) deep stabilisation is an improvement on other methods because: (Figure 2.30) it is flexible and cost-effective; it economises on energy costs and materials; the engineering properties of the soil rapidly improve; and, because there is no settlement, it can be flexibly linked to its surroundings and to other structures .

Figure 2.27: Dry mass soil mixing to strengthen soft soils beneath a planned roadway expansion at U.S. Highway 1, Key Largo, Florida (Inc., 2012)

2.4.2. Ex-situ stabilisation

In ex-situ stabilisation, which is especially applicable to the dredging of harbours and rivers since there is really no alternative, the soil is taken away and treated somewhere else. Whether the dredging is to reduce the toxicity of contaminated sediments or to keep navigation channels safe for vessels to pass (Epa, 2004), the sediments are treated in off-site confined disposal facilities (CDF) before being taken to a selected site. Planning such an operation requires knowledge of the removal method, the transport method, the available treatment locations and whether there is a demand for the material after it has been stabilised (PIANC, 2009). What treatment is used at a CDF is dependent on the sediment's nature and water content (Figure 2.31).

Figure 2.28: Mass stabilisation with dry soil mixing of soft wet organics to control settlement for storage tanks at Port Everglades, Florida (Inc., 2012)

Figure 2.29: Ex-situ for on-site use stabilisation (Inc., 2012)

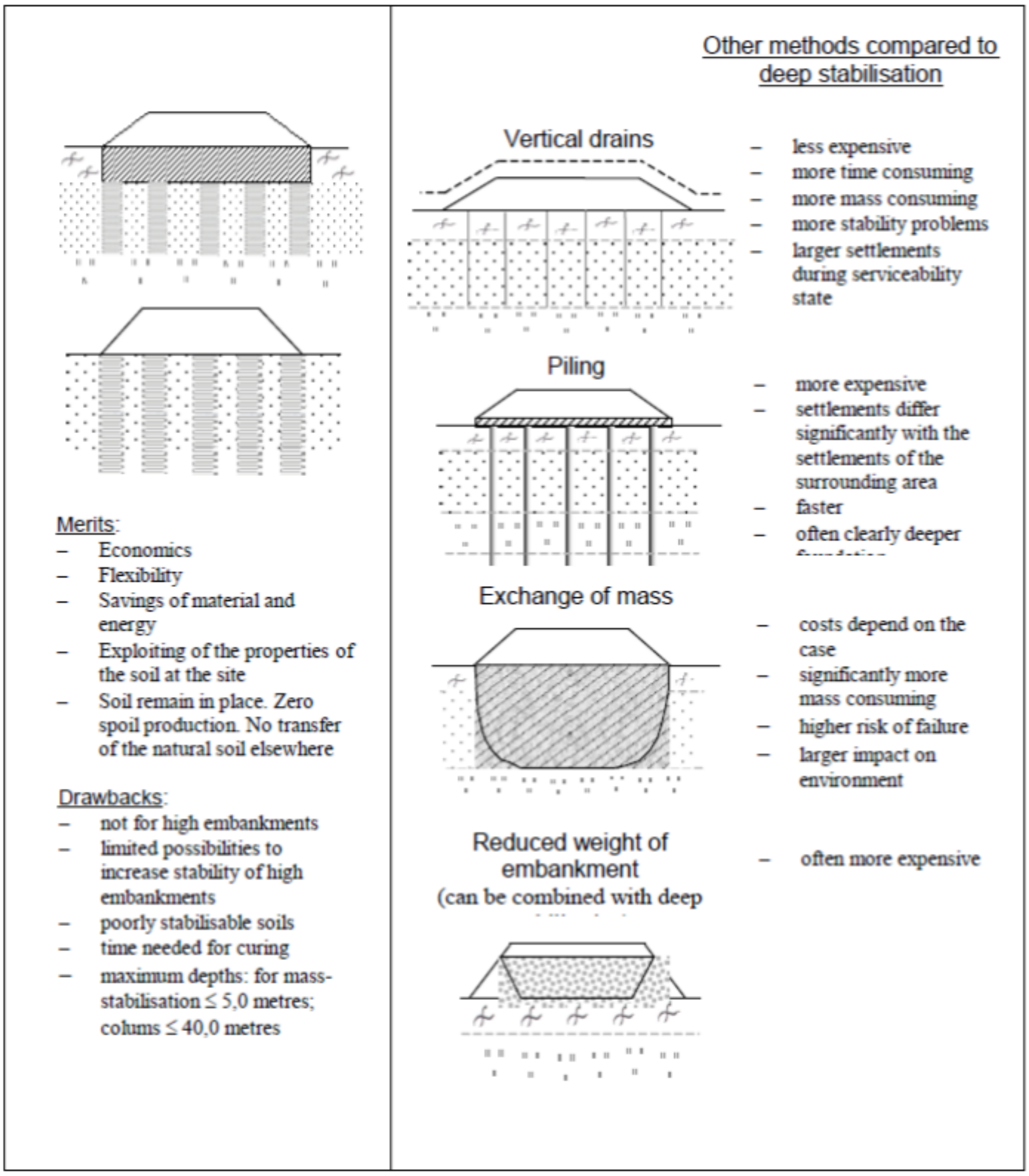


Figure 2.30: Comparison between deep stabilisation method and other methods (Stab, 2002)

2.3. Previous studies

The use of fly ash and its effect on the strength of stabilised soils was researched by Ansary et al. (Ansary et al., 2007) who studied UCS (qu) as well as compaction and flexural properties. In that study, the admixture was fly ash mixed with lime; fly ash additions of 0%, 6%, 12% and 18% were studied, with 3% lime in each case. The study concluded that the strength of the stabilised soil was increased more as the amount of lime/fly ash increased. In comparison with untreated samples, soils treated with fly ash and lime showed significantly greater UCS, with the actual increase dependent on volume of additive and curing time. This was also true of flexural strength and flexural modulus which, in comparison with untreated soil, increased by about 4.6 and 4.7 times and 3 and 4.3 times, in the case of both soils.

Laboratory investigations carried out by Dahale et al. (Dahale et al., 2017) on a clay soil stabilised with a mixture of fly ash and hydrated lime had an effect on the UCS (unconfined compressive strength) CBR (California bearing ratio) and compaction that varied with the amount of lime, the amount of fly ash and the number of days of curing . Figure 2.31 shows the results of an attempt to find the relationship between compressive strength and tensile strength in stabilised mixes. The BTS (Brazilian tensile strength) of soil cured for 56 days after being stabilised with a mixture of fly ash and lime ranged between 22 and 143 kN/m^2 while the range in UCS was between 143 and 2172 kN/m^2. The increase in strength demonstrates that it is possible to stabilise a clay soil productively with a mixture of fly ash and hydrated lime.

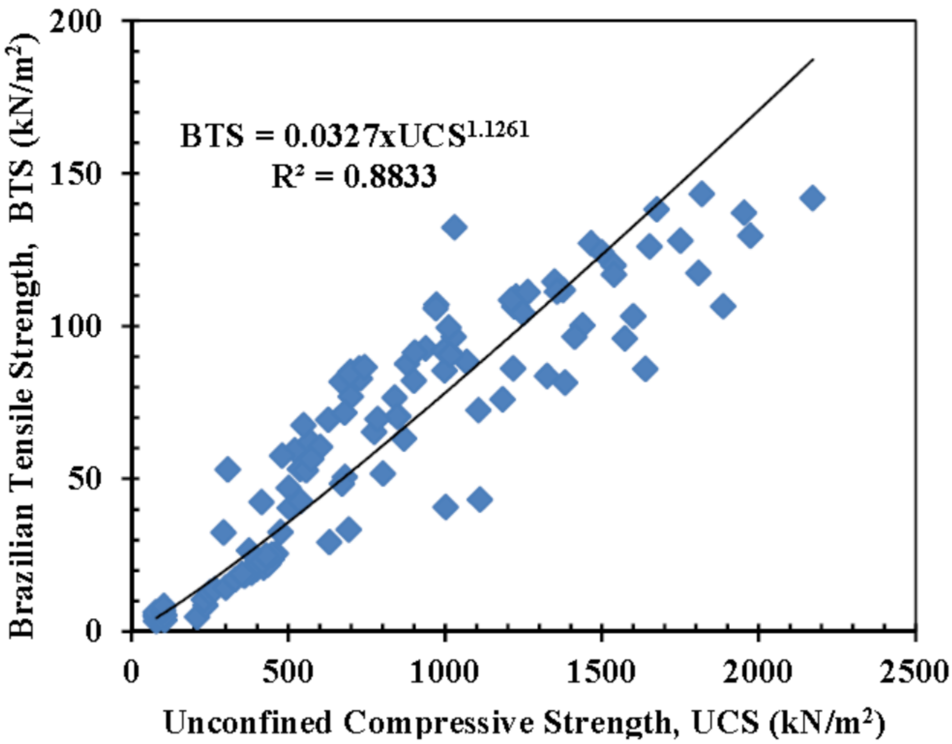
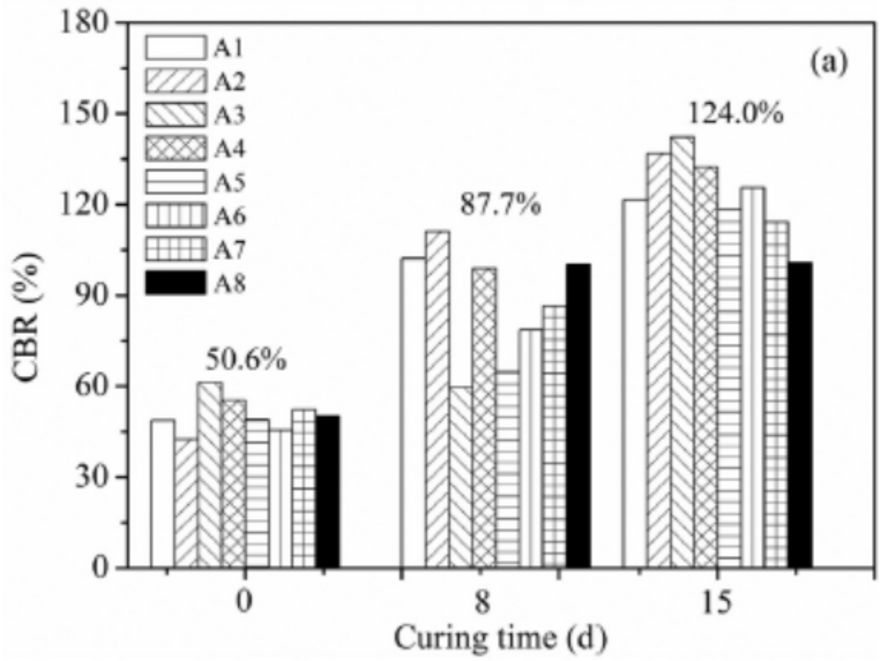
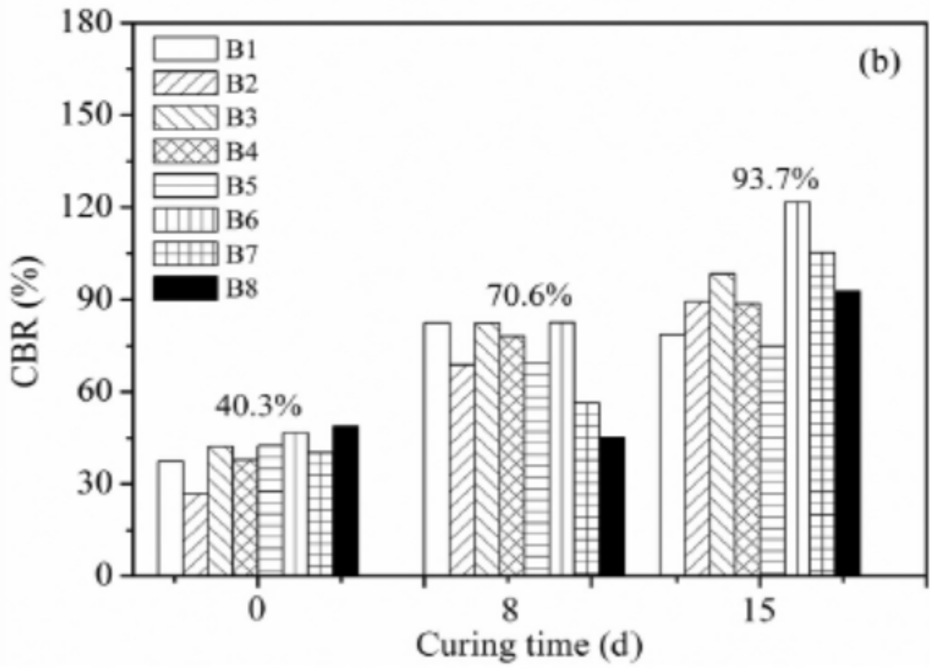


Figure 2.31: BTS/UCS relationship for soil stabilised with fly ash and lime (Dahale et al., 2017)

Zhang et al. (Zhang et al., 2017) conducted a field trial to explore the viability of using silty soil stabilised with lignin as a material for highway subgrade courses. Quicklime, traditionally used for stabilising soils, was chosen as the control. The study presented the construction procedures for a silty subgrade soil that had been stabilised by lignin and by quicklime. After subgrade construction, field tests conducted included CBR

(California Bearing Ratio), Resilient Modulus (Ep), Benkelman beam deflection, and Dynamic Cone Penetrometer (DCP) test to investigate what effect curing time and additive content had on the stabilised silt's bearing capacity and mechanical properties. Compaction degree and moisture content tests were also carried out to assess the compacted subgrade soils' quality. Results indicated that, in the lowest zone of the filled soil layers with 96% compaction, mechanical performance exhibited after fifteen days curing by the 12% lignin stabilized silt was superior to that of the 8% quicklime and these results are shown in Figure 2.32 a, b, and c.





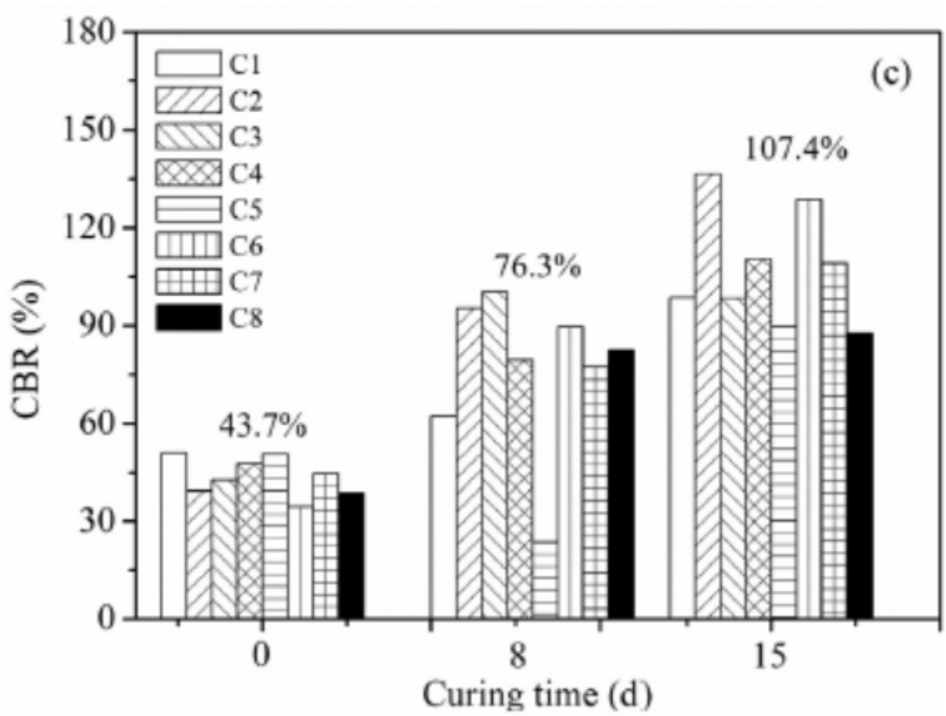


Figure 2.32: Effect of curing time on CBR of the stabilised silt (Zhang et al., 2017): (a) Section A, silt stabilised with 12% lignin; (b) Section B, silt stabilised with 8% lignin; and (c) Section C, silt stabilised with 8% quicklime

Zhang et al. (Zhang et al., 2018) used a non-destructive testing technique to examine lignin-stabilised soils' strength development. They experimented with silty soils stabilised by a variety of additives over a range of durations. At different time intervals, they carried out shear-wave velocity (Vs) and unconfined compression tests to assess the properties and, when the tests were over, they correlated the small-strain shear modulus (G_0) and shear-wave velocity (Vs) tests and analysed unconfined compressive strength (UCS), concluding that there is a logarithmic increase in line with the curing period for both G_0 and UCS of lignin-stabilized soils. G_0 and UCS values for cement-stabilised soils are a great deal higher than those returned by soils stabilised with lignin in the same percentages and cured for the same length of time. The increase in both G_0 and strength shows a common trend after normalisation. If Vs is monitored while the soil is curing, the evolution and strength can be plotted non-destructively. This research has contributed to using shear wave velocity as a non-destructive alternative to traditional civil engineering design methods which have tended to be destructive. Analysis was carried out on lignin by SEM (Scanning Electron Microscope) to ascertain its chemical composition and functional groups and the results are presented in Figure 2.33, from which it can be seen that the main components were carbon (C), oxygen (O), sodium (Na), and sulphur (S).

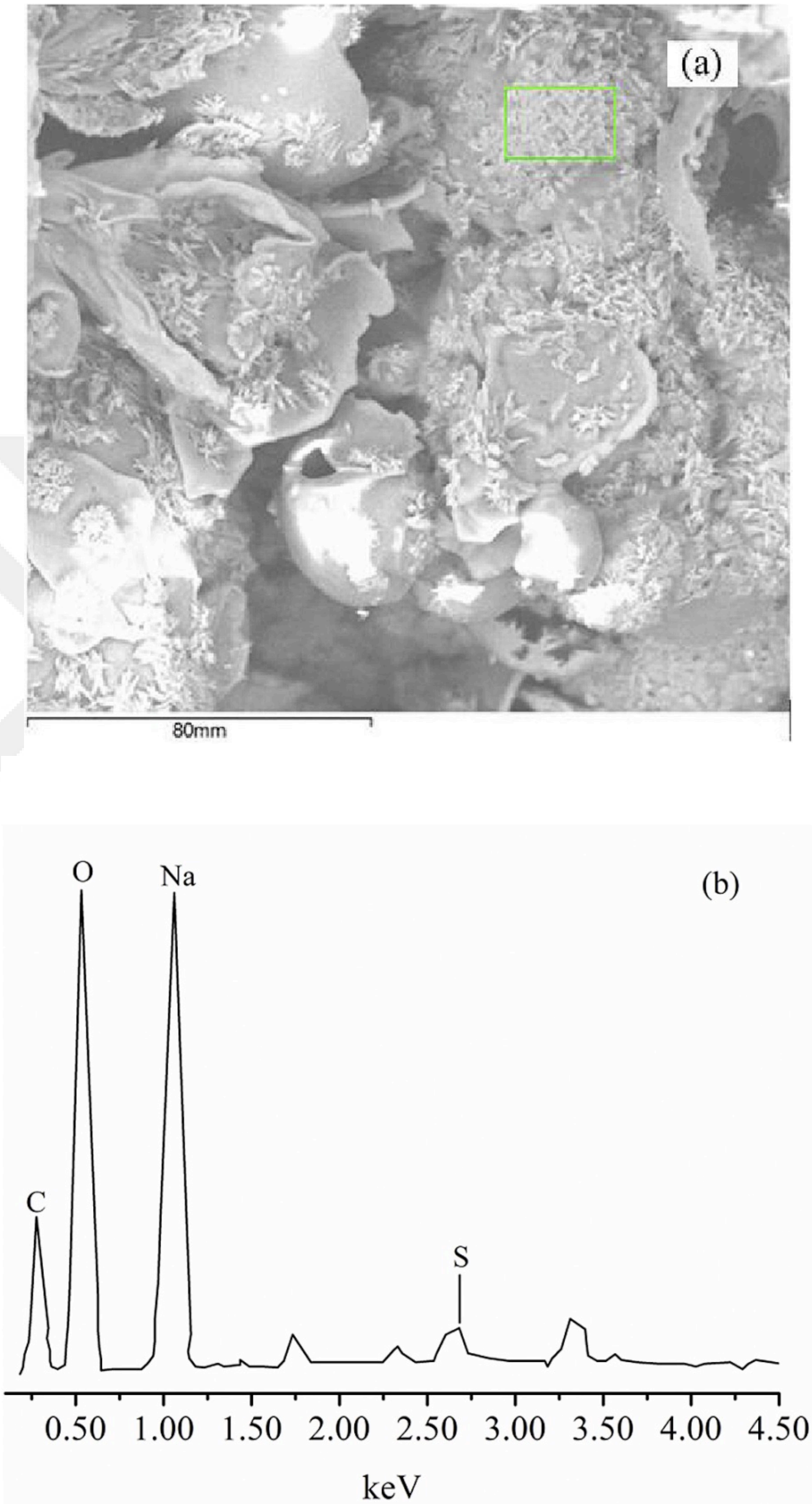
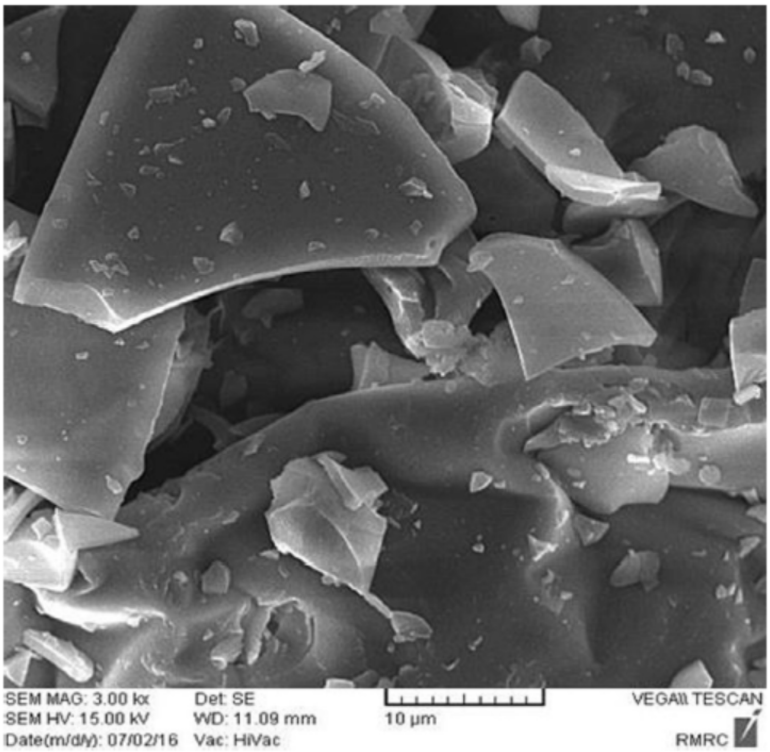
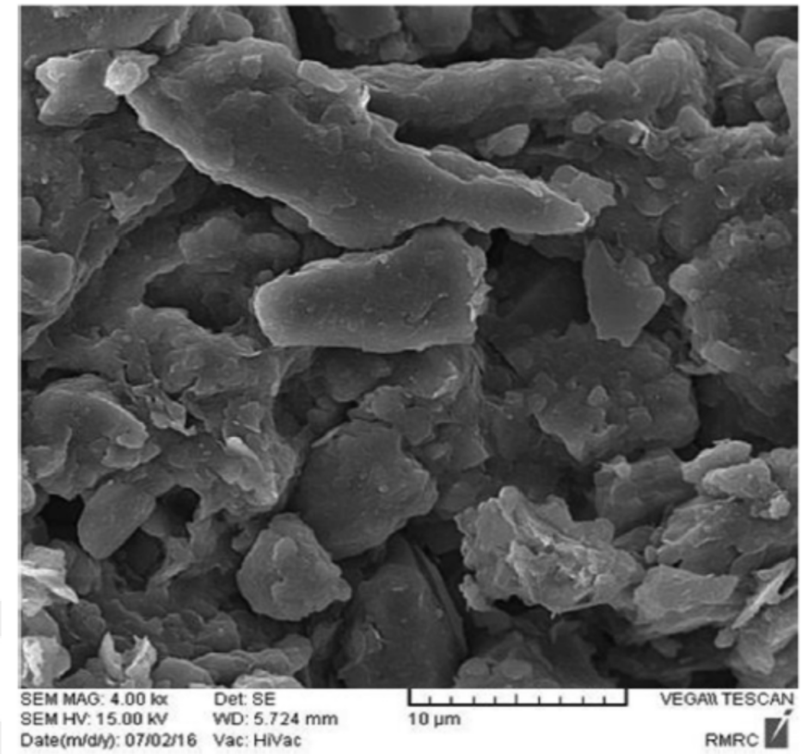


Figure 2.33: Images from scanning electron microscopy of silt stabilised with lignin with: (a) 0% additives and (b) 8% additives after being cured for 28 days (Zhang et al., 2018).

A research study reported in Bahram et al (Ta'negonbadi and Noorzad, 2017) shows the results of an investigation of an alternative stabiliser, Lignosulphonate (LS). SEM research was carried out on LS and on treated clay to understand how strength developed with LS treatment. The results are shown in Figures 2.34a and 2.34b. LS percentages by weight of dry soil were 0.5, 0.75, 1, 2, 3 and 4% and curing times were 0, 4, 7, 14, and 28 days. The results show that treatment with LS considerably reduces the soil's PI (plasticity index) and that stabilising with LS led to a slight increase in optimum water content and a very slight reduction in the soil's maximum dry unit weight. LS stabilisation increased the soil's UCS and stiffness without resulting in unduly brittle behaviour. An electrostatic reaction between soil particles and water containing LS is considered to be the cause of the increase in strength.



(a)



(b)

Figure 2.34: Microscopic image of (a) lignosulphonate particles magnified 3000 times (b) clay with 0.75% LS magnified 4,000 times (Ta'negonbadi and Noorzad, 2017).

A series of UC tests carried out on soil treated with slag cement by Louis Ge et al. (Ge et al., 2018) were conducted at slag cement-to-soil ratios of 15%, 20%, 25%, 50%, and 75%. In order to simulate the improvement of soft ground with cement, they also selected differing water contents of 1.8, 2 and 2.2 times the kaolinite liquid limit. UCS was tested at 3, 7, 14, 28 and 56 days. The conclusion was that strength development slows appreciably once curing time has reached 14 days, with specimens cured for longer than 28 days showing either very slight growth in strength or none at all. There was a rapid reduction in UCS as the water and clay/cement ratio (wc/c) increased for as long as wc/c was lower than 6.0, but values greater than that saw a smoothing in the reduction trend. As the ratio of cement-to-soil (C/S) increases, so does the UCS so that it can be said that UCS increases as the clay-water/cement ratio falls and the cement-to-soil ratio rises. This is shown in Figure 2.35.

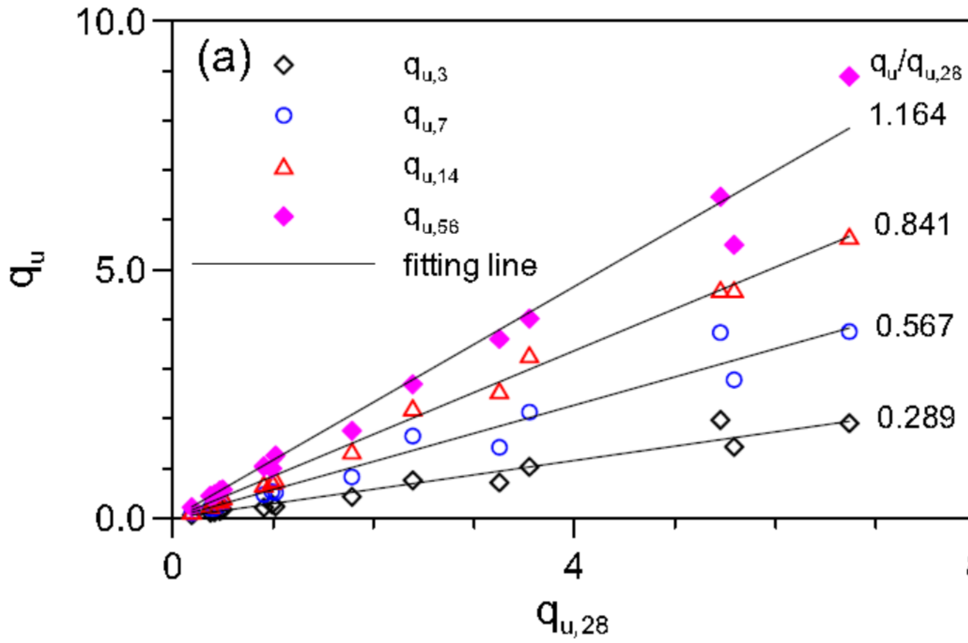
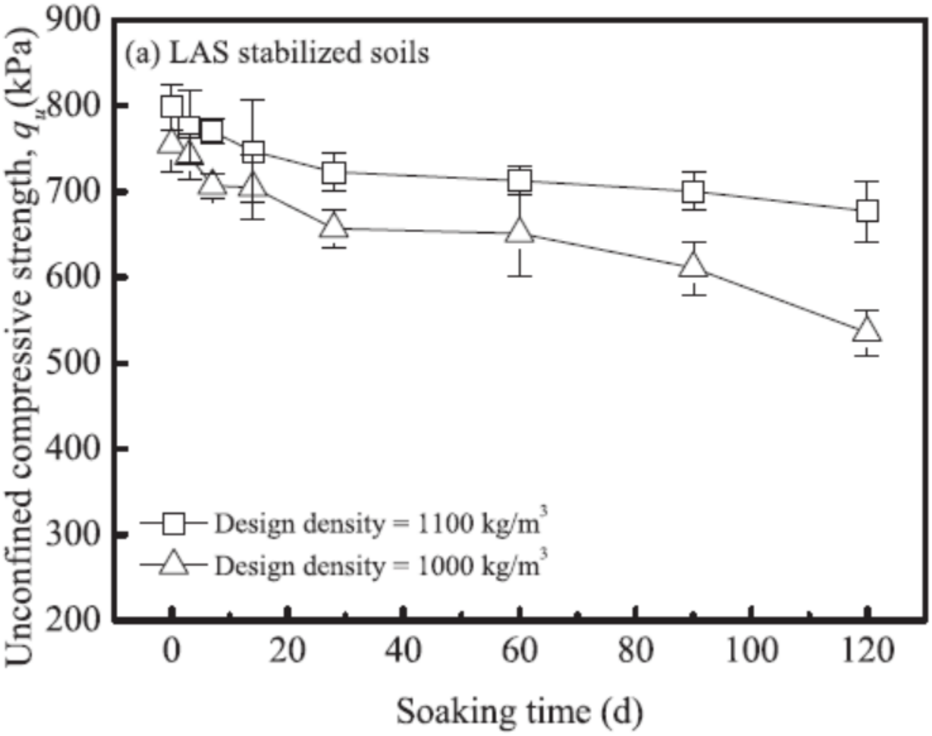
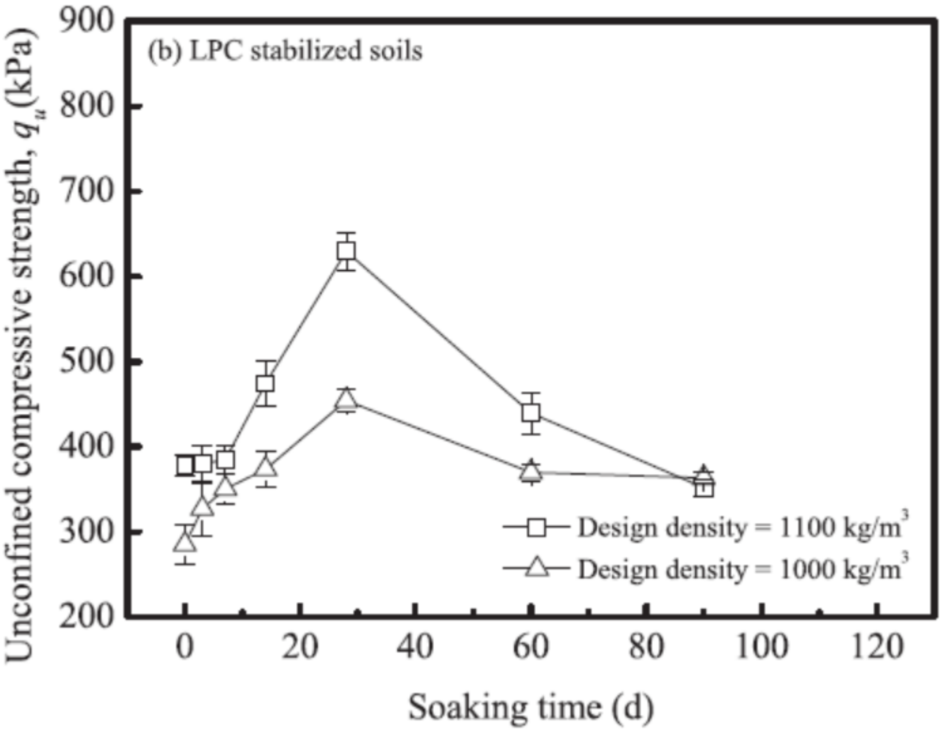


Figure 2.35: How UCS varies at different curing ages and at 28 days (Ge et al., 2018).

Jiang et al. (Jiang et al., 2018) carried out an experiment to research the resistance to sulphate attack of a clay soil stabilised using ground granulated blast furnace slag (LAS), where the whole comprises GGBS, sodium silicate, calcium carbide residue (CCR), air foam, and the clay soil. Testing was by submerging specimens of stabilised soil in a solution of sodium sulphate for a range of time periods. At the end of each period, the percentage changes in mass, unconfined compressive strength and thermogravimetric characteristics were recorded. As a control, the same soil stabilised by LPC (lightweight Portland cement) was tested. The results indicated that soil stabilised with LAS had greater resistance to sulphate attack as measured by water absorption and strength. The results are shown in Figures 2.36 a and b.



(a)



(b)

Figure 2.36: UCS of stabilised soils at varying soaking time ((a) LAS; (b) LPC) (Jiang et al., 2018).

Shalabi et al. (Shalabi et al., 2017) explored the ability of by-product steel slag to improve clay soils' engineering properties, using laboratory and field experiments to determine what the effect would be of adding steel slag in various percentages on shear strength, CBR, compaction, compressibility, swelling and plasticity. The results showed that plasticity, cohesion intercept, swelling potential and soil dry density all fell with increases in the steel slag content, while the angle of internal friction increased. CBR increased as slag content increased, while UCS decreased, as shown in Figure 2.37 (curve A). When tested at maximum dry density and optimum water content, UCS as shown in Figure 2.7 first decreases as slag content increases but then shows a slight increase when slag content exceeds 15%.

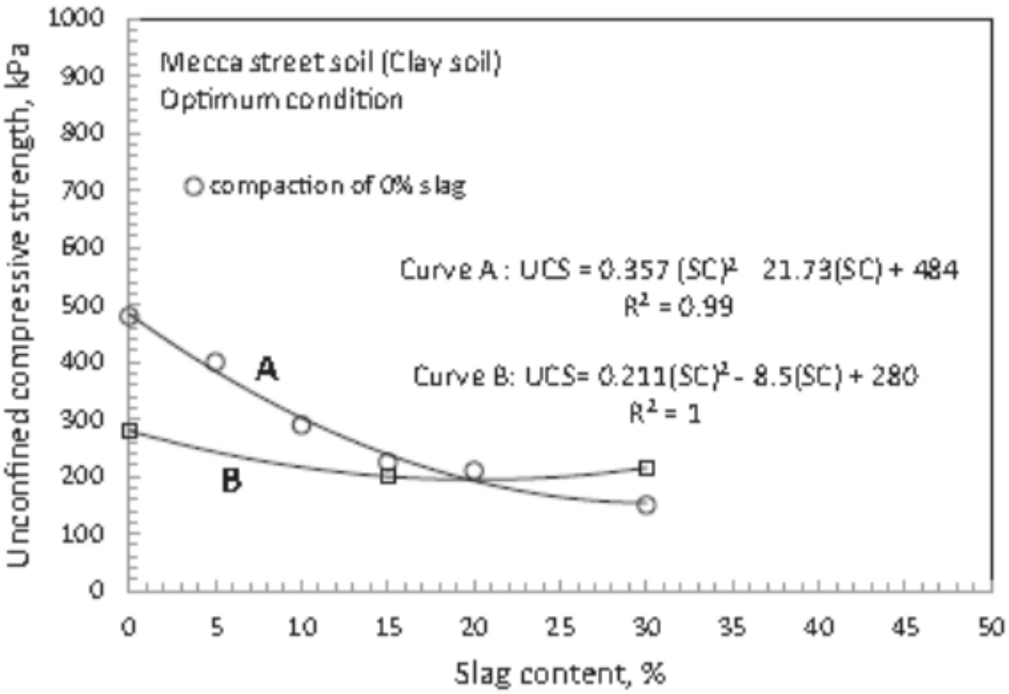


Figure 2.37: UCS of clay soil treated with steel slag content (Shalabi et al., 2017).

A lab scale batch test by Kim et al. (Kim et al., 2018) stabilised arsenic (As) for one hour in mine waste samples, with varying amounts of BOF (basic oxygen furnace) slag and distilled water being added. The stabilisation efficiency that resulted varied with such stabilising conditions as the ratio of BOF slag content and water to mine waste, but ranged between 75-92% and 92-95% for 5% (w-slag/w-mine waste) and 10% BOF slag treated mine waste samples, respectively. At 3% BOF slag treatment, the point of zero charge and the stabilising pH both indicated a negative charge on the BOF slag surface. On the basis of comparing fresh and Ca-reduced BOF slags, the mechanism for arsenic stabilisation was concluded to be adsorption through cation bridges by Ca^{2+}.

Goodarzi et al. (Goodarzi and Salimi, 2015) used two industrial by-product types, GBFS (granulated blast furnace slag) and BOFS (basic oxygen furnace slag) to study dispersive soil stabilisation's effectiveness. The additives were added to laboratory dispersed samples in a range of measures from 2.5% to 30% and experiments conducted to reveal the stabilised soil's mechanical, physiochemical, and microstructural changes. The conclusion was that it is possible to eliminate soil dispersion with a 10% addition of BOFS. This was concluded to be a result of exchanges by multivalent cations from the agent of interlayer sodium ions on the clay surfaces. It was also noticed that increasing the ion concentration in soil-additive mixtures results in increased depression of the diffuse double layer with a corresponding reduction in the soil's potential to disperse. The success of these treatments is due to cementitious compounds being formed by pozzolanic reactions and this is confirmed by XRD analysis and SEM micrographs. GBFS activity appears to be lower than BOFS, exerting less influence on the soil engineering parameters. The result is that a greater percentage (20–25%) of GBFS is needed to govern soil dispersion. Using the slags is shown to be very effective in overcoming problems of dispersive soils, and this is particularly true of BOFS. Increases in curing time respond to improvements in the strength of composite samples and this is illustrated in Figure 2.38, with BOFS outperforming GBFS.

Research by Manso et al. (Manso et al., 2013) into LFS (Ladle Furnace Slag) and its effect on a number of clay soils showed behaviour similar to that of soil and lime mixtures that have been reported in the literature. Table 2.1 shows such geotechnical properties of soils and mixtures as bearing capacity, durability, expansiveness, and the plasticity index.

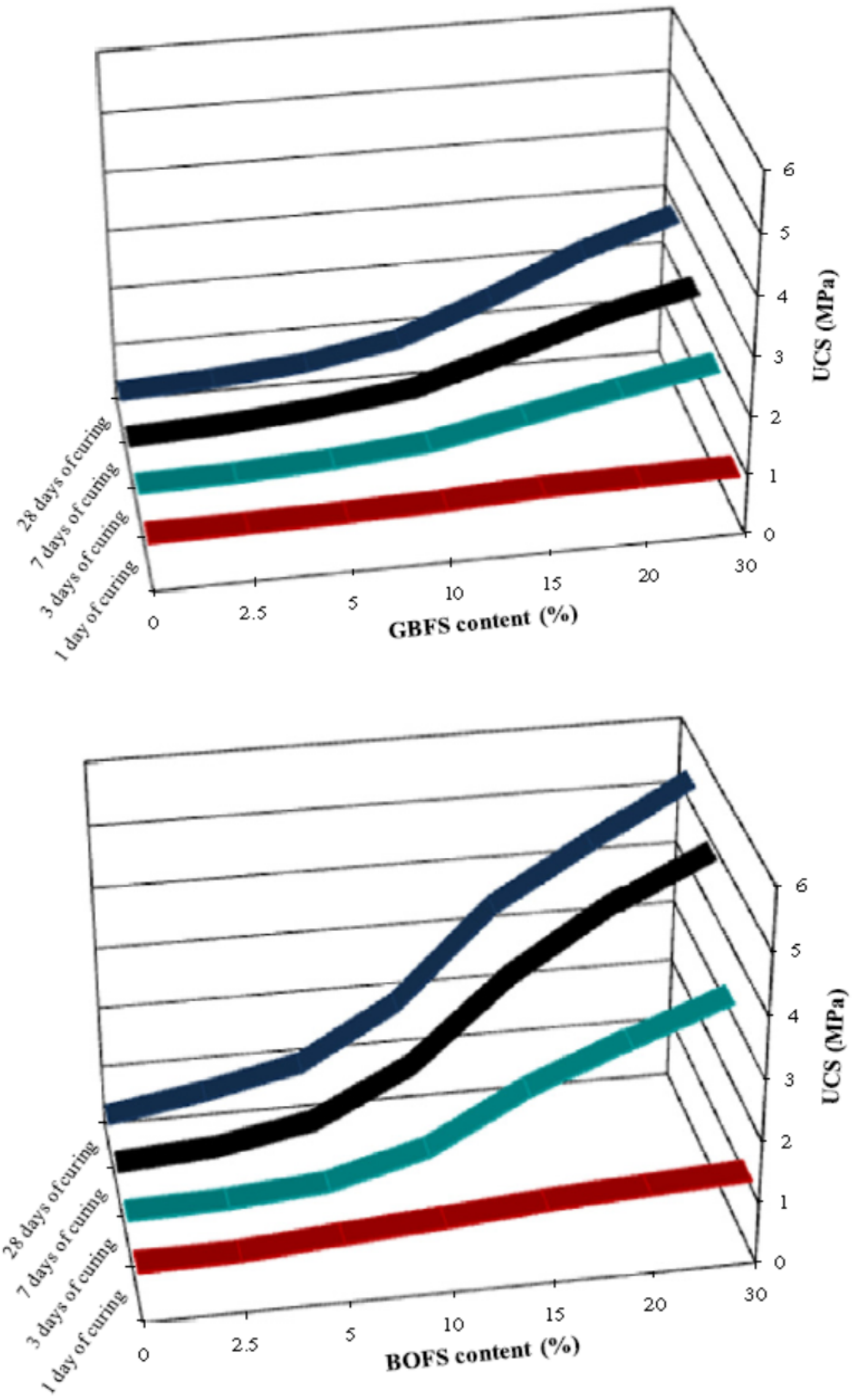


Figure 2.38: Effect of curing time and stabiliser content on the compressive strength of smectite samples treated with GBFS and BOFS (Goodarzi and Salimi, 2015).

Table 2.1: Geotechnical properties of the soils and their mixtures (Manso et al., 2013).

Mixtures	S1	S1C2	S1E5	S2	S2C2	S2E4	S3	S3C2	S3E5
Density PM (Mg/m3)	1.6	1.6	1.6	1.7	1.7	1.7	1.9	1.9	1.9
Humidity PM (%)	22.5	23.5	23.5	19.3	19.3	19.3	14.1	14.6	14.6
Absorption (%)	4.2	4	4.2	9.2	11	9.6	5.1	3.5	3.2
CBR Swelling (%)	3.4	0.7	0.8	10.2	9.7	9.9	6.2	2.5	2.6
CBR Index	2.8	58.6	52.6	1.4	4.3	4.4	2.3	18	13.2
Free swelling (%)	3.2	0.87	1.18	11.4	10.1	10.1	5.8	2.16	2.35
Collapse slump (%)	0.05	0	0.05	0.16	0.11	0.11	0	0.05	0.05
Atterberg limits (LL-LP)	68–25	54–33	56–29	85–29	85–33	87–42	52–21	52–32	57–26
Plasticity index (IP)	43	21	27	56	52	43	31	20	31
Strength at 28 days (kPa)	452	782	1096				740	998	1047
Expansion 7-days D–4792 (%)			0.1			1.8			0.3

- Soil S1 was mixed with 2% of lime (S1C2 mixture) and with 5% of LFS (S1E5).

- Soil S2 was mixed with 2% of lime (S2C2) and with 4% of LFS (S2E4).

- Soil S3 was mixed with 2% of lime (S3C2) and with 5% of LFS (S3E5).

A study by Sekhar et al. (C Sekhar and Nayak, 2018) into the use of GBFS and cement in manufacturing compressed stabilized earth blocks (CSEB) involved testing the index and strength properties of two soils available locally with added granulated blast furnace slag. The optimum replacement percentage was established and then, as shown in Figure 2.39, a range of percentages of cement were added to produce the blocks sized 305 mm x 143 mm x 105 mm. All blocks were cast to a target density and then cured for 28 days. The results showed that blocks in which cement and GBFS have been mixed are suitable for construction of load bearing walls. Energy consumption is reduced because the percentage of cement required to manufacture the blocks with an optimum GBFS content is small.

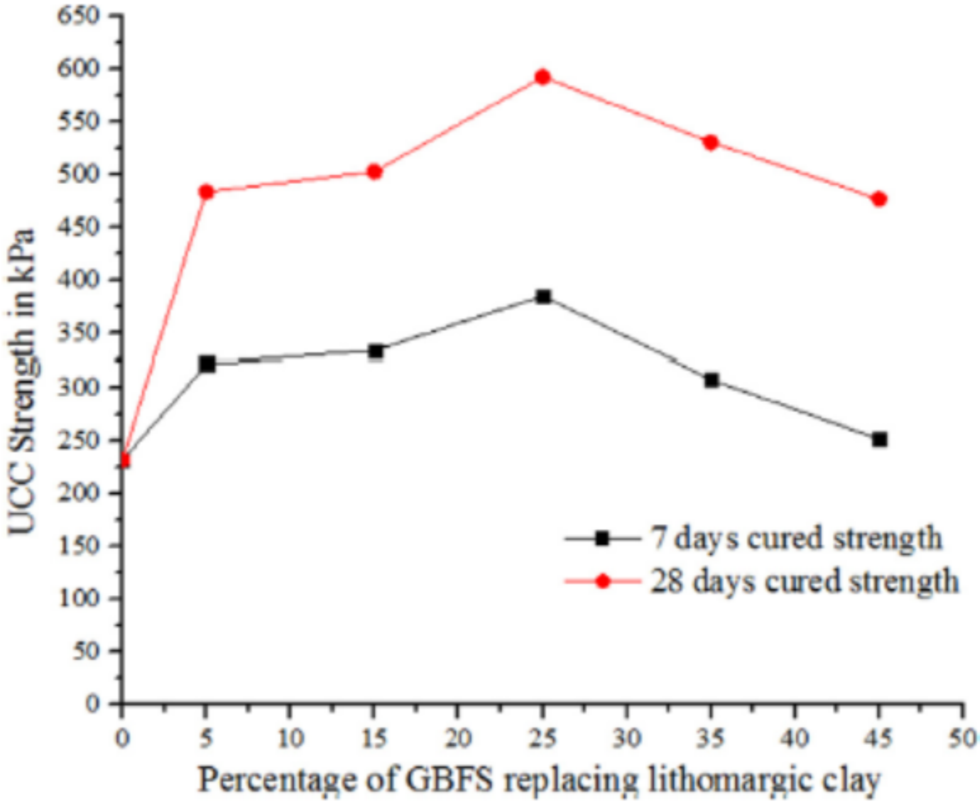


Figure 2.39: How UCS of lithomargic clay soil varies as the percentage GBFS replacement changes (C Sekhar and Nayak, 2018).

Zhang et al. (Zhang et al., 2016) described a study to determine the engineering and microstructural properties of a silt foundation soil stabilised by industrial by-products with a lignin base and demonstrated the potential of lignin to improve silt's engineering properties. This is promising as an environmentally friendly soil stabiliser. The engineering properties of silt stabilised with lignin are significantly influenced by lignin content and curing time, as is shown in Figure 2.40. Lignin's optimum content when used in foundation silt is about 12%. The precipitated cementitious material forms after the soil has been stabilised by lignin and then cured for a period. The presence of lignin transforms the stabilised silt from brittle to ductile. This study improves the understanding of the value of industrial by-products based on lignin as soil stabilisers in foundation construction.

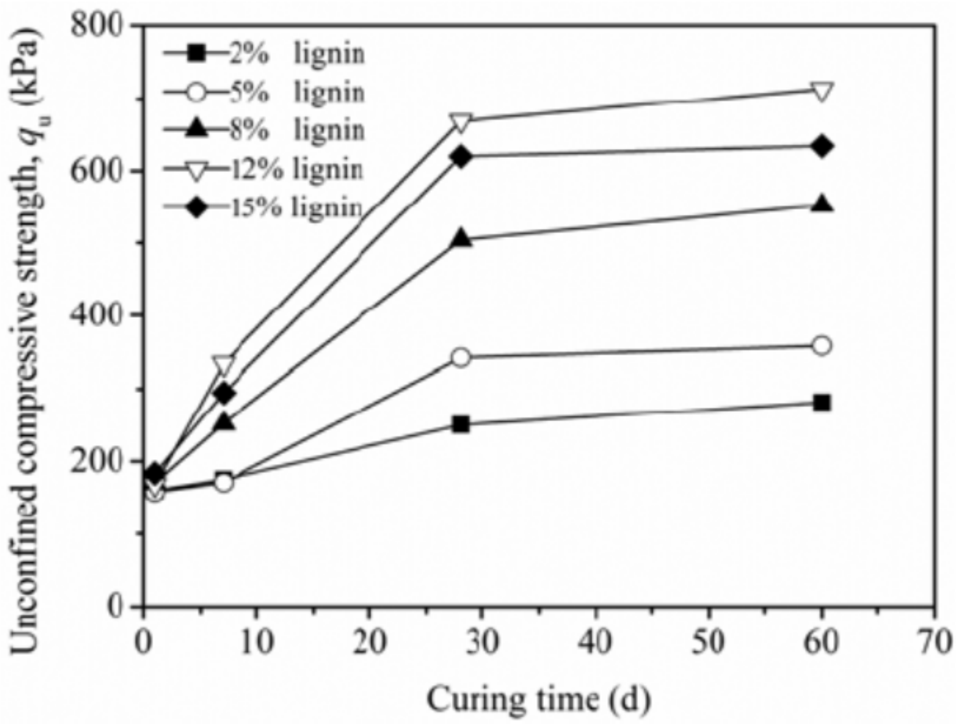


Figure 2.40: Variations during curing in UCS of silt treated with Lignin (Zhang et al., 2016).

A study by Vinod et al. (Vinod et al., 2010) investigated lignosulphonate-treated dispersive clay's resistance to erosion as well as advantages it has over a traditional cement admixture and showed that increases in lignosulphonate improve critical shear stress and soil erosion coefficient (see Figure 2.41). It is necessary to understand the mechanisms through which lignosulphonate and clay particles interact in order to assess treated soils' long-term environmental sustainability and this is presently not well understood at the microscopic level. Figure 3. Erosion rate against hydraulic shear stress for lignosulfonate treated and untreated dispersive clay. The lignosulphonate-treated soil's improved performance can be understood as a result of the reduction in double layer thickness when clay particles' surface charges neutralised and polymer bridging leads to more stable particle clusters being formed.

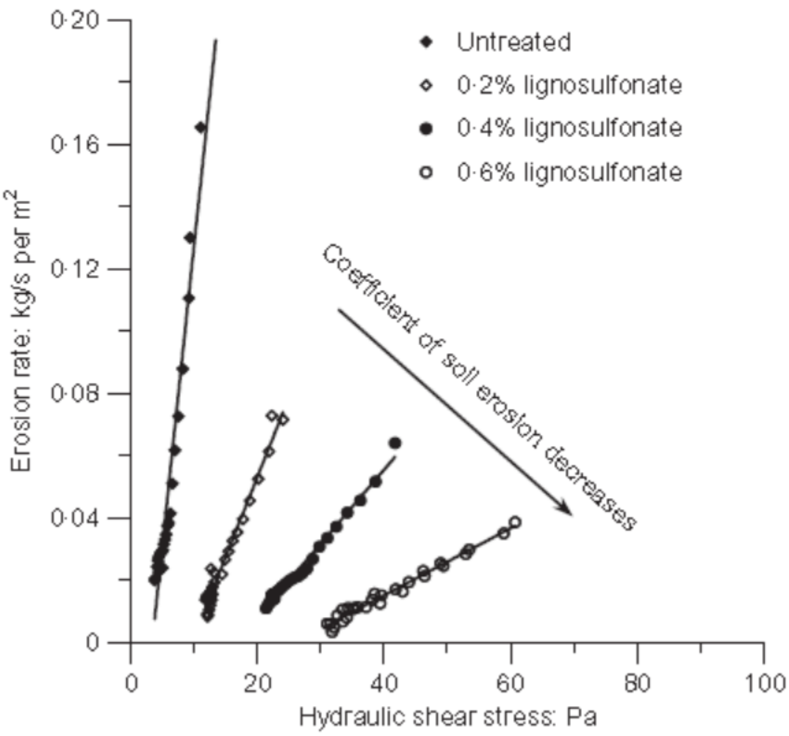


Figure 2.41: Erosion rate and hydraulic shear stress for dispersive clay both untreated and treated with lignosulphonate (Vinod et al., 2010).

CHAPTER 3

METHODOLOGY

3. Experimental Procedure

3.1. Materials used in this study

3.1.1. Soils

Two different types of soils are used in this study were obtained from Erbil city in Iraq. One is reddish in color from Shaqlawa district (S1), while the other being yellowish in color from Kore district (S2). Shaqlawa and Kore district soils were chosen due to supposed many strategic road projects planned to implement in these areas. The Physicochemical properties and Geotechnical features of soils were measured and illustrated in Tables 3.1 and Table 3.2, respectively. According to Skempton's (Skempton, 1953) activity classification, both soils are classified as ‘‘active." Because of plasticity index and activities were 29 and 1,31 for the soil type 1 whereas the results existed 32 and 1.33 for the soil type 2, respectively. Based on the Unified Soil Classification (USC) System (from ASTM D 2487 (ASTMD2487-17, West Conshohocken, PA, 2017)) depending of on the Atterberg limits which measured as per ASTM D4318 (ASTMD4318-17e1, West Conshohocken, PA, 2017), both soils are categorized as CH "Inorganic clays or high plasticity, fat clays." On the other hand, Fig 3.1 demonstrated the grain size distribution of soils using hydrometer test and sieve analysis with respect of ASTM D 422 (ASTMD422-63(2007)e2, West Conshohocken, PA, 2007).

Table 3.1: Physicochemical properties of clay soils tested.

Weight%	S1	S2	S	FA
CaO	12.3	25.05	27.5	6.3
SiO2	44.8	21.05	31.5	50.55
MgO	1.25	7.9	9.1	0.95
Al_2O_3	14.95	9.7	15.5	27.6
Fe_2O_3	6.9	3.35	6.1	4.4
K_2O+ Na_2O	3.3	1.95	1.1	1.2
TiO_2	0.85	0.4	2.75	0.59
SO_3 [a]	0.08	5.065	3.3	1.2
CO_2 [a]	4.5	3.9	1.35	1.3
Loss of ignition (%)	11	21.3	1.8	2.2
Soluble salts (%)	0.095	4.2165		
Organic material (%)	0.0915	0.845		
Soluble sulphates (%SO_4)	0.028	1.877		
Gypsum contents (%)	0.05	4.037		

[a] Calculated from the sulphates and carbonates contents.

Table 3.2: Geotechnical properties of the soils.

Physical property	S1	S2
Atterberg limits (%)		
LL	52	55
PL	23	23
PI	29	32
Average specific gravity of particles	2.4	2.5
Bulk density (Mg/m^3)	1.95	1.75
Optimal humidity (%)	14.3	19.4
Color	Reddish	Yellowish
Density (Mg/m^3)	1.8	1.6
Absorption (%)	5.5	9.3
Swelling (%)	5.9	9.8
CBR index	2.4	1.5

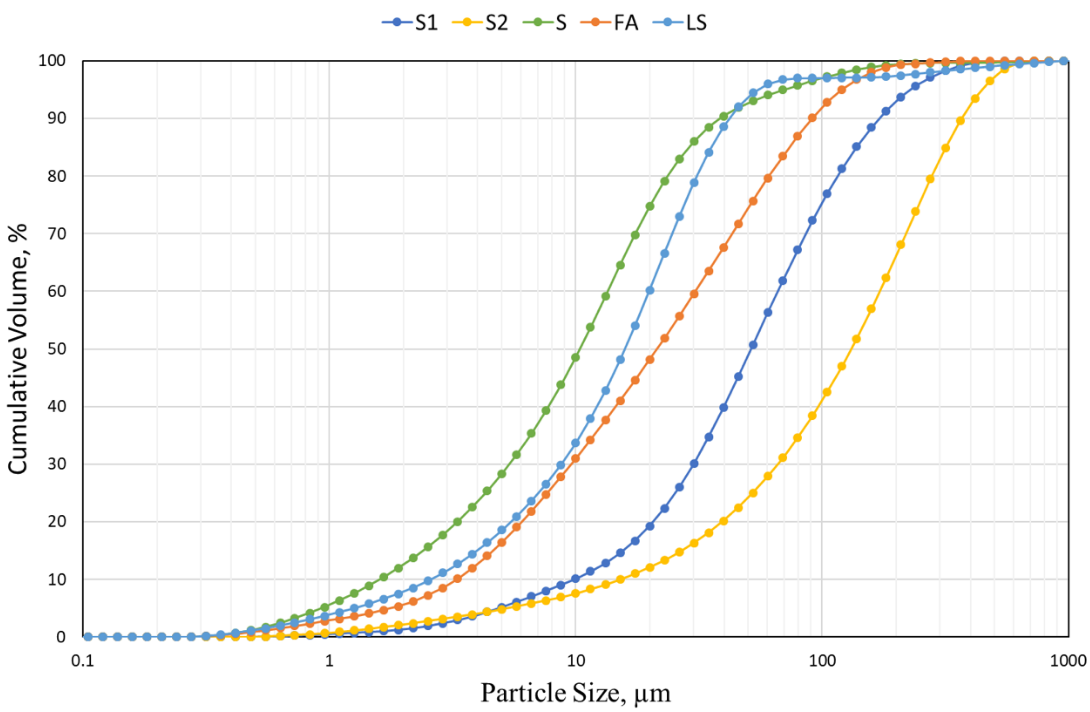


Figure 3.1: Grain size distribution of soils and admixtures.

3.1.2. Stabilizers

The admixtures were selected as admixture to stabilize soils in geotechnical engineering, they were used in partial replacement with soils by weight in different shape combinations of unary, binary, and ternary cementitious blends. Ground granulated blast furnace slag (S) with respect to ASTM C989-06 (ASTMC989-06, West Conshohocken, PA, 2006) obtained from the Iskenderun iron Production Factory, Iskenderun, Turkey. Class F fly ash according to ASTM C-618 12a (ASTMC618-12a, West Conshohocken, PA, 2012) was supplied from Çatalağzı Thermal Power Plant, Zonguldak, Turkey. Table 3.1 summarizes physical properties and chemical composition of S and FA. The specific surface area for S and FA measured using a Brunauer-Emmett-Teller (BET) method with a value of 420 m^2/ Kg and 190 m^2/ Kg, respectively, that were near to the studies of Jiang et al. (Jiang et al., 2018) and Nagendra et al. (Nagendra et al., 2016). Fineness of the S and FA were measured by Blaine method and Loss on ignition (LOI) by calculating the weight loss upon heating. On the other hand, Fig 1 shown the grain size distribution of stabilized (soils) and stabilizers such as S, FA, and LS. In contrast, the used water-soluble Lignosulfonate (LS) is a processed waste by-product from a paper mill, which demonstrated a yellow brown powder color with a smell of fragrance. LS is composed of sulfur (S), sodium (Na), carbon (C), and oxygen (O) that also mentioned by Zhang et al. (Zhang et al., 2017). LS is a polymer composite with lignin-base, which contains several hydrophilic groups includes phenylic hydroxyl besides sulfonate and alcoholic hydroxyl (Ta'negonbadi and Noorzad, 2017; Chen and Indraratna, 2014).

3.2. Mixture preparations

Forty-four mixtures with two soils types (two groups) and various conditions (unary, binary, and ternary) were prepared to observe the effect of binders at different proportions illustrated in Table 3.3. The amount of binders, soil types, and replacement conditions were selected as three important variables. Reference mixtures were produced which any binders were not added for control purposes. The natural collected soils were air dried then broken down to particle size that could pass a sieve 2 mm for conducting tests. The designed amount of water and stabilizers were considered by dry weight of soils. The additive contents were set as 6%, 8%, and 10% for S, FA, and LS. The optimum moisture content and maximum dry density of the stabilized soils were found for finding amount of water used. Each stabilizers and remained amount of air dried soils were carefully mixed to prepare for testing then water was added to the mixture till the moisture content reaches optimum moisture content and they were mixed again to certain homogeneity. Subsequently, the mixtures were poured into cylindrical molds (Φ50×100 mm) and compacted with a hydraulic jack. The soil specimens extruded from the molds carefully using a hydraulic jack. They were then placed in air-tight plastic bags and cured in an ambient conditions controlled room at temperature 22 ± 2 °C and relative humidity of 95 ± 3%. At the end of 28 days curing period, three were tested to investigate the effects of stabilizers on the engineering properties of the soil sample. In the laboratory, usually tests done for 28-days as a curing time because of test results of most researchers optimized at this period, whereas 7 days may be selected in the site projects for the save of time (Gesoğlu et al., 2009).

Table 3.3: Mixture proportions for mixtures with respect to the soils types and shapes of replacements

#	Code	% S (by dry weight of soil)	%FA (by dry weight of soil)	% Lignin (by dry weight of soil)	Total binder %	Soil type
1	S1	0	0	0	0	S1
2	S6S1	6	0	0	6	S1
3	S8S1	8	0	0	8	S1
4	S10S1	10	0	0	10	S1
5	F6S1	0	6	0	6	S1
6	F8S1	0	8	0	8	S1
7	F10S1	0	10	0	10	S1
8	L6S1	0	0	6	6	S1
9	L8S1	0	0	8	8	S1
10	L10S1	0	0	10	10	S1
11	SF6S1	3	3	0	6	S1
12	SF8S1	4	4	0	8	S1
13	SF10S1	5	5	0	10	S1
14	SL6S1	3	0	3	6	S1
15	SL8S1	4	0	4	8	S1
16	SL10S1	5	0	5	10	S1

17	FL6S1	0	3	3	6	S1
18	FL8S1	0	4	4	8	S1
19	FL10S1	0	5	5	10	S1
20	SFL6S1	2	2	2	6	S1
21	SFL8S1	8/3	8/3	8/3	8	S1
22	SFL10S1	10/3	10/3	10/3	10	S1
23	S2	0	0	0	0	S2
24	S6S2	6	0	0	6	S2
25	S8S2	8	0	0	8	S2
26	S10S2	10	0	0	10	S2
27	F6S2	0	6	0	6	S2
28	F8S2	0	8	0	8	S2
29	F10S2	0	10	0	10	S2
30	L6S2	0	0	6	6	S2
31	L8S2	0	0	8	8	S2
32	L10S2	0	0	10	10	S2
33	SF6S2	3	3	0	6	S2
34	SF8S2	4	4	0	8	S2
35	SF10S2	5	5	0	10	S2
36	SL6S2	3	0	3	6	S2
37	SL8S2	4	0	4	8	S2
38	SL10S2	5	0	5	10	S2
39	FL6S2	0	3	3	6	S2
40	FL8S2	0	4	4	8	S2
41	FL10S2	0	5	5	10	S2
42	SFL6S2	2	2	2	6	S2
43	SFL8S2	8/3	8/3	8/3	8	S2
44	SFL10S2	10/3	10/3	10/3	10	S2

3.3. Test methods

The procedures of the tests such as California Bearing Ratio, Unconfined Compressive strength, Atterberg limits, and Permeability were conducted based on the standards of

ASTM D1883 (ASTMD1883-16, West Conshohocken, PA, 2016), ASTM D5102 (ASTMD5102-09, West Conshohocken, PA, 2009), ASTM D4318 (ASTMD4318-17e1, West Conshohocken, PA, 2017), and ASTM D2434-68 (ASTMD2434-68(2006), West Conshohocken, PA, 2006), respectively.

3.3.1. California Bearing Ratio

This empirical test indicates a soil's shear strength. Its value lies in the ease with which it can be carried out and the fact that, thanks to its use worldwide, a very large amount of data exists to help interpret results. While it may occasionally be carried out on site, it is normally run in the laboratory.

Samples for the CBR test can be undisturbed or remoulded and the test involves penetrating a pavement material with a 50 millimetres diameter cylindrical plunger at a speed of 1.25 millimetres per minute, recording the loads at 2.5 millimetres and five millimetres. Expressing the load as a percentage of a standard value at a particular level of deformation establishes the CBR value. The sample is passed through a 20 millimetre IS sieve. Five kilograms are then mixed with a volume of water such that the sample reaches the optimum or field moisture content and the water and soil are thoroughly mixed. A spacer disk is placed on the base plate at the bottom of the mould and covered with a coarse filter paper. The soil and water mix is divided into five equal portions, the mould is cleaned and oiled and then one fifth of the mould is filled with the soil sample. The soil layer is compacted with 56 blows distributed evenly using a hammer that weighs 4.89 kilograms. The compacted soil is scratched on the top surface before the process is repeated with the second layer. After the third layer has been added, a collar is attached to the mould and the process repeated. The collar is removed after the fifth layer and excess soil is removed. The baseplate is then removed and the mould inverted before being clamped to the baseplate and weights amounting to 2.5 kilograms placed on its top surface. The mould containing the specimen is positioned on the test machine and the plunger applied to the soil under a four-kilogram load to establish contact between plunger and soil. At this point, the dials are set to 0 and then a sufficient load is applied to achieve the penetration rate of 1.25 millimetres per minute. A record is taken of the load at the penetrations 0.5, 1, 1.5, 2, 2.5, 3, 4, 5, 7.5, 10 and 12.5 millimetres.

3.3.2. Unconfined Compressive Strength

This test involves testing a soil cylinder with no lateral support through simple compression at a constant strain until it fails. The compressive load per unit of area at which the specimen failed is noted. This load is the soil's unconfined compressive strength.

Procedure of Unconfined Compressive Strength test are as below,

1. When the sample is at the design density and water content, it is placed in the large mould.
2. The sampling tube is pushed into the mould and removed full of soil. When an undisturbed sample is required, the sampling tube should be pushed into the clay sample.
3. The soil sample is saturated in the tube by any suitable method.
4. The split mould is coated lightly with a thin layer of grease and the mould is weighed.
5. The sample is extruded from the tube into the split mould by means of a knife and a sample extractor.
6. Both ends of the specimen are trimmed in the split mould and the mould is weighed with the specimen.
7. The split mould is separated into two and the specimen removed.
8. Vernier calipers are used to measure the specimen's diameter and length.
9. The specimen is placed on the compression machine's bottom plate and the upper plate is adjusted so that it is in contact with the specimen.
10. The dial gauge and proving ring gauge are set to 0.

11. The compression load is applied such as to cause axial strain of ½ to 2% per minute.
12. The readings on the dial gauge and the proving ring are recorded every 30 seconds until the strain has reached 6%, after which readings may be taken every 60 seconds until the strain reaches 12% and every two minutes thereafter.
13. The test is continued until an axial strain of 20% has been reached or until the sample shows clear failure surfaces.
14. If it can be done, the angle between the failure surface and the horizontal is measured.
15. A sample is taken from the specimen's failure zone in order for its water content to be determined.

3.3.3. Atterberg limits

The stability shown by a soil varies according to its water content and is known by the term "consistency." It specifies the state of a soil that has been remoulded and is cohesive. The range of possible states is as follows:

(dry) solid state → semi-solid state → plastic state → liquid state (wet)

The purpose of these limits is the production of indices such as the consistency index and the plasticity index which are used to characterise soils mechanically. Before liquid and plastic limits can be ascertained, the soil sample must pass through a 0.425 millimetre sieve. The liquid limit should be tested for first. It can be difficult to control the water content of a soil with a coarse texture in order to carry out the liquid limit test and such a soil may not have a plastic limit at all.

3.3.3.1. Liquid limit

The liquid limit is the level of water content at which a soil's state turns from plastic to viscous, or liquid, and is measured by use of the Casagrande cup. It amounts to finding the water content corresponding to the number of drops (25) required to close a 13 millimetre section of a groove that has been cut in the soil sample:

1. Put the moistened sample in the Casagrande cup.
2. Cut a "V"-shaped groove in the sample by means of a standardised tool.

3. Raise the cup and drop it from a 10 millimetre height at about two drops per second.
4. Stop when a 13 millimetre section of the bottom of the groove has closed.
5. Record the number of drops that were needed to close the groove. The number must be more than 15 and less than 35 and the sample's water content should be progressively adjusted until this is so.
6. Determine the water content gravimetrically by drying the soil sample in the oven at 105°C for 18 to 24 h.

It is usual to carry out this process three times and take the average value as the liquid debit.

3.3.3.2. Plastic limit

The plastic limit is the soil water content as the semisolid and plastic or flexible states converge and is the gravimetric water content at which it is possible to roll a soil sample by hand into a 3.2 millimetre diameter thread without breaking it.

1. The sieved soil sample is moistened until it can be rolled easily between the hands.
2. The sample is rolled in the hands into an ellipsoid shape and then placed on a smooth, hard surface and rolled into a thread with the fingers or the palms to produce a thread in which the diameter is 3.2 millimetres.
3. When it has reached the desired diameter, the thread is broken into pieces and kneaded into another ellipsoid before being rolled once more into a thread.
4. This process is repeated continually until the pressure causes the thread to crumble to the point where it cannot be reshaped into a 3.2 millimetre diameter thread. The crumbled thread is collected to find its water content.
5. The soil sample is dried in the oven at 105°C for 18 to 24h for a gravimetric determination of the water content.

3.3.4. Permeability

Permeability tests are done in two ways, with a falling head test applied to soils with fine grains and a constant head test for soils with coarse grains.

Procedure:

1. The sample is compacted in the Permeameter's lower chamber in layers of approximately 1.5 centimetres deep until they are within about two centimetres of the rim of the lower chamber. The sample is compacted to a desired density by tamping with a suitable tool.
2. The tie rods are removed from the upper section of the chamber and the upper porous stone is placed on the specimen, and a spring used to secure the chamber's upper section to the unit. The length of the specimen is measured and recorded.
3. The clamp is used to attach the falling head burette to the support rod. The burette is positioned as high as is practicable and the metre stick is placed directly behind the burette to allow measurement of the water's height in the burette above the chamber outflow port. The specimen is saturated as set out above.
4. The heights of the two levels from the outflow level are measured.

CHAPTER 4

4.1. RESULTS AND DISCUSSION ON EXPERIMENTAL STUDY

4.1.1. California Bearing Ratio (CBR) test

4.1.1.1. California Bearing Ratio (CBR) test results

CBR (The California Bearing Ratio test) was developed in California as a way of evaluating and classifying soil subgrades and materials for the base course of a flexible pavement. It measures a material's resistance to penetration by a standard plunger under conditions of controlled moisture and density.

Effects of using unary, binary, and ternary admixtures on the CBR of the two soil types at 28 days are shown in Figure 4.1, and Figure 4.2, respectively. Excitingly three distinct scenarios are observed: Firstly, CBR results of the soils increased up to of 8% mineral admixtures content beyond which results began to decrease, irrespective of the shape of replacement. The decrease of the results at 10% of replacement is may be due to agglomeration of disposal material particles (Gesoglu et al., 2016). Secondly, the ternary replacement had higher CBR results than binary then unary respectively, irrespective to the soil types. Actually, among the different replacement levels, the CBR value with 8% of the mixture SFL8 had highest results of 6.3% and 4.4% for the first and second groups, respectively. However, the highest improvement of CBR with ternary replacement than the others could be attributed to the regularly distribution of different mineral admixtures from micro to millimeter (see Figure4.1) that made the mixture more homogeneous. Additionally, mixing some mineral admixtures having different chemical composition (see Table 3.1) were made the soil particles to withstand a higher force because of completing the demand of most pozzolanic and high pozzolanic reactions are the reason for the increase in the CBR results. Finally, the higher CBR of the soil type 1 exceeded those of soil type 2 results, owing to the fact of superior general properties of the former than later as seen in Tables 3.1 and 3.2. Nevertheless, the rate of increment of the soil2 better than that of soil1, as noticed in Figure 4.3, this may be considered as the properties of soil2 is more responsive with the mineral admixtures than soil1.

Many other researchers revealed similar trends on the effects of pozzolanic materials on the CBR values of the clay soils (Garzón et al., 2015; Senol et al., 2006; Ta'negonbadi and Noorzad, 2017). For instance, Rahgozar et al. (Rahgozar et al., 2018) improved the CBR value from 19% to 50% when they increased the amount of pozzolanic materials from 2% to 8%. In the study of Zhang et al. (Zhang et al., 2017) the CBR values of silty soils enhanced by 70.6% and 87.7% through replacing lignin at 8% and 12%, respectively, compared to the reference mixture.

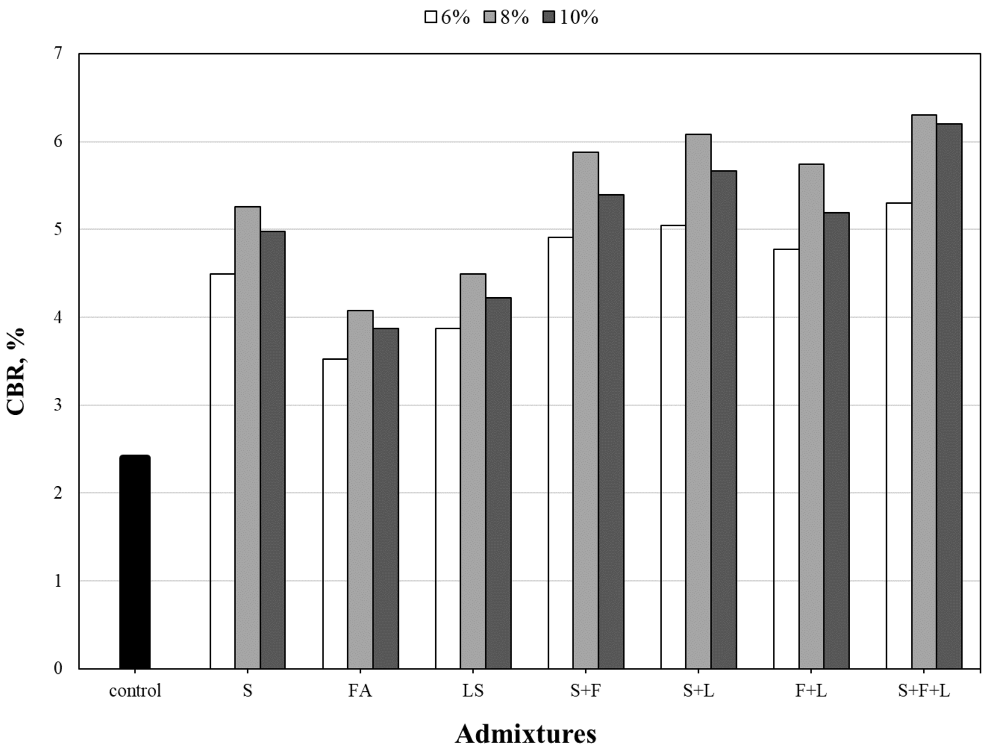


Figure 4.1: CBR test results for stabilized soil type 1.

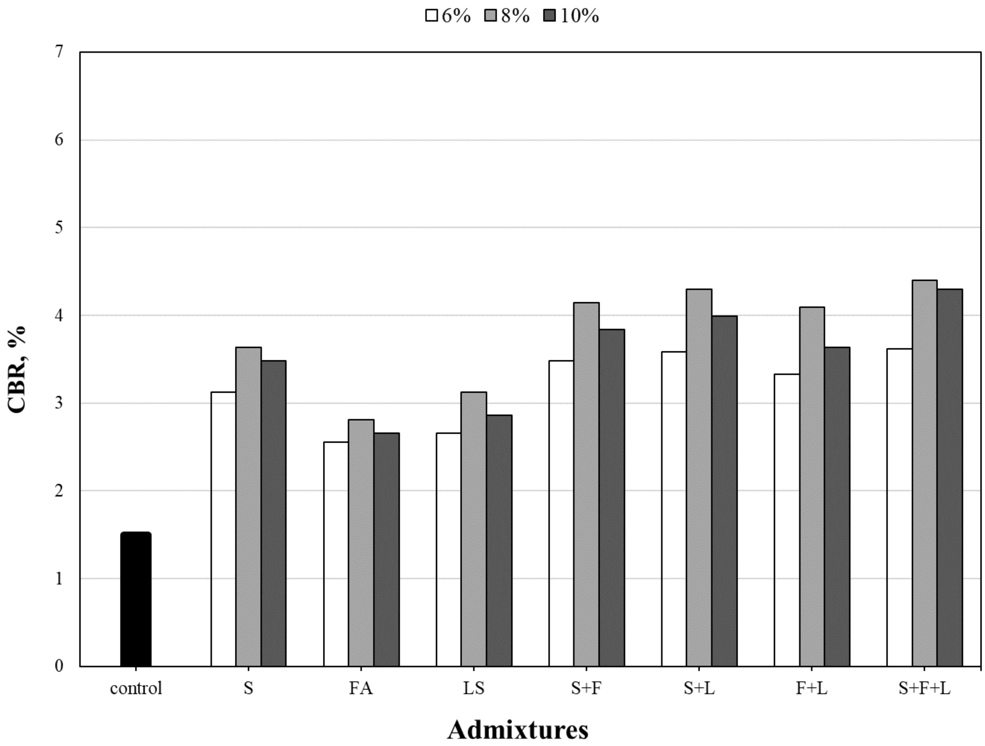


Figure 4.2: CBR test results for stabilized soil type 2.

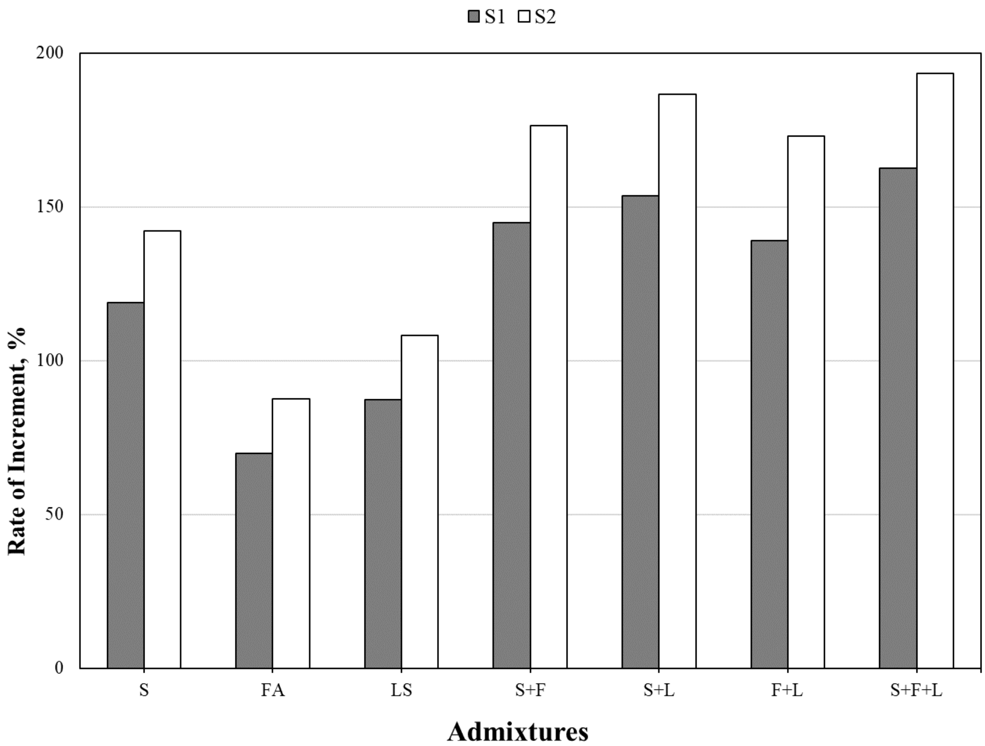


Figure 4.3: The rate of increment of CBR test with respect to soils 1 & 2 and pozzolanic materials

4.1.1.2. Application of California Bearing Ratio (CBR) test results

The California State Highway Department developed CBR (California bearing ratio) to determine subgrade soil properties for flexible pavement design. CBR was subsequently adopted by The Road Research Laboratory in the UK. Design curves for a range of wheel loads combined with CBR values indicated the required thickness. This approach has been taken up by the Indian Roads Congress (IRC) which has produced design charts matching construction depth to CBR values taking into account the traffic classification which represents daily commercial vehicle traffic.

Figs. 4.4 and 4.5 show, respectively, the total pavement thickness needed for subgrade CBR values ranging from 2% to 10% for the range of traffic volumes 1 to 10 million standard axles (msa) and 10 to 150 msa taken from charts produced by the IRC (Congress, 2012).

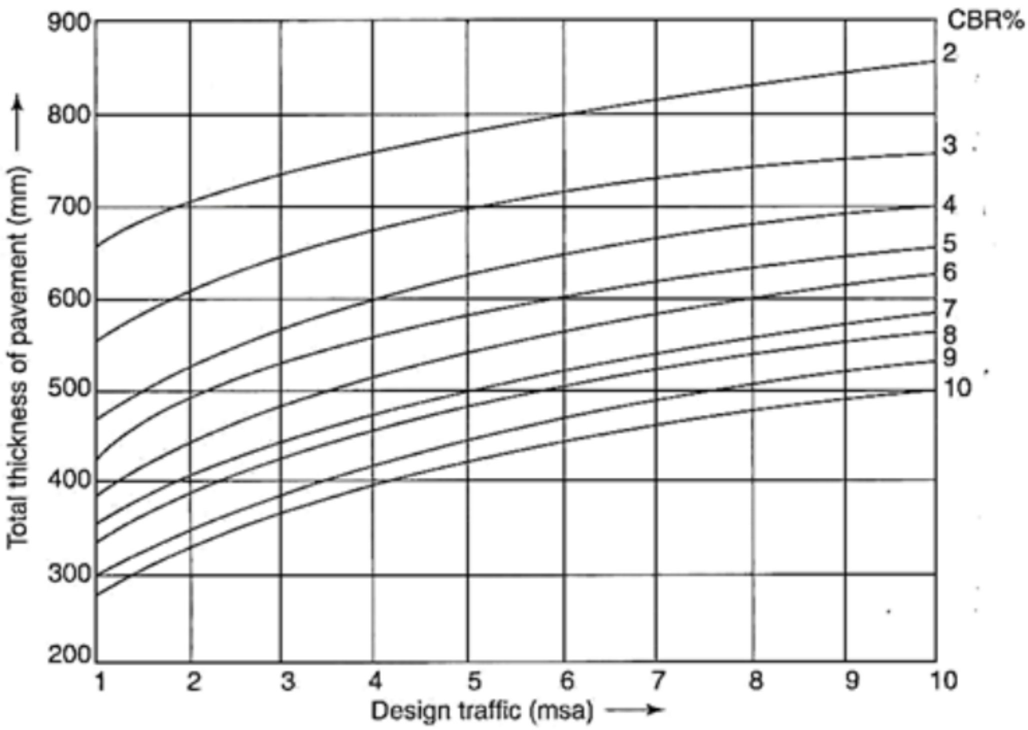


Figure 4.4: Pavement thickness design chart for traffic 1 to 10 msa (Congress, 2012)

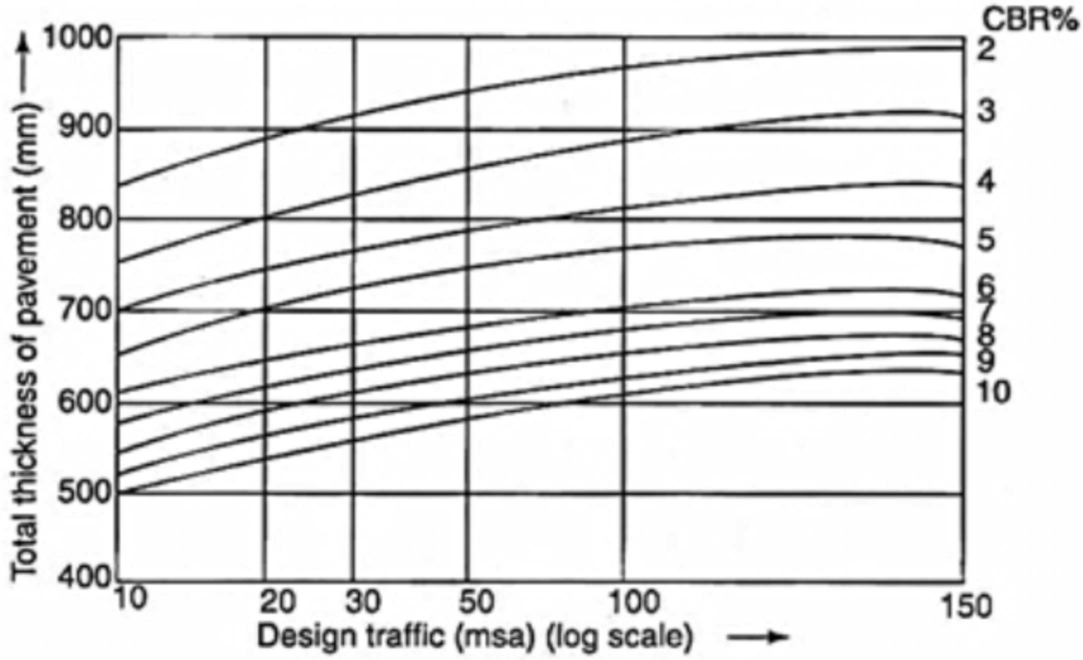


Figure 4.5: Pavement thickness design chart for traffic 10 to 150 msa (Congress, 2012)

The pavement design tables in Tables 4.1 and 4.2 show, respectively comprise the pavement composition and total thickness recommended for 1 to 10 msa traffic range for the control mixture and SFL8 soil type 1 mixture for all subgrade CBR values of 2% and 6% (Congress, 2012). There was a decrease in all road depth layers when soils with low CBR values were stabilised with pozzolanic materials that would otherwise have been an environmental headache and the potential economic savings are large.

Table 4.1: Recommended pavement composition for 2 % CBR

cumulative traffic (msa)	Total pavement thickness (mm)	Pavement composition			
		Bituminous surfacing		Granular base (mm)	Granular sub-base (mm)
		Wearing course (mm)	Wearing course (mm)		
1	665	20PC		220	430
2	710	20PC	50BM	220	445
3	755	20PC	60BM	255	445
5	790	25SDBC	70BM	255	455
10	855	40BC	100BM	255	465

Table 4.2: Recommended pavement composition for 6 % CBR

cumulative traffic (msa)	Total pavement thickness (mm)	Pavement composition			
		Bituminous surfacing		Granular base (mm)	Granular sub-base (mm)
		Wearing course (mm)	Wearing course (mm)		
1	395	20PC		220	170
2	455	20PC	50BM	220	180
3	495	20PC	50BM	255	195
5	540	25SDBC	50BM	255	215
10	620	40BC	65BM	255	265

Legend:
SD-Surface dressing
MS-Mix seal
BC-Bituminous concrete
DBM-Dense bituminous macadam
PC-Premix carpet
SDC-Semi-dense carpet
BM-Bituminous macadam
SDBC-Semi-dense Bituminous carpet

Economically, improved CBR value from 2% to 6% for the soil (S2), for the implementation of a road project with a length of 10 km and a width of 7 meter that planned to be designed for 5 million standard axels are illustrated in Table 4.3.

Table 4.3: CBR value economic analyses

CBR Value (%)	Thickness of Granular Subbase (mm)	No. of layer *	USD/m2	Total m^2 of 10km road	Total cost ($)
2	455	2	10	70,000	1,400,000
6	215	1	10	70,000	700,000

*Generally each sub-base layer is consisting of 25 cm and each layer costs around 10 $/m

From the above mentioned table, its concluded that the economic benefit is 700,000 USD due to incorporating of %8 ternary effect of admixtures (S+FA+LS) through stabilization process.

4.1.2. Unconfined Compressive Strength (UCS)

Results of the test are available immediately and give an approximation of the remoulded soil's compressive strength. It is performed within a time limit that prevents water draining out of or into the tested specimen.

Effects of S, FA, and LS on the unconfined compressive strength (UCS) of soil1and soil2 are presented in Figure 4.6, and Figure 4.7, correspondingly. Besides, Figure 4.8 demonstrated the percent increase in the 28-day UCS of the soils for 8% replacements of disposal materials. It was observed that the UCS of the soils continuously increased up to 8% of replacement mineral admixtures for unary and binary mixtures, regardless of the disposal types. Nevertheless, at ternary replacements, the growth of strength from 8% to 10% seemed to be quite limited or near to each other. Indeed, soil 1 with 8% of the ternary mixture of S+FA+LS had the highest UCS enhancements of 115% comparing with their reference mixture (see Fig. 4.8). Thus, ternary replacements seemed to be preferable than binary then individually. Such phenomenon was probably due to compensation of most admixtures through a ternary mixture of disposal materials that necessary to complete chemical reactions. On the other hand, at binary replacements of both soils, the most favorable replacements were related to mixture of SL8 followed by SF8 and FL8, respectively. In contrast, S was improved unconfined compressive strength than correspondingly replacing of LS and FA, regardless of the soil types. Consequently, the test results suggested replacement orders of S+FA+LS, S+LS, and S for the ternary, binary and unary, respectively, considering their beneficial effects on the unconfined compressive strength.

Effecting of replacing mineral admixtures on the unconfined compressive strength was considered by many researchers. For example, Zhang et al. (Zhang et al., 2018) investigated the stabilization of silty soil by different percentages of lignin. They enhanced the UCS of silty soils from 95kPa to a value of 280kPa, 310kPa, and 500kPa through replacing lignin by 2%, 5%, and 8% at 28 days of curing.

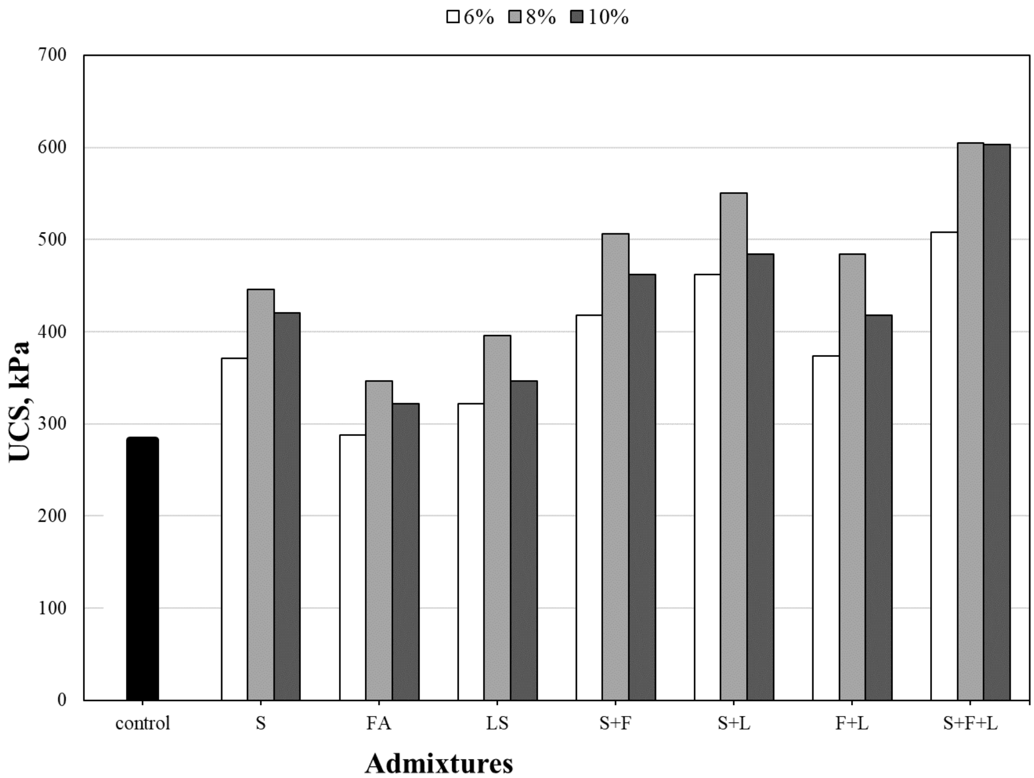


Figure 4.6: UCS test results for stabilized soil type 1

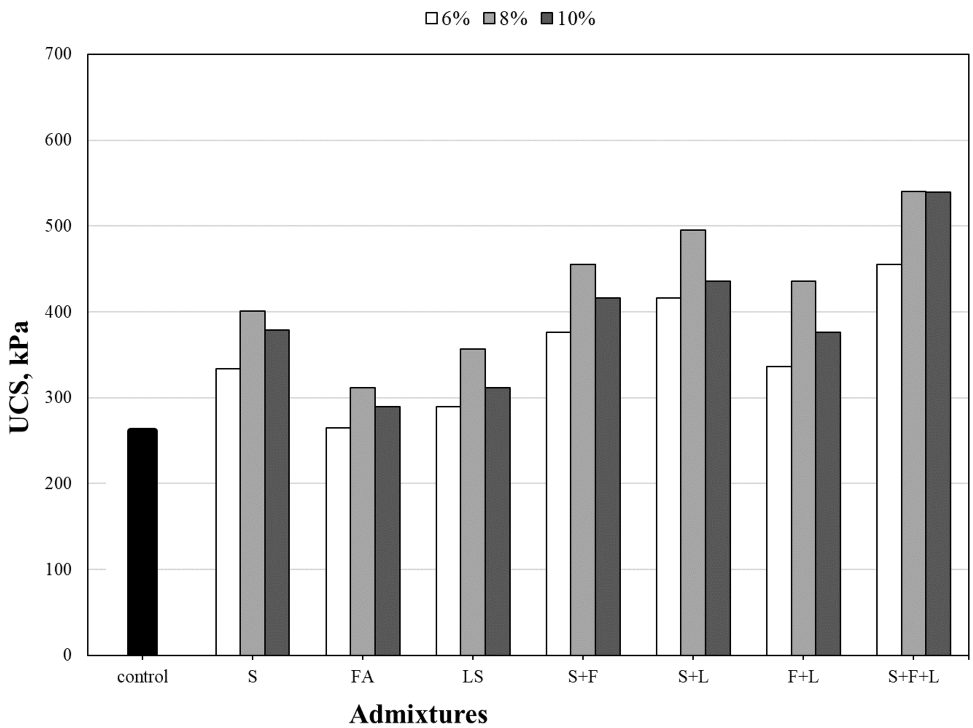


Figure 4.7: UCS test results for stabilized soil type 2.

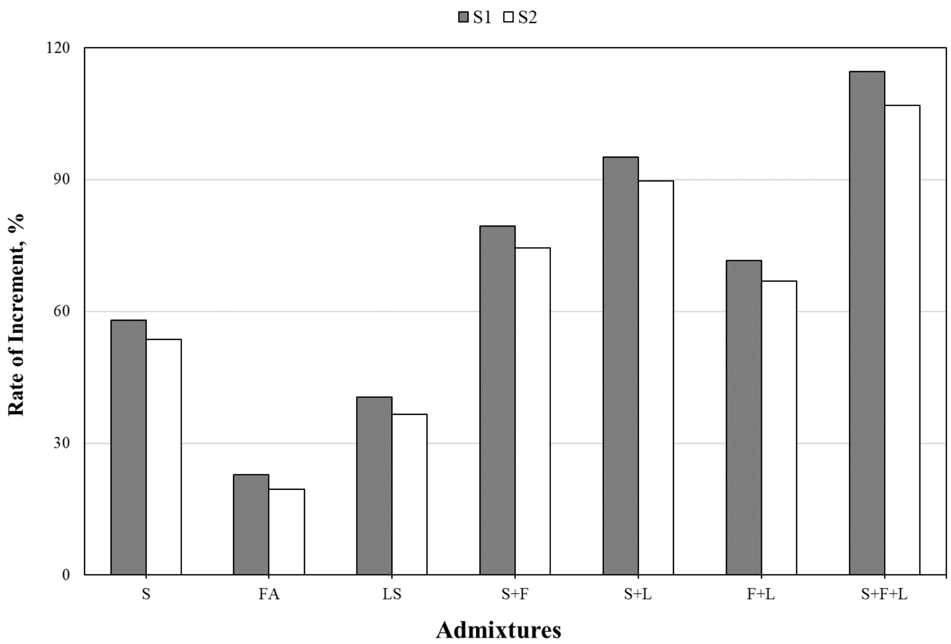


Figure 4.8: The rate of increment of UCS test with respect to soils 1 & 2 and pozzolanic materials

4.1.3. Atterberg limits tests

Early in the twentieth century, Albert Atterberg, a Swedish chemist, developed a system of classification to measure consistency based on the water content at specified points of transition between soils of different consistency. The transitions, which are known as Atterberg limits, are the shrinkage limit, the plastic limit, and the liquid limit and of these, the plastic and liquid limits are the ones most frequently used. The limits' values depend on a number of soil parameters including the size of particles and the specific surface area of particles able to attract molecules of water.

Figure 4.9 and Figure 4.10 demonstrated correlation equations of Atterberg limit tests of the soil 1 and soil 2, respectively, which replaced by unary, binary, ternary of S, FA, and LS. As it can be seen from Figure 4.9 and Figure 4.10, addition of mineral admixtures to the soils decreases the LL and PI with keeping unchanged of plastic limits (PL). Indeed, dropping in LL and PI of the stabilized soils is predictable because of non-plastic nature of the S, FA, and LS particles. Thus, it is significant to notice that the added mineral admixtures transformed the soil classification from highly plastic to low plastic clay. In addition, from above-mentioned figures, better performance of the ternary replacement than binary then unary have been noticed. Specifically, comparing to non-stabilized soils, 10% of S+F+L directed to a reduction of LL by 79% and PI by 112% for soil 1, whereas, the declined registered for LL by 62% and for PI by 175%

for soil 2. Changing of atterberg limits due to pozzolanic replacements can be attributed to the adjustment of the flocculation and matrixes of soil minerals that effect on soil liquidity and plasticity. The ternary or binary makes a denser and stronger soils by decreasing the voids existing in the mixture due to different material properties working at the same time to enhance the atterberg of soils.

Decreasing plasticity of soils via disposal materials was reported by the other researchers like Shalabi et al. (Shalabi et al., 2017). They decreased LL and PI by 15% and 9% through replacing 30% of steel slag, respectively. On the other hand, in the study of Ta'negonbadi and Noorzad (Ta'negonbadi and Noorzad, 2017), the atterberg of soils were reduced by 11% for LL and 9% for PI when 4% of LS added to untreated soils. Furthermore, Alazigha (Alazigha et al., 2016) treated the swilling of the soil from 6% to about 4.6% with adding 2% of LS.

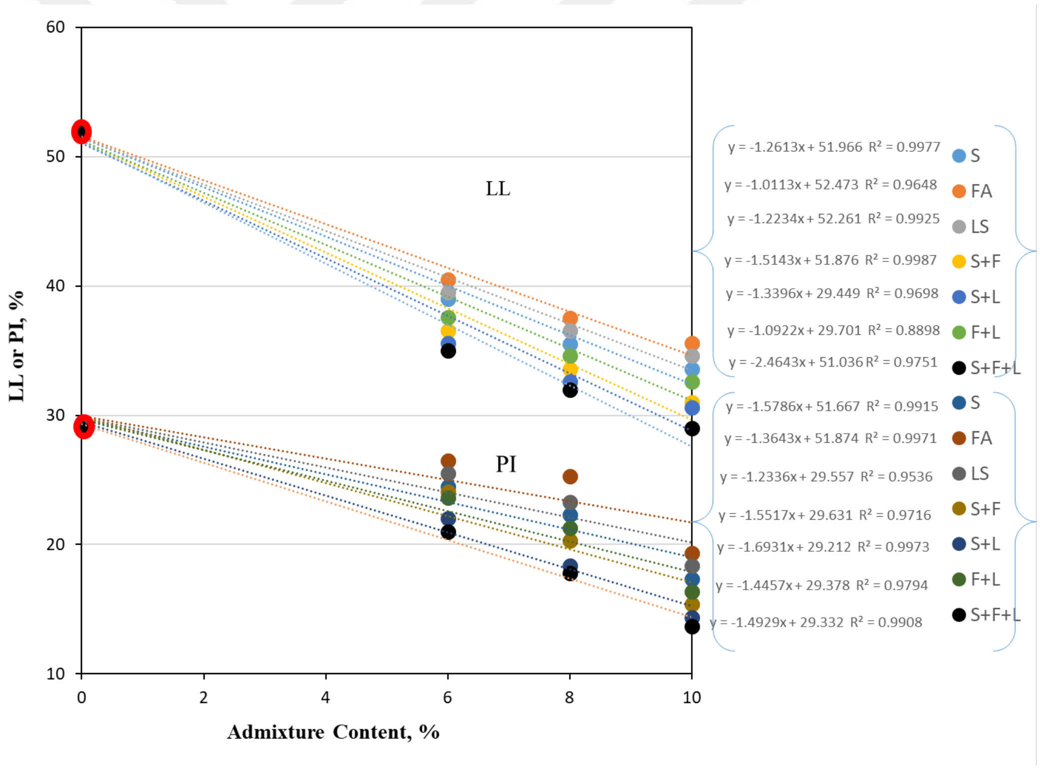


Figure 4.9: Atterberg limittest results for stabilized soil type 1

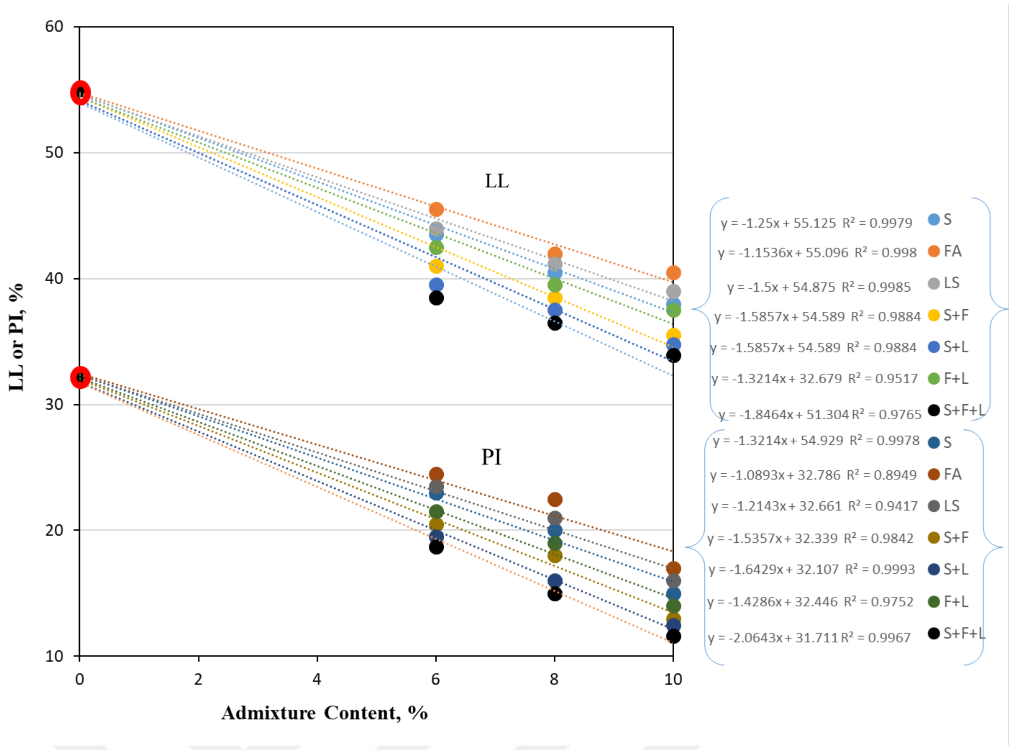


Figure 4.10: Atterberg limittest results for stabilized soil type 2.

4.1.4. Permeability

Permeability measures how easily water can flow through soil and is among the most significant geotechnical parameters but is probably more difficult than any other to establish. It plays a significant role in controlling soils' strength and deformation characteristics and has a direct impact on the quantity of water that will flow in the direction of an excavation, the design of a landfill liner's layer of clay, and how cut-offs should be designed that will go under a dam on foundations that are permeable.

The values of the permeability coefficients (k) versus mineral admixtures content at different faces of replacements (unary, binary, and ternary) is presented in Figures 4.11 and 4.12 for soil1 and soil2, respectively. It's clearly observed that adding mineral admixtures caused inclined towards reduction regardless to soil types for both groups of replacements. This may be attributed to regularly distribution of different particle sizes of soils, and the other mineral admixtures directed decrease voids of the materials. Therefore, materials with low void ratios have more tendencies towards resistance to water flow through the soil resulting to decrease permeability (Jerez et al., 2018). Throughout soaking process, some clay particles repelled the water and stay dry. Thus, the tests were conducted after 3 days of water soaking to ensure of fully saturation of soils. This behavior may be attributed to perfect adhesion of the fine particles of the clay. Furthermore, it can be noticed from aforementioned figures, precisely, at the unary shape of replacements, the permeability coefficients decreased with addition of pozzolanic materials up to 8% for the two series of soils. Nevertheless, this behavior is different at binary and ternary replacements, as the results of 8% and 10% replacements are approached to each other for the former and improvements slightly continued at the later face of replacement, irrespective of the soil types. Specifically, adding 10% of S+FA+LS (i.e. SFL mixture) caused an enhancement of a permeability coefficients by 34%, 40%, and 42% for the first and 30%, 35%, and 36% for the second groups, compared to non-stabilized soils. Improving permeability of soils via mineral admixtures also mentioned by the other researchers (Wang et al., 2018; Jerez et al., 2018; Zhou et al., 2014). For example, Jerez et al. (Jerez et al., 2018) decreased the permeability conductivity of soils from 82x10^{-8} cm/s to 39x10^{-8} cm/s through replacing soils by 3% of coal-bearing metakaolin.

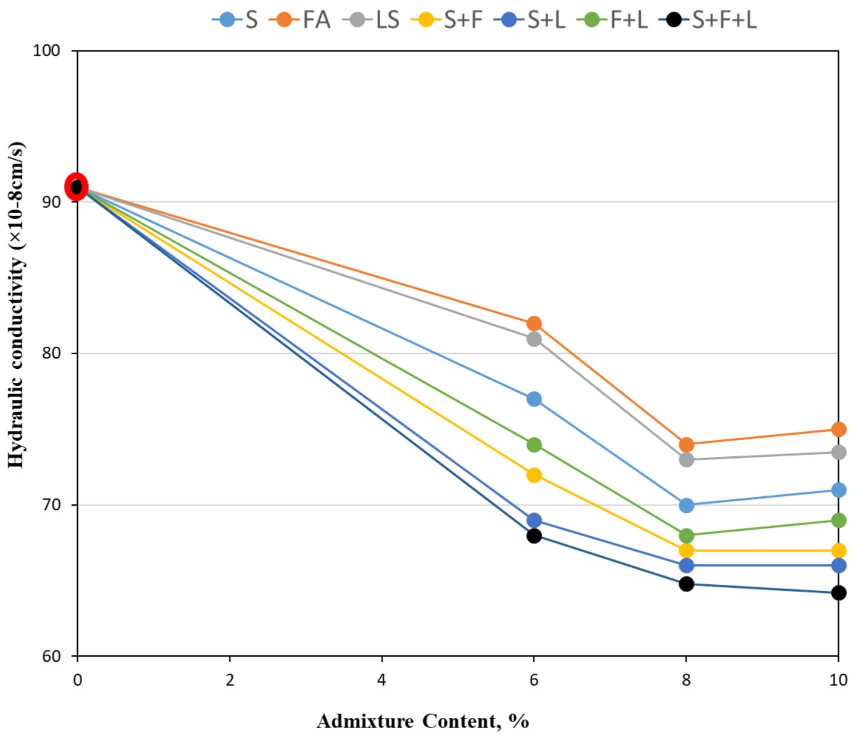


Figure 4.11: Permeability test results for (a) stabilized soil type 1

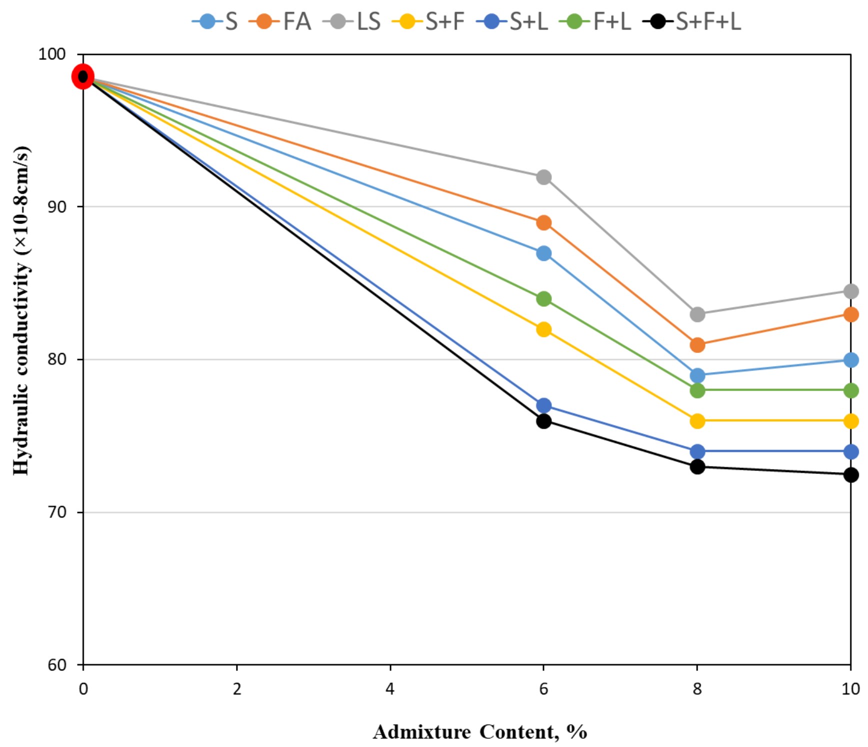


Figure 4.12: Permeability test results for stabilized soil type 1

4.1.5. Mechanism working of admixtures

Adding pozzolanic materials covers the clay particles in an activated soil mixture with a coating that is impermeable and insoluble, thereby improving its strength [16]. In this new matrix, the void between soil particles is filled by pozzolanic particles which increases the density, eliminates voids and decrease the impermeability, binding the particles together with a considerable improvement in strength [17]. Increasing the admixture contents increases the amount of C_2S and C_3S which, in turn, increases the dry compressive strength [18]. Mixing pozzolanic material into the soil when water is present causes hydration reactions and the C_3S and C_2S in the pozzolanic material react with water to form complex calcium silicate hydrates.

4.2. RESULTS AND DISCUSSION ON ENVIRONMENTAL EFFECTS

4.2.1. Safety issues

The reason that cement bags are printed with health and safety warnings is because cement is highly alkaline and because the process is exothermic so that wet cement is very caustic and can be the cause of severe skin burns if it is not immediately washed off with water. In the same way, serious respiratory or eye irritation can result from contact between dry cement powder and mucous membranes, while cement dust reacting in the lungs and sinuses with moisture that occurs naturally there can cause chemical burns, headaches, fatigue (Oleru, 1984) and lung cancer (Rafnsson et al., 1997). Research is ongoing into producing cements that are less alkaline ($pH<11$) (Coumes et al., 2006). Regulations in the United Kingdom, France and Scandinavia limit the level of the toxic skin irritant chromium to a maximum of 2 parts per million (ppm), while in America the Occupational Safety and Health Administration (OSHA) has set the legal upper limit for workplace exposure to Portland cement during an eight hour workday at 50 mppcf (million particles per cubic foot). The recommended exposure limits set by the National Institute for Occupational Safety and Health (NIOSH) are 10 mg/m^3 total exposure and 5 mg/m^3 respiratory exposure over an 8-hour workday. At levels of 5000 mg/m^3, Portland cement is immediately dangerous to life and health (CDC, Archived from the original on 21 November 2015. Retrieved 21 November 2015.).

4.2.2. CO_2 emissions

The concentration of carbon in cement ranges from ≈5% in cement structures to ≈8% in cement roads (Scalenghe et al., 2011). Manufacturing cement releases CO_2 into the atmosphere directly when calcium carbonate is heated and produces lime and carbon dioxide (EIA; Matar and Elshurafa, 2017) and indirectly through energy consumption if producing the energy involves CO_2 emissions. Some 10% of man-made carbon dioxide emissions worldwide is produced by the cement industry, with 60% coming from the chemical process itself and 40% from the burning of fuel (Jos et al., 2012). A 2018 Chatham House study estimates that annual cement production produces 8% of worldwide CO_2 emissions (Lehne and Preston, 2018). Every tonne of Portland cement produced results in the emission of almost 900 kg of CO_2. The European Union has reduced the energy consumed in producing cement clinker by about 30% since the 1970s, reducing the primary energy requirements by the equivalent of about 11 million tons of coal each year. This accounts for about 5% of anthropogenic CO_2 (Mahasenan et al., 2003).

4.2.3. Environmental impacts

Cement manufacture is environmentally polluting at every stage including the emission of airborne dust, gases, noise and vibration from machinery operating and quarry blasting as well as damage to the countryside from quarrying. There is widespread use of equipment to reduce the emission of dust both during quarrying and in the manufacture of cement, while there is also increasing use of equipment that captures and separates exhaust gases. Another aspect of environment protection is the return of quarries to the countryside, either in cultivation or in a natural state, after they have ceased production.

4.2.4. Green admixtures

Increasing awareness of environmental issues have led to such admixtures as fly ash, slag and lignin becoming increasingly valued as recycled materials with a consequent reduction in environmental impact because they conserve resources and save energy. There is a need for a study of how effective it is to use recycled and recyclable materials instead of natural resources in terms of both social and environmental sustainability (Chen et al., 2017). Many cementitious materials are used to improve soil’s engineering properties and reduce the damage that an expansive soil can do to structures by way of swelling or strength reduction.

Some 60% of the emissions of carbon dioxide produced in Portland cement manufacture comes from limestone’s chemical decomposition to lime which is used in Portland cement clinker, and reducing the amount of clinker in cement would reduce the emissions. An alternative would be to change production methods, for example by using pozzolanic materials instead of cement. A number of countries have regulated for emissions to be limited, though in America as at 2011, it was said that cement kilns were "legally allowed to pump more toxins into the air than are hazardous-waste incinerators (Berkes, 2011).

A cementitious material that can meet or exceed OPC’s functional performance levels through the incorporation and optimisation of recycled material is known as green powder. It reduces consumption of both energy and natural raw materials and therefore offers a sustainable alternative construction material to cement. A great deal of research is currently being carried out into the replacement of cement by other materials for soil stabilisation to bring down and possibly eliminate the production of pollutants and greenhouse gases and particularly of carbon dioxide (Harichane et al., 2012; Abdi, 2011; Chen et al., 2014; Zhang et al., 2017).

4.2.5. Conventional admixtures

Cement, gypsum, lime, and other materials containing calcium are among the cementitious materials used to improve soil's engineering properties (Sariosseiri and Muhunthan, 2009; Horpibulsuk et al., 2012; Shen et al., 2014). If the intention is to use cement in significant volumes, the improved soil's properties will tend towards concrete or cement mortar, in which case traditional geotechnical assessment methods are not appropriate (Tsuchida and Tang, 2015). The cementitious approach can have economic limits and it has become quite common to use chemical modification as a way of combating shortcomings in problem soils (Manso et al., 2013; Aldaood et al., 2014; Saride et al., 2013). Conventional admixtures can have an effect on soil brittleness and studies have investigated the effect chemical stabilisation using gypsum and cement on such of the soil's mechanical properties as brittleness, stiffness, and peak and residual strength. A summary will be found in Table 4.3 (Haeri et al., 2006; Lee et al., 2009; Schnaid et al., 2001; Consoli et al., 1998; Abdulla and Kiousis, 1997), which shows that the majority of the soils became extremely brittle after conventional admixture stabilisation. A drawback to the use of chemical stabilisers, however, is that they can damage the environment, limit plant growth, and change the quality of groundwater (Chen and Indraratna, 2014; Alazigha et al., 2016). It is also the case that using traditional stabilisers can, through making the soil behave in a more brittle manner, affect seismic stability in geotechnical projects (Chen and Indraratna, 2014).

Traditional chemical stabilisers, though, may not find ready acceptance in engineering construction because quicklime and cement are not renewable, their production involves consumption of large amounts of energy, they release considerable volumes of greenhouse gases and the brittleness of the resulting stabilised soils can affect structures' stability, especially under traffic or impact loading (Horpibulsuk et al., 2004; Sariosseiri and Muhunthan, 2009; Okyay and Dias, 2010).

Table 4.4: Summary of the effect of cement and gypsum stabilisation on soil behaviour reported in recent literature.

Stabilizer	Test condition	With increase the amount of stabilizer				Reference
		Peak strength	Stiffness	Residual strength	Brittleness	
Gypsum	Consolidated, drained, at dry condition	Increases	Increases	Higher for treated soil	Increases	Haeri et al. (Haeri et al., 2006)
Gypsum	Saturated, consolidated, drained triaxial tests	Increases at higher level of cementation	–	Not affected	Higher for higher cementation level	Lee et al. (Lee et al., 2009)
Cement	At MDD[a] and OMC[b], 7 days cured, Saturated, drained triaxial tests	Increases	Increases	Not affected	Increases (become highly brittle)	Schnaid et al. (Schnaid et al., 2001)
Cement	At MDD and OMC, 7 days cured, Saturated, drained triaxial tests	Dramatically increased	Dramatically increased	Not affected	Strongly brittle	Consoli et al. (Consoli et al., 1998)
Cement	Compacted, dry triaxial tests	Increases	Increases	-	Increases (become highly brittle)	Abdulla and Kiousis (Abdulla and Kiousis, 1997)

[a] MDD: Maximum dry density.
[b] OMC: Optimum moisture content.

4.2.6. Economic impacts

Stabilising soils with problem or waste materials has particularly significance in environmental protection and economic terms. The cost of cement stabilisation is assumed to be $20 per ton, 85% of which is energy costs. Using waste materials reduces the cost to about $10/ton in Turkey since the only cost of most waste materials is the cost of transport. Using cement is therefore much more expensive than using waste materials and, even then, the calculated costs for cement stabilisation do not include greenhouse gases, environmental damage from cement production and binding large amounts of limestone. The final assessment will only be possible after extensive technological, macroeconomic and environmental analysis.

Finding uses for waste materials helps the factories to get rid of them and also benefits the environment in a number of ways. Using waste materials is not only more economic but also makes it possible to preserve scarce and expensive natural resources required in cement manufacture and makes the construction industry more sustainable. Stabilisation analysis should thus be directed at encouraging geotechnicians to use waste materials for environmental and economic reasons.

CHAPTER 5

CONCLUSIONS

An experimental program was conducted to investigate the effects of different disposal materials on the properties of two soil types through stabilization process. Based on the findings of this study the following conclusions were drawn:

1. The ternary replacements seemed to be preferable than binary then individually for all test results. This probably due to compensation of most pozzolanic through the ternary mixture of disposal materials that necessary to complete chemical reactions.

2. At binary replacements of both soils, the most favorable replacements were related to the mixture of SL8 followed by SF8 and FL8, respectively. In contrast, S was improved soil properties than correspondingly replacing of LS and FA.

3. The test results suggested replacement orders of S+FA+LS, S+LS, and S for the ternary, binary and unary, respectively, considering their beneficial effects on the stabilized soils.

4.CBR results of the soils increased up to 8% mineral admixtures content beyond which results began to decrease. The decrease of the results at 10% of replacement is may be due to agglomeration of cementation material particles. The ternary replacement has higher CBR results than binary then unary respectively, which could be attributed to the regularly distribution of different mineral admixtures from micro to millimeter.

5. The higher CBR of the soil type 1 exceeded those of soil type 2 results owing to the fact of superior general properties of the former than later. Nevertheless, the rate of increment is better in soil2 than soil1. This may be considered to the properties of soil2 is more responsive with the mineral admixtures than soil1.

6. It was observed that the UCS of the soils continuously increased up to 8% of replacement mineral admixtures for unary and binary mixtures. Nevertheless, at

ternary replacements, the growth of strength from 8% to 10% seemed to be quite limited or near to each other.

7. Addition of mineral admixtures to the soils decreases the LL and PI of this soils with keeping unchanged of plastic limits (PL), because of non-plastic nature of the S, FA, and LS particles. Thus, it is significant to notice that the added mineral admixtures transformed the soil classification from highly plastic to low plastic clay.

8. It can be noticed from the results, precisely, at the unary shape of replacements, the permeability coefficients decreased with addition of pozzolanic materials up to 8% for the two series of soils. Nevertheless, these behavior is different at binary and ternary replacements, as the results of 8% and 10% replacements are approached to each other for the former and improvements slightly continued at the later face of replacement.

9. Traditional chemical stabilisers use quicklime and cement which are not renewable, their production consumes large amounts of energy and releases large volumes of greenhouse gases, and the stabilised soils that result is brittle which can affect structural stability, especially under traffic or impact loading.

10. Stabilization analysis should therefore be designed to encourage geotechnicians to use waste materials for environmental and economic reasons.

11. The depth of all road layers can be reduced when soils with low CBR values are stabilized with pozzolanic materials with environmental and economic benefits.

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