STUDIES ON LITHOMARGIC CLAY STABILIZED USING GRANULATED BLAST FURNACE SLAG AND CEMENT

Thesis

Submitted in partial fulfilment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

by

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D E C L A R A T I O N

by the Ph.D.Research Scholar

I hereby *declare* that the Research Thesis entitled "**Studies on lithomargic clay stabilized using granulated blast furnace slag and cement**" which is being submitted to the **National Institute of Technology Karnataka, Surathkal** in partial fulfilment of the requirements for the award of the Degree of **Doctor of Philosophy** in **Civil Engineering**, is a *bonafide report of the research work carried out by me*. The material contained in this Research Thesis has not been submitted to any University or Institution for the award of any degree.

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C E R T I F I C A T E

This is to *certify* that the Research Thesis entitled “**Studies on lithomargic clay stabilized using granulated blast furnace slag and cement”** submitted by **Mr. DARSHAN C SEKHAR** (Register Number: **145022CV14F02**) as the record of the research work carried out by him, is *accepted as the Research Thesis submission* in partial fulfilment of the requirements for the award of degree of Doctor of Philosophy.

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DEDICATED TO

MY PARENTS

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ABSTRACT

Southwestern coast of India has vast deposits of problematic silty soil (locally called shedi soil) normally referred as lithomargic clay in the literature. This problematic silty soil is characterized by its high sensitivity to moisture content with high erosion potential and low shear strength, especially in wet conditions or when it absorbs sufficient moisture. In order to overcome these problems, an industrial by-product obtained from the iron industry i.e. granulated blast furnace slag (GBFS) has been used to improve the strength properties of the lithomargic clay. To achieve the study objectives, lithomargic clay was replaced with the GBFS in different proportions. From the experimental results, it was observed that lithomargic clay when replaced with 25% GBFS produced good improvement in UCC strength and shear strength. The study includes an investigation on a combination of lithomargic clay replaced by optimum percentage of GBFS with addition of varying percentage of cement on their shear strength parameters. The improvement in strength was justified by conducting microstructural analysis using SEM and XRD. The experimental results are used in numerical analysis i.e., in PLAXIS 2D for load-settlement analysis of a strip footing and for a typical embankment slope stability problem. In addition, studies were carried out to check the effectiveness of GBFS and cement in the production of compressed stabilized earth blocks (CSEB). From the current study, it is concluded that lithomargic clay stabilized with GBFS and cement can be effectively used in geotechnical applications, thereby increasing the rate of effective disposal of GBFS.

Keywords: Lithomargic clay, Granulated Blast Furnace Slag, Stabilization, PLAXIS 2D, Compressed Stabilized Earth Blocks, SEM, XRD.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOMENCLATURE

Abbrevations

CAH	Calcium Aluminum Hydrate
CAOH	Calcium Aluminum Oxide Hydrate
CASH	Calcium Aluminum Silicate Hydrate
CBR	California Bearing Ratio
CSEB	Compressed Stabilized Earth Blocks
CSH	Calcium Silicate Hydrate
CSHH	Calcium Silicate Hydroxide Hydrate
GBFS	Granulated Blast Furnace Slag
GGBFS	Ground Granulated Blast Furnace Slag
IS	Indian Standards
LC	Lithomargic Clay
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
OPC	Ordinary Portland Cement
PC	Portland Cement
pH	Potential of Hydrogen
UCS	Unconfined Compressive Strength
UU	Unconsolidated Undrained

Notations

w_L	Liquid Limit
w_P	Plastic Limit
w_S	Shrinkage Limit
I_P	Plasticity Index
γ_d	Maximum Dry Unit Weight
c	Cohesion
φ	Angle of Internal Friction
$^\circ$	Degrees

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Every civil engineering structure, whether it is a building, a bridge, a tower, an embankment, a road pavement, a railway line, a tunnel or a dam, has to be founded in soil or rock and thus shall transmit the dead and live loads to the soil stratum. Proper functioning of a structure depends critically on the success of the foundation element resting on the subsoil.

In earlier days, the land possessing good engineering sites were available in plenty. Now, the situation has changed. The good sites are not available. Due to rapid urbanization and industrialization, the low lying areas are filled with mainly poor soil. The construction and design of embankments and foundations on such kind of problematic soils is challenging assignment for engineers. With increase in construction activity both onshore and off shore, it has become imperative to solve geotechnical problems concerned with soft, organic and compressible soil. Soil used for foundation and embankment construction should possess adequate strength and incompressibility. Obtaining such soil satisfying all conditions is impossible. In such cases, the available soil is modified and its properties are improved by some chosen method (Sarvade and Nayak 2014).

1.2 SOIL STABILIZATION

Sites having poor engineering properties like low lying agricultural and marshy lands that were once discarded as not suitable for construction activities are now being used because of the rapid industrialization and urbanisation. Hence, ground improvement is the need of the hour.

Soil stabilization is the process of improving the engineering properties of the soil by addition of a special soil or a cementing material or other chemical materials and thus making it more stable. This technique is used to improve the shear strength and to reduce the permeability and compressibility of the soil mass. One of the most common stabilization process includes the mixing of natural coarse grained soil and fine grained soil to obtain a mixture that have adequate angle of internal friction and cohesion thus providing a material that is stable and workable during the placement operations.

In many cases, the ground conditions and the earth materials are not ideal for proposed or planned development. Poor soil conditions inhibit sound construction and development of quality infrastructure. As development continues throughout the world, many of the ideal sites have already been built upon, leaving less desirable sites for future use. This necessitates the use of sites with soils of marginal quality. Thus, ground improvement techniques have been used which provide suitable alternatives for new construction in a previously discarded site with poor soil conditions. Modification of the earth materials or stabilization of soils can be done to achieve the desired goals of assuring adequate engineering properties and responses for a variety of applications and conditions. In addition, the precious resources of earth materials can be preserved with this better treatment of existing soils to provide acceptable engineering properties. Nowadays, the disposal of industrial waste materials have become a serious issue and thus the reuse of waste materials or industrial by-products for stabilization of soil can solve the problem to a great extent. Various techniques can be used and they are aimed at improving the properties by:

- Reducing the compressibility to avoid settlement.
- Increasing shear strength, stability, bearing capacity, stiffness and durability.
- Modifying permeability.
- Mitigating the undesirable properties such as shrinkage or swell potential and liquefaction potential.
- Improving durability to dynamic/repeated loads, including freeze-thaw.

These improvements can be done during the preconstruction, construction or post construction phases. They provide a diverse choice of approaches for solving the challenges.

1.3 LITHOMARGIC CLAY

The southwestern coastal area of India has a hard crust on the top. These top layers of the laterite formations are highly porous but hard and strong. Below these hard laterites, soils consisting mainly of silt is present. These silty soils dissolve and flow like water when water gushes through this layer during monsoon and many times washes–off the fine soil, creates cavities and at time causes heavy settlement and sliding of the top layers after the application of load. This bed soil is termed as lithomargic clay, locally known as shedi soil. This type of soil is abundantly available in regions starting from Cochin to Goa (Nayak and Sarvade 2012).

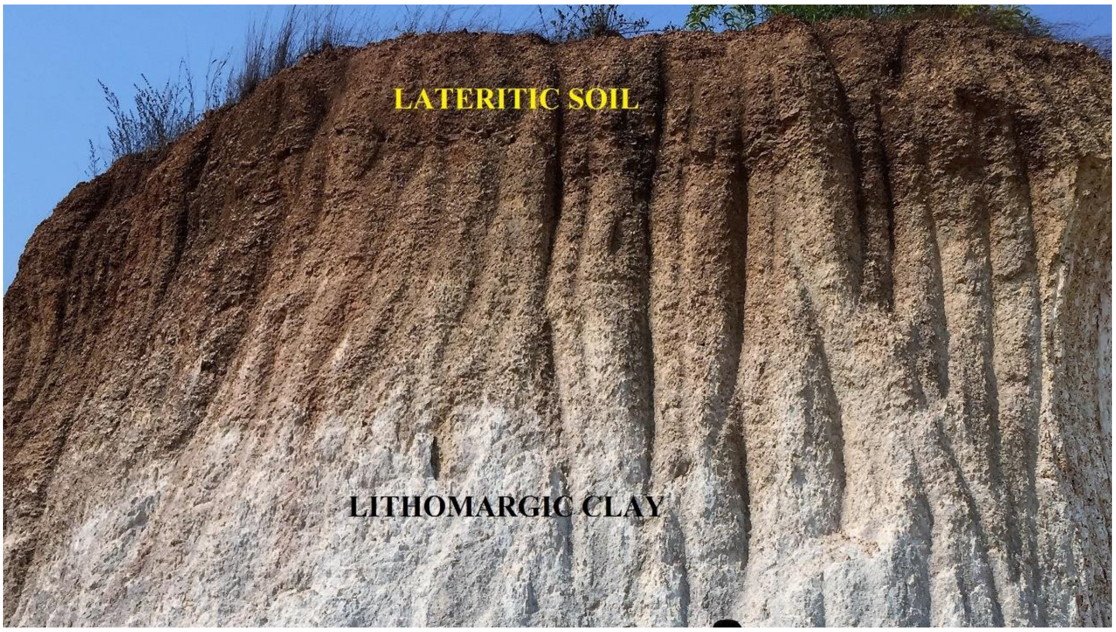

Fig. 1.1 Profile of lateritic soil and lithomargic clay at Padupanambur site, Mangalore, Karnataka, India

Lithomargic clay is mainly composed of hydrated alumina and kaolinite powder. It is whitish, pinkish or yellowish in colour, consisting mainly of silt and sand particles. This soil is present in between weathered laterite and hard granite gneiss and is found at a depth of 1–3 m below the top lateritic outcrop throughout the western coast of south India (figure 1.1). Lithomargic clay is the product of tropical or subtropical weathering. Lithomargic clay mainly composes of silty particles in a size range of 2 to 75 microns.

These are commonly formed by the mechanical weathering of rocks. It tends to exist in a meta-stable state (Nayak and Sarvade 2012).

When lithomargic clay loses its strength when it is exposed to water. This makes it unsuitable for construction of many structures. This soil also has high erosion potential. Predominance of dissolved Na^+ makes the soil susceptible for erosion. Such soil tends to disperse even in still water. It does not possess any desirable engineering properties and its behaviour is unpredictable, especially when the soil is fully saturated. As a consequence of wetting, slope failure, embankment failures, cavity formation in tunnels, heaving of soil and settlement of foundation may occur. This soil is neither suitable as foundation material nor as a filling material in construction activities. Therefore, construction on this type of soil requires special design and precautions.

1.4 GRANULATED BLAST FURNACE SLAG (GBFS)

The use of normal stabilizers like cement, lime etc. for soil stabilization is being reduced because of the increase in the cost of cement and its effect on the carbon footprint. Nowadays, the emphasis is on the material handling ability and achieving the desirable properties. Utilization of industrial wastes/by-products mainly reduces their disposal problems and the cost of the project. Such materials provide social, economic and environmental benefits when used in soil stabilization.

Due to rapid industrialization and urbanization, steel is one of the major constituent in construction activities. Granulated Blast Furnace Slag (GBFS) is obtained from the iron and steel industries in large quantities (i.e. >10 million tons/annum). This slag is obtained by quenching the molten slag with high-pressure jets. Quenching prevents the crystallization leading to the formation of granular glassy aggregates. When these slags are crushed, powdered and screened, they can be used in the production of Ground Granulated Blast Furnace Slag (GGBFS) and slag-cement because of their binding properties similar to that of cement (Gruskovnjak et al. 2006). For every ton of crude iron produced, about 300-540kg of slag is obtained. These slags mainly consists of calcim, magnesium, manganese and aluminum silicates in numerous combinations. Blast furnace slag with its high lime content (30-45%) upon exposure to water, hydrates forming cementitious pozzolanic reactions products similar to those formed during the hydration of cement (Kavak and Bilgen 2016). Moreover, this would reduce the CO_2 emissions produced from the manufacture of cement and conserve the limestone deposits that would be utilized in the production of cement. The National Slag Association (NSA) has proved that the use of these slags in construction activities does not possess any threat to the humans and environment (Indian Minerals Yearbook 2015). In addition, the cost of Granulated Blast Furnace Slag (GBFS) is inexpensive when compared to cement.

Fig. 1.2 Profile of granulated blast furnace slag at Kirloskar Ferrous Industries Limited, Koppal, Karnataka, India

With increase in demand for steel, iron industries are increasing their capacities for production and hence the higher amount of slag is being generated. Moreover, with the upsurge in cost of grinding these coarse slags to a finer size, which would be later used in production of GGBFS and slag-cement, there is a large accumulation of these slags near these iron industries. This is an added burden for these industries because large stacking of these slags disturb the surrounding environment and consume large areas (figure 1.2). Hence, to increase the rate of disposal of these slags and to check the efficiency of granulated blast furnace slag in stabilizing lithomargic clay, this work is proposed, as it would be a cost effective method of disposing slag. Thus, the overall objective is to maximize the slag utilization without any appreciable loss in strength (Rabbani et al. 2012, Dermatas and Meng 2003).

1.5 SCOPE AND OBJECTIVES OF THE INVESTIGATION

GBFS is a byproduct produced during iron manufacturing and is easily available in very large amounts and it is relatively economical compared to cement and other additives. Using GBFS and cement as soil stabilization agents may impart new and improved strength properties in lithomargic clay soils. It may impart greater strength than using any additive alone in the long term, which improves the durability and performance of structures. Using GBFS and cement for lithomargic clay stabilization is still a novel process in southwestern coast of India and it has never been used in manufacturing compressed stabilized earth blocks. Hence, there is need for research in this regard.

The main objectives of the study are:

1. To characterize lithomargic clay and GBFS according to their geotechnical properties.

2. To investigate the effect of GBFS with and without cement, on index and strength properties of lithomargic clay.

3. To identify the reaction products of the stabilized soil and to
    i. Identify mechanisms by which the changes in engineering properties will be achieved using X-ray diffraction.
    ii. Study the change in structural morphology using scanning electron microscopy.

4. To apply the experimental results in PLAXIS 2D software to
    i. Load-settlement analyses of strip footings.
    ii. Stability analyses of embankment slopes [soon after construction –UU condition].

5. The usage potential of GBFS in the manufacture of compressed stabilized earth blocks (CSEBs)

1.6 ORGANIZATION OF THE THESIS

The thesis work titled '***Studies on lithomargic clay stabilized using granulated blast furnace slag and cement'*** is presented in eight chapters as follows:

Chapter 1: The first chapter includes introduction to lithomargic clay, GBFS and their problems, the need for stabilization and the research objectives adopted for the thesis.

Chapter 2: A comprehensive literature available for the present study is reviewed in this chapter.

Chapter 3: This chapter provides a detailed discussion on the different materials, experimental tests and their methodology.

Chapter 4: The fourth chapter deals with the results obtained from the experiments conducted on lithomargic clay, GBFS and lithomargic clay stabilized with GBFS and cement.

Chapter 5: The microstructural analysis of stabilized soil using Scanning Electron Microscopy and X-Ray Diffraction is presented in this chapter.

Chapter 6: Load settlement analyses of strip footings and slope stability analyses (of embankment soon after construction) for soil alone as well as stabilized soil using PLAXIS 2D is dealt in this chapter by inputting the results obtained from the laboratory experiments.

Chapter 7: This chapter deals with the utilization of GBFS and cement in the manufacture and testing of compressed stabilized earth blocks made from two locally available soils.

Chapter 8: The conclusion drawn from the study along with scope for future work is presented in this chapter.

CHAPTER 2

REVIEW OF LITERATURE

2.1 GENERAL

There is a need for innovative research, knowledge transfer and best practice regarding the ground improvement methods. Soil stabilization is a useful civil engineering method that allows the in-situ ground to support an engineered structure. With the rise in carbon emissions resulting in global warming and climate change, successful new methods are vital. The impact of the traditional stabilizers does not restrict itself to the contamination of the geoenvironment, but turns out to be a serious global threat due to the emission of greenhouse gases. Hence, it becomes essential to find new alternatives. This environmental impact due to conventional additives can be reduced by the utilization of industrial by-products, which are sustainable, and easily available (Jayanthi and Singh2016). These by-products should be able to satisfy the requirements of an additive, to create more durable and sustainable composites. Through such materials, sustainable infrastructure development is possible. An attempt is made in this chapter to review the available literature regarding the usage of slag and cement in soil stabilization and their applications.

2.2 LITHOMARGIC CLAY AND LATERITIC SOIL

Laterite is well known in southwestern coast of India as a building material for generations. Lateritic soil is defined as a soil layer that is rich in iron oxide and derived from a wide variety of rocks weathers under strongly oxidizing and leaching conditions. It forms in tropical and subtropical regions where the climate is humid. The term lateritic soil is often substituted for ferricrete but technically refers to a soil rich in iron oxides and aluminum. It is a highly weathered material, rich in secondary oxides of iron and aluminum or both and hence its colour is reddish brown. Generally, it is expected that deeper the excavation, harder the ground met with but in lateritic formation, this situation is reversed as seen in the geoprofile. Hard stratum is met at top followed by soft stratum of weathering zone for considerable depth and the parent rock at the deepest level. According to strength criteria, it goes reducing from top to deeper level due to varying engineering properties. Laterites are formed by the decomposition of the rock, removal of silica and bases and accumulation of aluminum of iron sesquioxides, titanium, magnesium, clay and other amorphous products. Generally, a coarse-grained concretionary material with ninety percent or more of these laterite constituents is termed as laterites. These top layers of the laterite formations are highly porous but hard and strong. While relatively fine-grained material with lower concentrations of oxides are referred to as lateritic soils (Hegde and Davare 2010).

The problematic lithomargic clay soil, locally known as shedi soil is found from a depth of about 2 meters to 20 meters, underlying lateritic soil layer in the coastal area of Karnataka. This shedi soil is a non-expansive soil containing Kaolinite and Smectite as major clay minerals. (Ramesh et al. 2009). In between the top low-level laterites and bottom high-level laterites, lithomargic clays are present. They have a size distribution between sand and silt. Due to its dispersive nature, these soils dissolve and flow like water whenever it is exposed to water, which creates cavities and at time causes heavy settlement and sliding of the top layers after the application of load (Ramesh and Venkataraja Mohan 2011, Ramesh et al. 2013).

The strength property of shedi soil depends on density and compactive effort. The strength of shedi soil increases continuously with the addition of fly ash. The strength of lithomargic clay treated with 20% fly ash increases the strength with curing both for soaked and unsoaked conditions. However, for unsoaked condition the increase is 18 folds and for soaked condition it is 14 folds compared to shedi soil alone (Ramesh et al. 2011). The strength of shedi soil increases when it is stabilized with certain additives and it can be successfully used in the construction field (Shruthi and Kishore Kumar 2015). Shedi soil stabilized with pond ash showed improvement in strength. The gain in strength for 7 days cured samples may be because of long term reaction such as pozzolanic and carbonation (Suresh et al. 2009).

Studies on lithomargic clay showed a reduction both in modulus of subgrade reaction and in elastic modulus by 72% and 70% respectively due to soaking (Ravi Shankar and Suresha 2006). Improvement in strength is observed when lithomargic clay is stabilized with coir, pond ash and lime. The CBR of shedi soil at light compaction condition was <1% for soaked condition and 6% for unsoaked condition (Ravi Shankar et al. 2013). Reduction in plasticity characteristics and improvement in strength were observed in lithomargic clay when stabilized with quarry dust and cement (Nayak and Sarvade 2012).

When Portland cement is added to lateritic soil, CASH is formed. As the pozzolanic reaction progresses, CASH is slowly converted into a well crystalline phase to form calcium silicate hydrate (CSH) and calcium aluminium hydrate (CAH) which hardens with age to form a permanent compound that binds the soil particles. As a result, the shear strength of the stabilized soil is improved. UCS and CBR of the laterite sample would increase significantly with a cement quantity of 3% (Jaritngami et al. 2014).

2.3 SOIL STABILIZATION USING CEMENT, SLAG AND OTHER ADDITIVES

Utilization of conventional stabilizers like cement, lime put great pressure on the natural resources and to the global environment. There are significant environmental impacts associated with Portland cement (PC) production, such as high CO_2 emissions (0.95 t CO_2/t PC), energy consumption (5000 MJ/t PC), and non-renewable resources (1.5 t limestone and clay/t PC) (Higgins 2007). Hence, there is a great need to find new alternatives. Granulated blast furnace slag (GBFS) is a by-product obtained from the iron industry. Granulated slag is obtained by quenching the molten slag by means of high-pressure water jets. Quenching prevents crystallization, thus resulting in granular, glassy aggregates. This slag is crushed, pulverized and screened for use in various applications, particularly in cement production because of its pozzolanic characteristics. More than 10 million tonnes of slag per annum is produced in India.For every tonne of crude iron produced, about 300-540kgs of slag is obtained. The cement industry consumes up to 70% of the blast furnace slag generated. Nevertheless, the rate of disposal of these slags is less when compared to the amount of slag generated. With increasing capacities, there is large stacking of these slags in the vicinity of these industries, which causes environmental and disposal issues. Nevertheless, slag generation remains unavoidable and emphasis on its recycling remains one of the most serious concerns that need attention. Pertinently, the concerns of today are to pay adequate emphasis on minimization of waste generation, recycling and re-use of waste, and minimizing the adverse impact of disposals to the environment. Slag cement has low heat of hydration, low alkali aggregate reaction, high resistance to chlorides and sulphate and it can substitute for ordinary Portland cement. Hence, slag cement is helpful in reducing the impact on the global climate and environment caused by the production of cement (Indian Minerals Yearbook 2015).

Cement stabilization of clays showed large improvements in strength. This improvement is due to the increased reticulation forming CSH gel. Thus, clay particles flocculate into larger size clusters. The fast hydration reaction is accompanied by the much slower pozzolanic reaction over time. The secondary cementitious products appear to be deposited on or near the surfaces of the clay clusters. This give rise to a reduction in entrance pore diameter but an increase in particle size (Chew et al. 2004). The strengths of a soft marine clay significantly increased when mixed with cement. It was found that growths of CSH and ettringite with curing time were responsible to improved strength. Strengths were increased proportionally with amounts of the major hydration products such as CSH and ettringite that were formed. (Nontananandh et al. 2010). Water/Cement ratio is the key parameter controlling the strength of the treated soil samples. Higher initial water content requires more cement content to attain desired strength as compared to the soil treated at lower initial water content for the same water cement ratio (Suganya and Sivapullaiah 2013). C_3S and C_2S are responsible for the early and later strength respectively. With the addition of water, C_3S rapidly reacts to release Ca ions, OH^- ions, and a large amount of heat. This reaction slowly continues producing Ca and OH^- ions until the system becomes saturated. Soon, the calcium hydroxide starts to crystallize. Simultaneously, CSH begins to form. The formation of the $Ca(OH)_2$ and CSH crystals, results further formation of CSH gel (Ouf 2012).

Cement stabilization improves the soil structure by increasing inter-cluster cementation bonding and reducing the pore space. With time, the large pores are filled with the cementitious products; thus, the small pore volume increases, and the total pore volume decreases. This is the reason for improvement in strength with curing (Horpibulsuk et al. 2010). A three-step model of dissolution, orientation and hardening takes place in the mix. The reaction products depend also on the activator and on the prime materials (Pacheco-Torgal et al. 2008).

The formation of pozzolanic reaction compounds (CSH, CAH) in lime treated soil confirms the reduction of compressibility behavior by aggregation and strong binding of particles, which is mainly due to the pozzolanic reaction. Broad peaks of pozzolanic reaction compounds (CSH, CAH) are observed with curing period, causing more reduction in soil compressibility. Increase in ratio of Ca:Si with lime content and curing period may be attributed to formation of pozzolanic reaction product (Jha and Sivapullaiah 2014). Through Scanning Electron Microscopy analyses, it was observed that for any given percentage of lime, strength improved by the aggregation of soil particles and formation of compacted matrix with curing period. Cementitious compounds such as CASH, CAH and CASH were observed in lime treated soil at higher lime content from XRD analyses (Jha and Sivapullaiah 2015).

In lime stabilized soils, liquid limit decreased by altering its diffuse double layer. With increased lime content the amount of cementitious compounds that result in visible strength increases. Among these, the major cementitious compounds are gyrolite, CSH, and CASHH, which improves the strength and stiffness of the soil. However, further addition of lime reduced improvement in strength. This is due to the excess formation of silica gel, a highly porous material (Dash and Hussain 2012). The strength development in lime-stabilized soils is mainly due to the pozzolanic reaction and is a time-dependent phenomenon. The remaining lime content after short-term reaction (flocculation and cation exchange) is used for pozzolanic reaction (long term reaction). With the use of 2% and 4% lime, the maximum lime is consumed in the short term reaction and, hence, very less amount of lime is available for the long term reaction, leading to the marginal increase in strength. Strength reduction in lime stabilized gypsiferous soil is due to the formation of more ettringite needles. The formation of larger voids, reduction in cementitious ability and disturbance in the soil matrix leads to the sharp reduction in the strength of soil at 28 days (Jha and Sivappullaiah 2015).

The addition of a sufficient amount of lime induces a highly alkaline environment ($pH \geq 12$), which helps to promote slow clay dissolution and the formation of aluminate and silicate anions. Ca^{2+} cations can link aluminate and silicate anions and induce the formation of calcium aluminosilicate hydrates, a cementitious product bonding the adjacent soil particles together. These reactions, called pozzolanic reactions, are very slow at room temperature and yield various amorphous phases (gels). Adding more lime (>4%) seems to fill the porosity of the sample and reduces its permeability. With curing time, permeability decreases for the treated sample as the continuous lime/clay reaction modifies the pore size distribution by cementation and filling some pores. Changes in the texture, pore size and pore accessibility increase the tortuosity which leads to a reduction in permeability (Al-Mukhtar et al. 2012). In case of cement, the reactions are mainly hydraulic, while with lime they are pozzolanic. This means that cement needs only water to react, since the pozzolanic component is already incorporated into the cement, whereas lime needs water and a pozzolanic material, like clay (Cristelo et al. 2013).

Knowledge of the chemical, mineralogical, and morphological properties of slags is essential because their cementitious and mechanical properties, which play a key role in their utilization, are closely linked to these properties. Although slag without an activator does react with water, the rate of hydration is very slow. Coatings of alumino silicate form on the surface of slag grains within a few minutes of exposure to water, and these coatings were impermeable to water. Unless a chemical activator is present, further hydration is inhibited. Portland cement, gypsum and many alkalies have been used as activators, and it has been observed that the rate of hydration is faster at higher alkali concentrations. The pH of the solution plays an important role in the hydration process and also in determining the nature of C-S-H formation. With a solution pH below 11.5, it is hard to solubilize silica in spite of the presence of a chemical activator in the aqueous phase (Song et al. 2000).

Soft clays containing high organic matter when stabilized with cement or lime have shown no desired improvement in properties, due to the humic acid that affects the reaction. Hence, the partial replacement of cement by GBFS has produced effective results due to its potential hydraulicity that enabled pozzolanic reactions and increased the unconfined compressive strength of the soft clay. The strength of cement and slag stabilized clays is higher than the cement alone stabilized at high curing periods (Jiang et al, 2004). In PC-GGBFS blends, GGBFS is mainly activated by the portlandite, one of the PC hydration products indicating that hydrated lime may be used to activate GGBFS directly.

Dispersive clays can be stabilized satisfactorily using blast furnace slag and basic oxygen furnace slag (BOFS). This is due to the ion exchange phenomenon. As the additive content increases, causing a more flocculated structure and producing larger particles that have a lower tendency to erosion (Goodarzi and Salimi 2015). Improvement in bearing capacity is seen in clayey soils mixed with ladle furnace slag than those stabilized with lime. The curing time is longer and durability is better than that of lime-soil mixtures (Manso et al. 2013).

CBR values increased with GGBFS addition to the soil, especially in presence of lime. The un-soaked and soaked CBR values of samples have increased significantly (Rabbani et al. 2012). Granulated blast furnace slag (GBFS) and GBFS cement (GBFSC) reduced the expansion of expansive soil. With the addition of GBFS and GBFSC clay fractions decreased and silt fractions increased upon adding GBFS and GBFSC and Plasticity index decreased (Cokca et al. 2009).

Use of slag as an admixture for improving engineering properties of the soils is an economical solution to use the locally available poor soil. It is observed that with increase of additives, both the UCS and stability of soil is improved when compared to using lime alone. UCC strength of lime-slag stabilized black cotton soil increased up to 18 times that of natural soil (Manjunath et al. 2012). With the increase in GGBFS content, compressive strength increases by the re-arrangement of soil particles to reduce the voids. The pavement thickness can be reduced considerably with increases in percentage of GGBFS (Pathak et al. 2014). As the CaO/SiO_2 ratio increases, the rate of reactivity of the GGBFS also increases up to a limiting point. Increasing the CaO content makes granulation to glass phase content. (Takhelmayum et al. 2013). In comparison to conventional stabilizers like lime, cement etc. GGBFS is very economic and should thus be given serious consideration when specified for highways and foundations (Veith 2000). With the increase of GGBFS content liquid limit, plastic limit and plasticity index decreases, which makes the soil less plastic and hence plasticity index reduces. With the increase in GGBFS content, compressive strength as well as the CBR value increases (Kumar et al. 2015). Shape and size of fly ash and GGBFS are the main factors controlling the compressibility, resilient and permanent deformation characteristics (Sharma and Sivapullaiah 2016).

Since the slag introduces extra and more freely available alumina and silica due to its high reactivity in the presence of lime, the formation of the strength contributing silicates and aluminates is enhanced. In PC–GGBFS systems, the higher amounts of C–S–H gel are produced (Wild et al. 1998). The stabilization performance of the high plastic clay is better than that of the low plastic clay for each of the GGBFS contents. This study has revealed that the use of GGBFS waste material has the potential to modify the properties of clays in order to decrease their swelling potential, and therefore positively affect the stabilized soil samples (Sivrikaya et al. 2014). Hydration reaction of GGBFS is slower than that of the hydration of cement. The blocking of pores leads to higher strength and lower permeability (Wild et al. 1996). The GGBFS basicity is considered as the factor governing for hydraulicity. The more basic the slag, the greater is its hydraulic reactivity. In the presence of alkaline activators, the hydration of slag involves complex chemical and physical reactions such as adsorption, ion exchange, dissolution, and hydrolysis (Lizarazo-Marriaga et al. 2011).

GGBFS when mixed with clay without any other activator is able to produce only a low amount of the hydration products after a long time curing period. In fact, when the GGBFS is in a moist condition, it forms an Al−Si−O layer on the surface of its particle. However, the pH of the mixture and OH^- increase due to absorbed H^+ ions by this layer, but it is not sufficient to break the Al−O and Si−O bonds to generate the hydration products and only a small amount of the CSH will be generated after a long curing time. Therefore, application of the GGBFS is based on the power of its activator for breaking these bonds. The formation of CSH in the mixture of the GGBFS with clay is due to addition of the PC as an activator. Ettringite was not observed in the GGBFS mixed with clay pattern, as it is an early hydration product generates in GGBFS activation (Keramatikerman et al. 2016).

Slag has a relatively constant chemical composition compared to fly ash, silica fume, pozzolanas etc. Besides, it has advantages like low heat of hydration, high sulfate and acid resistance, better workability, higher ultimate strength, etc. These properties are beneficial in specialized applications such as hydroelectric dams, large bridges, power stations, metro systems, motorways, and harbours (O nera et al. 2003).

The stiffness and strength improve with increase in PC content. The dissolution rate of GGBFS generally depends on the alkali concentration of the reacting system. Formation of hydration products, leading to higher rate of strength development, are determined by the availability of Ca^{2+} provided by the free lime content (Konsta-Gdoutos and Shah 2003). Initial tangent modulus and UCS of soil-lime and GBFS mixture increase with increase in additives and also with curing periods (Swamy et al. 2015). Utilization of lime and GGBFS in soil stabilization offers a slower early-rate of strength development, providing more time for construction operations. There is also extra ability to self-heal, in the case of early-life damage caused by overloading. There is an improvement in structural performance in the long term (Higgins 2005).

Soil stabilization with slag-lime reduces the dry unit weight. The soil transforms into a rapid structure and the modulus of elasticity increases. The improvement in strength with the combination of GGBFS and lime is much larger when compared to lime stabilization (Kavak and Bilgen 2016). Both the liquid limit and plastic limit decreased with an increase in the slag content. This is due to the less moisture-holding capacity of the slag. The soil–slag mix is a non-swelling-type material having less affinity towards water. There was a large improvement in UCS with little addition of cement content. Therefore, the slag–soil mix stabilized with adequate cement content can be used as a suitable construction material (Athulya et al. 2016). Both fly ash and GBFS are granular particles having no cohesion between the particles. The strength offered by the compacted fly ash–GBFS sample is mainly due to the mobilization of frictional strength of the materials. With increase in cement content in the mixture, the quantity of gel formation increases, which binds the particles more effectively resulting in higher CBR value (Singh et al. 2008). The hydrated lime-activated GGBFS with lime to GGBFS ratio of 0.10 is recommended to replace PC for this soft marine clay stabilization application for environmental and economical benefits (Yi et al. 2015).

Addition of GGBFS reduces optimum moisture content as well as maximum dry unit weight. This is due to the reduction in clay fraction of the soil and therefore the lesser water holding capacity. The reduction in MDD is due to the predominant effect of high frictional resistance offered by coarser GGBFS particles due to size and surface texture resisting the compactive effort effectively. However, the effect of reduction in water holding capacity and increase in frictional resistance are more or less evenly balanced at lower GGBFS contents and maximum dry unit weight remains unaffected (Sharma and Sivapullaiah 2012). Strength increases significantly with addition of little percent of lime to slag stabilized soil, however at higher content of lime the improvement in strength is negligible (Sivapullaiah 2013). Unconfined Compressive Strength (UCS) of black cotton soil increases with the addition of small amount of GGBFS, which remains constant up about 40% addition of GGBFS. With further addition of GGBFS, the UCS decreases continuously. This is due to the reduction in clay fraction of the soil with addition of coarser GGBFS particles and the effect of compaction parameters as the soil GGBFS mixes, which are compacted at their respective optimum water contents. With increase in the GGBFS percentage the available pozzolanic material increases but the available water for pozzolanic reactions reduces due to decrease in their moulding water content. Additionally, the moulding unit weights are also lower with increasing GGBFS percentages. At higher GGBFS replacement (i.e. >40%) the effect of decreased moulding water content and density dominate and hence the strength decreases (Sharma and Sivapullaiah 2011).

Fly ash mixed with GGBFS has the potential to improve the properties of expansive soil with a minimum requirement of chemical additives such as lime. The pozzolanic reaction can be enhanced by adding lime or cement that could improve the performance of fly ash/GGBFS mixtures (Sharma and Sivapullaiah 2016). The cementitious and pozzolanic behavior of ground granulated blast furnace slag is essentially similar to that of high-calcium fly ash. Since the pozzolanic reaction is slow and depends on the calcium hydroxide availability, the strength gain takes longer time for the GGBFS concrete. Calcium hydroxide is produced by the hydration of cement and consumed in the pozzolanic reaction. The pozzolanic reaction can only takes place after the Portland cement hydration starts. As the cement content increases, the hydration product calcium hydroxide also increases and more calcium silicate hydrates are formed due to reaction with GGBFS. Hence, pozzolanic reaction is slow and the formation of calcium hydroxide requires time (Oner and Akyuz 2007). The main hydration product of GGBFS was Calcium Silicate Hydrate, regardless of the activator type (Yi et al. 2014).

The decrease in initial strength of GGBFS cement could be overcome if the fineness of GGBFS were increased to promote higher rate of hydration. However, increasing the fineness of slag by pulverization could easily increase the manufacturing cost of GGBFS. GGBFS is a low performance cementitious material, which can achieve high compressive strength when an alkaline activator is used (Kim et al. 2011).

In lime-fly ash stabilized soils, the calcium from lime and fly ash reacts with soluble alumina and silica from clay and fly ash, in presence of water to produce stable CSH and CAH. The reduction in pore spaces was observed which generates long-term strength gain and improve the geotechnical properties of the soil (Sharma et al. 2012). For class-F fly ashes, which usually require high amounts of lime or cement as additives, the addition of GGBFS will help to enhance its mechanical properties largely even with a small amount of additive such as lime. The addition of lime to fly ash–GGBFS mixtures further increased the UCS by accelerating the pozzolanic reaction (Sharma and Sivapullaiah 2016). Stabilization of clayey soils with high calcium fly ash depends on the type of soil, the amount of stabilizing agent and the curing periods. Higher amounts of tobermorite is produced leading to a denser and more stable structure of the stabilized material. A further addition of cement provides better setting and hardening and the combination of these two binders can increase the early as well the final strength of the samples (Kolias et al. 2005).

2.4 SETTLEMENT ANALYSIS AND SLOPE STABLILITY ANALYSIS USING PLAXIS SOFTWARE

The soil beneath the footing was replaced with a granular fill and load settlement response were measured through experimental investigations. From the analysis in PLAXIS, it was observed that the granular-fill layer helps to improve the load-bearing capacity of the footing and reduces the settlement since the granular-fill layer is stiffer and stronger than the natural clay. The partial replacement of the soil with the granular-fill layer results in a redistribution of the applied load to a wider area and thus minimizing the stress concentration and achieving an improved distribution of induced stress. Hence, the bearing capacity increased while the footing settlement decreased (Ornek et al. 2012).

PLAXIS software was used to analyze strip footing resting on granular layer over weak soil improved by end bearing or floating granular piles. Granular piles of different diameters, lengths, stiffness and arrangements were modeled. It was found that stiffness of granular layer has little effect compared to a significant effect of other parameters on the vertical and differential displacements and the induced bending moment of the strip footing (El-Garhy and Elsawy 2017).

A comparison of embankment slope stability analysis with the limit equilibrium method computer program Slope/W and the finite element method computer program Plaxis 2D were analyzed. The results indicated that it is important to use the effective shear strength characterization of the soil when performing the slope stability analysis. The factor of safety computed from both Slope/W and Plaxis 2D decreases as the slope angle becomes larger. The limit equilibrium method overestimated the factor of safety as compared to the finite element method (Rahman 2012).

2.5 COMPRESSED STABILIZED EARTH BLOCKS

Shelter is one of the basic needs of humans, especially for the lower income groups. Lack of materials and their higher cost have encouraged research to find new substitutes to convectional building materials. Cost reduction in the housing sector especially with the lower income sections can be achieved by innovating new construction materials, which can be locally made, and with ease of construction. Among sustainable construction techniques, stabilized earth seems to be noteworthy. From the past 50 years, Compressed Stabilized Earth Blocks (CSEBs) are used for load bearing masonry construction in various countries. The term block is used to differentiate from brick, which is usually fired. The main advantage of these blocks is that they can be locally made with simple construction methods with semi-skilled labour, not requiring a very specialized equipment, offering high thermal and acoustic insulation. These CSEBs are about 2.5 times larger than conventional fired clay bricks and therefore construction is faster with lesser joints. Moreover, they consume lesser energy when compared to that of fired clay bricks or concrete masonry, thus making them cheap and affordable (James et al 2016). Regardless of these advantages, the use of CSEBs are restricted due to its below par performance in durability, tensile strength, impact and abrasion resistance when compared to conventional fired clay bricks. Moreover, the lack of guidelines for both manufacturers and builders converts it into low acceptance of these CSEBs in housing sector. CSEBs are manufactured by compressing a wet soil mixture and a suitable stabilizer in a manually operated press to get a high density block. For meeting the requirements of building codes, small amount of cement is generally used as a stabilizer as it is easily available and gives the necessary strength and durability properties to these blocks.

Compressed stabilized earth blocks (CESBs) are eco-friendly, having sufficient strength and durability properties with good insulation properties. The use of CSEBs promotes healthier living for lower income section of the society (Walker 1995). As clay content increases, strength of blocks are reduced. Reduction in compressive strength with immersion in water for 48 hours is due to the development of pore water pressures and the liquefaction of unstabilized clay minerals in the block matrix (Ramirez et al 2012). The amount of cement to be used will depend on the composition of the soil. Sandy soils require 5 to 9% cement by volume. Silty soils need 8 to 12%, and clayey soils require 12 to 15% cement as stabilizer. Cement content more than 15% is uneconomical (Nagaraj et al. 2014). It is the binding of sand particles, and the self-hydration products of the cement that contribute to the early strength of the blocks (Walker and Stace 1997). Cement stabilized soil blocks are ideal for low-rise residential construction, where minimum strength requirements are often dictated by handling rather than load carrying requirements. For this purpose, a minimum saturated compressive strength of 1.0 MPa may be considered satisfactory (Bahar et al. 2004). The reduction of compressive strength upon saturation goes up to 60% for cement-stabilized samples. The reduction in compressive strength upon saturation was lower when high percentages of cement is used (Taallah et al. 2014).

The increase in dry compressive strength would be due to the increasing amount of C_2S and C_3S brought about by increasing cement content (Kwon et al 2010). Strength and durability of cement stabilized blocks is dependent on soil gradation and their plasticity characteristics. In addition, it also depends on the clay type and the amount of clay content (Nagaraj et al. 2016). Water absorption capacity reduces with time for stabilized blocks. The decrease in water absorption of stabilized blocks is due to the interactions of cement with the alumino-silicates in the soil to form cementitious products that consequently bind the soil particles together and harden with time, thus reducing the interconnectivity of the voids (Oyelami and Van Rooy 2016). Lateritic soils are found to be suitable as materials for compressed earth blocks (CEB) with good compressive and durability strength, which qualifies them as sustainable and cost-effective materials for low-cost housing development (Reddy et al. 2007). Density significantly affects the strength and durability properties of CSEBs (Muntohar 2011). In the sand–clay matrix, since the sand is coarser and the clay finer, the clay particles will fill the void of the sand particles resulting in an increase in density and reduction of void spaces. The CSH gel formed fills the void spaces and results in a more impermeable structure, thereby resulting in an effective binding of particles with significant improvement in strength (Oti et al. 2009). Hydration reactions take place when cement is blended with soil in presence of water. The C_3S and C_2S present in cement react with water forming complex calcium silicate hydrates. This CSH gel produced will fill the void spaces and binds the soil particles together imparting rigidity to the mixture. When cement is blended with GGBFS and soil in the presence of water, the amount of gel formations increases and gel binds the particles more efficiently (Morel et al. 2007). Strength reduction on saturation is due to the softening of binders by water and development of pore water pressures. For an unstabilized soil block, the compressive strength when immersed in water is zero. The wet compressive strength is nearly half of the dry compressive strength for stabilized blocks (Oti et al. 2009).

Cement in the activated PC–GGBFS–soil mixture improves the strength by largely covering the clay particles with an insoluble and impermeable coating (Oti et al. 2009). Significant improvement in strength can be achieved by little addition of cement and the strength gain increases with cement content and curing time (Kaniraj and Havanagi 1999). Cement content >4% provides good strength and durability properties for CSEBs. The strength at 10% cement is almost double that of 4% cement (Tripura and

Singh 2015). Higher cement content leads to better stabilization and hence higher wet strength to dry strength ratio. Higher cement dosages lead to more cementitious material available to establish water insoluble bonds with the silt and sand particles and hence leads to higher strength for CSRE (Reddy and Kumar 2011). CSEBs stabilized with cement is found to have good strength and durability properties when compared with that of lime stabilized CSEBs (Holliday et al. 2016).

2.6 SUMMARY OF LITERATURE REVIEW

A review of literature revealed that many researchers have worked on the topic stabilization of soils and their contributions have great significance in the engineering field. Different laboratory tests and geotechnical investigations have been carried out to find the influence of various admixtures on the soil behaviour. Studies have been carried out on the stabilization of problematic soils using various admixtures such as cement, lime, fly ash and other industrial waste products. Portland cement was found to be one of the successful admixtures for stabilization from the literature. Nowadays, stabilization with industrial by products has also attained greater significance. Blast furnace slag having good pozzolanic character was one among them. Stabilization using ground granulated blast furnace slag (GGBFS) were studied in the past. However, detailed studies were not carried out on the stabilization of lithomargic clay with granulated blast furnace slag (GBFS) and its geotechnical applications. It was also observed that no study was carried out on the use of lithomargic clay, granulated blast furnace slag (GBFS) and cement in the manufacture of CSEBs and hence motivated us to take up the present study. Thus, this study is mainly focused on the behaviour of lithomargic clay stabilized using GBFS and cement and its application to typical engineering problems.

2.7 LIST OF DRAWBACKS

1. Experimental studies on soil stabilization were mainly carried out and very few studies were carried out with respect to its applications.
2. Detailed studies were limited to mainly strength properties and studies on gain in strength with time with industrial waste as stabilizers is limited.

3. No study was reported on stabilization of lithomargic clay using granulated blast furnace and cement.
4. The usage of lithomargic clay along with granulated blast furnace slag and cement in the manufacture of compressed stabilized earth blocks is not explored.

CHAPTER 3

MATERIALS AND METHODOLOGY

3.1 GENERAL

A geotechnical engineer aims at collecting and classifying soil samples and investigating for its geotechnical properties. In any project, evaluation of the index properties, compaction, shear and settlement characteristics is one of the important step. Several tests are conducted to determine the properties of both unstabilized and stabilized soils. The materials used and the methodology adopted for the various tests are clearly depicted in this chapter. All the tests were conducted as per the Bureau of Indian Standards.

3.2 MATERIALS USED

3.2.1 Lithomargic clay

The lithomargic clay also known as shedi soil is widely available weak soil in the southwestern part of India along the Konkan coast. For the present study, the soil samples were collected from a site at Padupanambur, Mangalore which is located in the South Canara district of Karnataka state, India. Lithomargic clay was dried, pulverized and sieved properly as per the requirements. Initially all the basic geotechnical tests were conducted for the soil alone. Experiments to determine the chemical properties such as pH, electrical conductivity, silica content etc. of the soil were also conducted. Then stabilization of lithomargic clay with granulated blast furnace slag and cement were done separately. Lithomargic clay was replaced by GBFS in different proportion by dry weight of the soil i.e. 5%, 15%, 25%, 35%, and 45%. Cement was added in different percentages of 2, 4, 6 and 8% by dry weight of the soil. For each trial, uniform mixing was ensured to study the geotechnical properties of the stabilized soil. Finally, experiments were conducted to determine the strength parameters of the soil stabilized with optimum amount of GBFS and varying percentages of cement. The testing of stabilized soil was carried out after proper mixing and curing.

Fig.3.1 Lithomargic clay sample

3.2.2 Granulated blast furnace slag (GBFS)

The GBFS is a byproduct obtained from the iron industry. It can be used effectively as a stabilizer to improve the soil properties. GBFS for the present study was collected from the industrial area at Kirloskar Ferrous Industries Limited, Bevinahalli, Koppal district located in the northern part of Karnataka state, India. For GBFS alone, basic geotechnical tests and some of the chemical tests to determine the pH, electrical conductivity, loss of ignition, the amount of silica, alumina, oxides of calcium etc. were carried out. Then various geotechnical tests were conducted on soil-GBFS mixtures in which percentages of GBFS replacing the soil was varied from 5% to 45% of dry weight of the soil. Strength properties of stabilized soil was determined after different curing periods.

Fig. 3.2 Granulated blast furnace slag (GBFS) sample

3.2.3 Cement

Cement is effective in improving the engineering properties of a wide variety of soils, including granular materials, silts and clays. Ordinary Portland Cement was used as a binding material in this study. Cement was added as an additive to the lithomargic clay in varying amounts of 2, 4, 6 and 8% by dry weight of the soil. Geotechnical tests were conducted to determine the index properties, compaction characteristics, unconfined compressive strength, CBR values etc. of the cement-stabilized soil. Finally, cement was added to the optimum slag stabilized soil and tests to determine the strength parameters were carried out. Since it needs time for the pozzolanic reaction to occur, samples were cured for 7 and 28 days and tested.

3.3 LABORATORY TESTS CONDUCTED

The following tests were conducted to determine the geotechnical parameters of the unstabilized and stabilized soil:

- Grain size analysis- sieve analysis and hydrometer analysis

- Atterberg’s limits tests
- Specific gravity test
- Standard Proctor test
- Unconfined compressive strength test
- California bearing ratio test
- Triaxial compression test- UU test
- Permeability test

For basic soil, tests to determine chemical characteristics such as pH, electrical conductivity, silica content etc. were also carried out.

To determine the chemical characteristics of GBFS the following tests were carried out:

- pH value test
- Electrical conductivity analysis
- Loss on ignition test
- Silica test
- Combined alumina and ferric oxide test
- Ferric oxide test
- Alumina test
- Calcium oxide test
- Magnesia test

3.4 TEST PROCEDURE

Experiments to determine the geotechnical properties of unstabilized and stabilized soils were conducted as per IS 2720 and SP 36. Both index and strength properties were determined. Tests were also conducted to determine the chemical characteristics of granulated blast furnace slag and study soil. These tests were carried out according to IS 2720 (Part 26)-1987 Reaffirmed 2002 and IS 1727-1967 (Reaffirmed 2004). The procedure adopted for all these tests are briefly explained below in the subsequent sections.

3.4.1 Grain size analysis

The test was carried according to IS: 2720 (Part 4)-1985 (Reaffirmed 2006). Soil was sieved through a set of sieves and the weights of the material retained on different sieves were determined. This can be used for determining the distribution of coarser fraction in the soil sample. To determine the distribution of fine particles, hydrometer analysis is carried out. Soil sample is washed through IS 75 micron sieve and the soil retained on the sieve was oven dried. The dried out sample was again sieved through the sieve set specified by the standards and 50 gram of dried sample from the washout (passing 75 micron sieve) was used for hydrometer analyses. The hydrometer measures the unit weight of the soil suspension, which depends on the mass of solids present that in turn depends on the particle size.

3.4.2 Atterberg's limit tests

Liquid limit and plastic limit tests are used to distinguish between silt and clay. Tests were conducted to determine the liquid limit, plastic limit and shrinkage limit of soil samples passing through 425μ IS sieve as per IS 2720 (Part 5)–1985 (Reaffirmed 2006). The liquid limit was determined with the aid of the standard mechanical liquid limit device, designed by Arthur Casagrande. It is the water content at which, grooves cut in a pat of soil by a grooving tool of standard dimensions will flow together for a distance of 13 mm, under 25 blows in the device. From the plot of water content versus log number of blows, the liquid limit was determined corresponding to 25 numbers of blows. Plastic limit corresponds to the water content at which the soil when rolled into 3 mm diameter thread just begins to crumble.

3.4.3 Specific gravity test

The ratio of weight of given volume of soil solids to the weight of an equal volume of water gives the specific gravity value. Great care must be taken to expel all the entrapped air inside the soil while doing the test. The test follows procedure as per IS 2720-1980 (Part 3) (Reaffirmed 2002).

3.4.4 Standard Proctor test

Standard Proctor test also known as light compaction test was carried out to assess the amount of compaction and water content required in the field. The degree of compaction is measured in terms of dry unit weight and it is maximum at the optimum moisture content. A curve was drawn between the water content and the dry unit weight to obtain the maximum dry unit weight and the optimum moisture content. The equipment and test procedures adopted are as per IS 2720 (Part 7)-1980 (Reaffirmed 2011).

3.4.5 Unconfined compressive strength test

The unconfined compressive strength is the load per unit area at which the cylindrical specimen of a soil fails in compression. This test gives the unconfined compressive strength of soil. UCC tests were performed on statically compacted samples at maximum dry unit weights and optimum moisture contents as obtained from standard proctor test. The unconfined compressive strength is taken as the maximum load attained per unit area, or the load per unit area at 20% axial strain, whichever occurs earlier while conducting the test. The test was conducted according to IS 2720 (Part 10)-1991 (Reaffirmed 2006).

3.4.6 California Bearing Ratio (CBR) test

The CBR is defined as the force per unit area required to penetrate a soil mass with standard circular piston at the rate of 1.25mm/minute. The CBR test is conducted to evaluate the suitability of the sub grade and the materials used in sub-base and base of a flexible pavement. The plunger penetrates the specimen in the mould at the rate of 1.25 mm per minute. The loads required for a penetration of 2.5 mm and 5 mm were determined. The penetration load was expressed as a percentage of the standard loads at the respective penetration level of 2.5 mm or 5 mm. To replicate worst conditions in the field, the samples were kept soaked in water for 4 days before testing. The test procedure and equipment used were as specified in the IS 2720 (Part 16)-1987 (Reaffirmed 2002).

3.4.7 Triaxial compression test –UU test

Triaxial compression tests are conducted to determine the shear strength parameters of the soil sample. The specimen is subjected to three compressive stresses in mutually perpendicular directions; one of the three stresses (vertical stress) was increased until the specimen fails. In the case of Unconsolidated Undrained (UU) test, no drainage was permitted during the application of axial stress or all-round pressure. The UU test is also referred as the quick test because it is a relatively fast test. The stabilized soil samples were cured for 7 days and 28 days and the UU tests were carried out as per IS 2720 (part 11) 1993 (Reaffirmed 2002).

3.4.8 Permeability test

Tests were carried out to determine coefficient of permeability. The test apparatus and procedure adopted are according to IS 2720 (Part 17) 1986 (Reaffirmed 2002).

3.4.9 Test to determine pH value

The acidic or alkaline characteristics of a soil sample can be quantitatively expressed by hydrogen ion-activity commonly designated as pH. The pH is measured electrometrically by means of an electrode assembly consisting of one glass electrode and one calomel reference electrode with a saturated potassium chloride solution. The pH meter directly gives the pH value. The samples were sieved and tested as per IS 2720 (Part 26)-1987 (Reaffirmed 2002). The pH of GBFS was established by this electrometric method.

3.4.10 Electrical conductivity analysis

Electrical conductivity is an indirect measurement that correlates well with several soil physical and chemical properties. It is the ability of the material to conduct (transmit) an electric current and is commonly expressed in micro Siemens. Using conductivity meter, it can be directly measured in micro Siemens. This test was conducted as per IS 14767-2000.

3.4.11 Loss on ignition test

Loss on ignition test was conducted for GBFS according to the procedure mentioned in IS 1727-1967 (Reaffirmed 2004). The percentage loss on ignition is reported nearest to 0.1 as the ratio of loss in weight to the weight of moisture free sample used.

3.4.12 Test to determine silica, alumina, iron oxide, calcium oxide and magnesia

The various chemical compounds present in GBFS were determined as per the procedure laid down by IS 1727- 1967 (Reaffirmed 2004). The amount of silica, alumina, iron oxide, calcium oxide and magnesia present in it were determined through a series of chemical tests. The silica content in lithomargic clay was also determined.

CHAPTER 4

LABORATORY TEST RESULTS AND DISCUSSION

4.1. GENERAL

In the present study, experimental investigation was carried out on the lithomargic clay, granulated blast furnace slag (GBFS) and lithomargic clay stabilized using GBFS and cement. To understand the benefits obtained from the stabilization, several tests were conducted on both unstabilized and stabilized lithomargic clay. All the test results are analyzed and presented systematically in this chapter.

4.2. PROPERTIES OF LITHOMARGIC CLAY

The lithomargic clay present below the weathered laterite is problematic as its strength reduces drastically upon saturation. In this work, an attempt is made to study the improvement in the behavior of the study soil (lithomargic clay) after being blended with GBFS and cement. The results obtained from various tests are presented in tabular and graphical form. Test results obtained for the lithomargic clay are presented in Table 4.1.

Table 4.1 Properties of lithomargic clay (shedi soil)

Sl. No.	Properties	Particulars
1	Particle size distribution Gravel size (%) Sand size (%) Silt size (%) Clay size (%)	 2 10 59 29
2	Atterberg's limits Liquid limit (%) Plastic limit (%) Shrinkage limit (%) Plasticity index (%)	 47 31 28 16
3	IS Classification	MI
4	Specific Gravity	2.52
5	Compaction characteristics Maximum dry unit weight (kN/m^3) Optimum moisture content (%)	 14.2 28
6	Unconfined compressive strength (kPa)	232
7	Strength parameters Cohesion c_{UU} (kPa) Angle of internal friction ϕ_{UU} (degrees)	 23 19
8	Coefficient of Permeability k (m/day)	0.00319
9	CBR Unsoaked (%) Soaked (%)	 14.5 2.9
10	pH	5.3
11	Electrical conductivity (µ Siemens)	60.2
12	SiO_2 (%)	76
13	Al_2O_3 (%)	19.2
14	Fe_2O_3 (%)	0.81
15	CaO (%)	2.19

The grain size distribution curve for lithomargic clay is shown in figure 4.1. The particle size distribution shows that major portion of the soil constitutes of fine-grained soil particles, silt size of 59% and clay size of 29%. The remaining 12% constitutes coarser particle size (Gravel size=2%, Sand size=10%).

4.3 PROPERTIES OF GRANULATED BLAST FURNCAE SLAG

The laboratory test results obtained from the basic geotechnical tests and some chemical tests on GBFS are presented in the Table 4.2.

Table 4.2 Properties of granulated blast furnace slag (GBFS)

Sl. No.	Properties	Particulars
1	Specific gravity	1.73
2	Grain size distribution	
	Gravel size (%)	01
	Coarse sand size (%)	14
	Medium sand size (%)	43
	Fine sand size (%)	42
3	Strength parameters	
	Cohesion (kPa)	4
	Angle of internal friction ϕ (degrees)	42
4	pH	9.2
5	Electrical conductivity (µ Siemens)	176
6	Calcium oxide (%)	41
7	SiO_2 (%)	35.3
8	Fe_2O_3 (%)	0.17
9	Al_2O_3 (%)	13.5
10	MgO (%)	7.2
11	Loss on ignition (%)	0.83

The grain size distribution curve for GBFS is included in figure 4.1. It showed that GBFS has predominantly medium and fine sand sized particles.

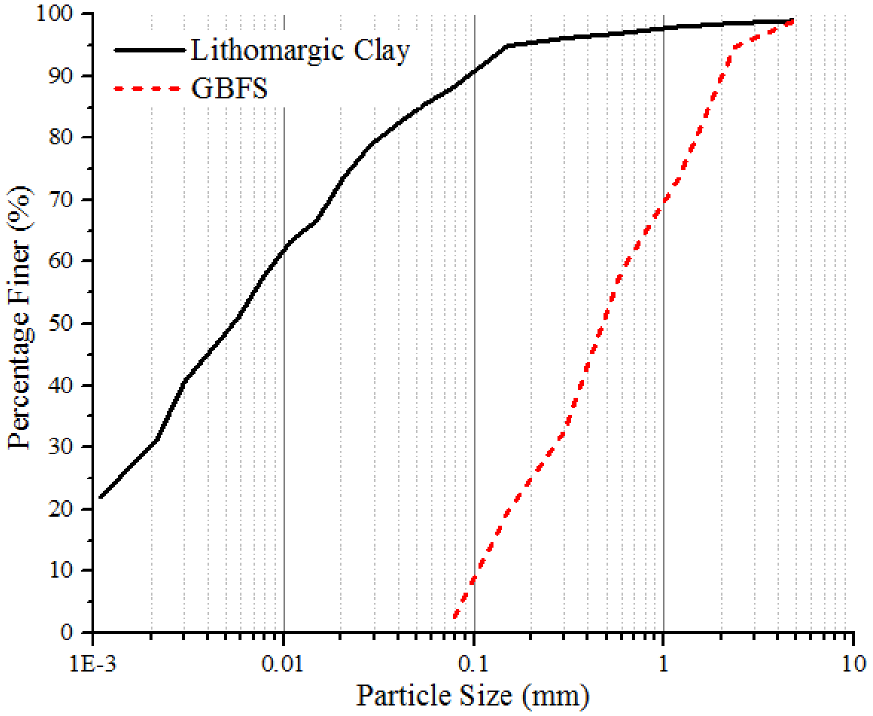

Fig. 4.1 Particle size distribution curves for both lithomargic clay and GBFS

4.4. GEOTECHNICAL PROPERTIES OF LITHOMARGIC CLAY STABILIZED WITH GBFS

In this study, the lithomargic clay was blended with granulated blast furnace slag (GBFS) and the improvement in properties were analyzed. The soil was replaced with GBFS in different proportions of 5%, 15%, 25%, 35% and 45% by dry weight of the soil. The results obtained from the tests are presented in tabular columns. All the relevant graphs are plotted and the variation in the properties of the stabilized soil is analyzed and discussed. The summary of the test results of lithomargic clay before and after stabilization with GBFS is presented in Table 4.3.

Table 4.3 Geotechnical properties of lithomargic clay before and after stabilization with GBFS

Sl. No.	Properties	Percentage of GBFS replacing soil					
		0%	5%	15%	25%	35%	45%
1	Liquid limit w_L (%)	47	43	38	37	35	33
2	Plastic limit w_P (%)	31	29	26	25	NP	NP
3	Plasticity Index I_P (%)	16	14	12	12	-	-
4	Specific gravity G	2.52	2.46	2.38	2.30	2.25	2.21
5	Maximum dry unit weight γ_d (kN/m^3)	14.2	14.3	14.3	14.2	14.2	14.1
6	Optimum moisture content (%)	28	27	25.5	24	22	19
7	Soaked CBR (%)	2.9	4.1	6.2	11.9	17	18.3

Typical plots of liquid limit determination from Casagrande's method are shown in figures 4.2 and 4.3 for lithomargic clay and lithomargic clay with 25% GBFS replacement respectively. From Table 4.3 and figure 4.4, it is observed that the liquid limit of the soil-GBFS mixes reduced with increasing replacement of GBFS to the study soil. This is due to the replacement of soil particles with that of a non-plastic coarser material i.e. GBFS to the study soil. The decrease in the specific gravity was due to the lower specific gravity of the GBFS when compared to lithomargic clay. The specific gravity reduced from 2.52 to 2.21 at 45% replacement (figure 4.5). Typical plots of maximum dry unit weight versus and water content are shown in figures 4.6 and 4.7 for lithomargic clay and lithomargic clay with 25% GBFS replacement respectively. No major variation in the maximum dry unit weight of the mix was observed because the GBFS particles due to its size and rough texture resisted the compaction effort. The

OMC reduced because of the replacement of soil with that of GBFS, which has lower water holding capacity than the study soil. The percentage reduction in OMC was found to be 32% when soil was replaced by 45% GBFS as shown in figure 4.8 (Sharma and Sivapullaiah 2012). Also from the figure 4.9, it was observed that there is a good improvement in soaked CBR values, with increase in percentage of GBFS replacement to the soil. The percentage improvement at 45% GBFS replacement to study soil is found to be 531%.

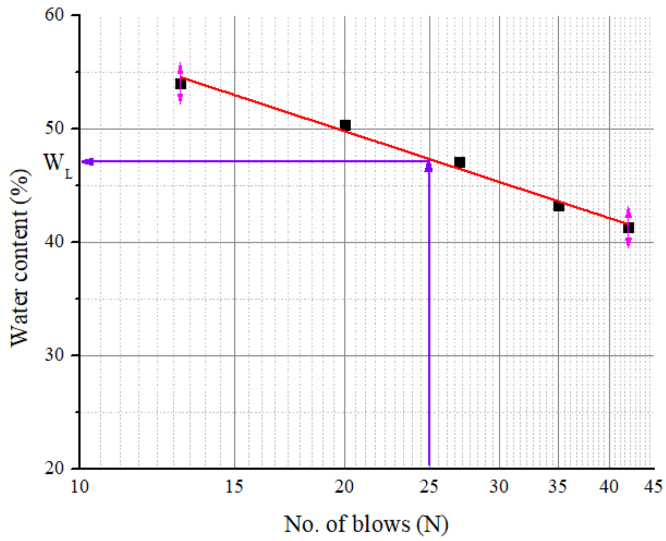

Fig. 4.2 Plot of water content Vs no. of blows for lithomargic clay

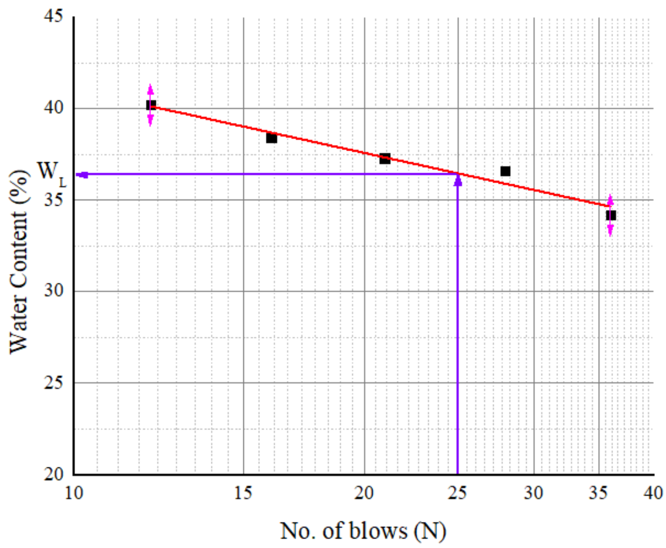

Fig. 4.3 Plot of water content Vs no. of blows for lithomargic clay with 25% GBFS replacement

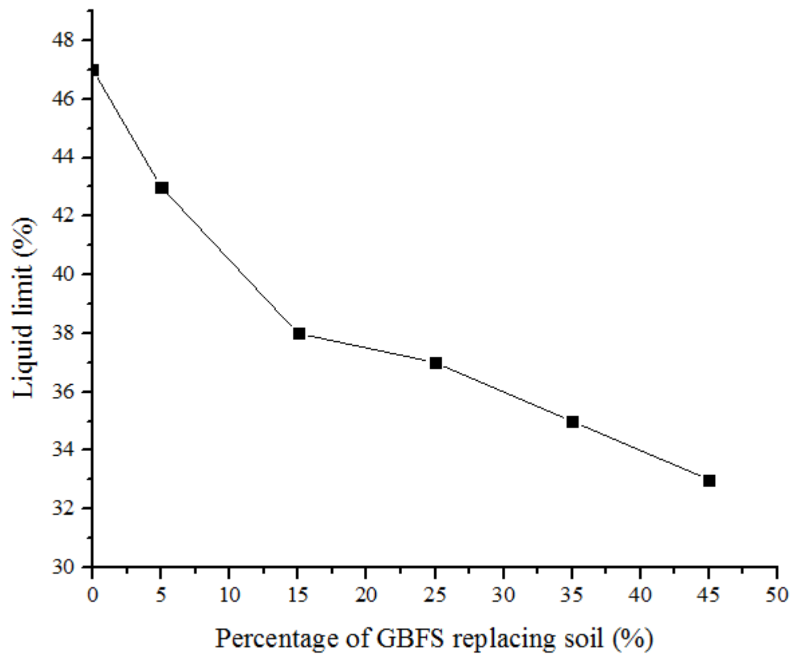

Fig. 4.4 Variation of liquid limit with increasing percentage of GBFS replacing soil

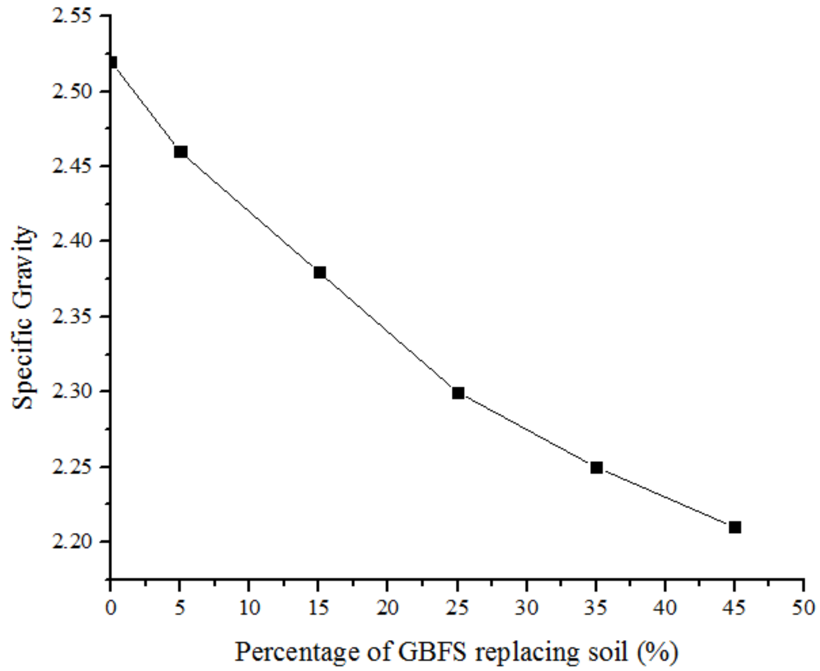

Fig. 4.5 Variation of specific gravity with increasing percentage of GBFS replacing soil

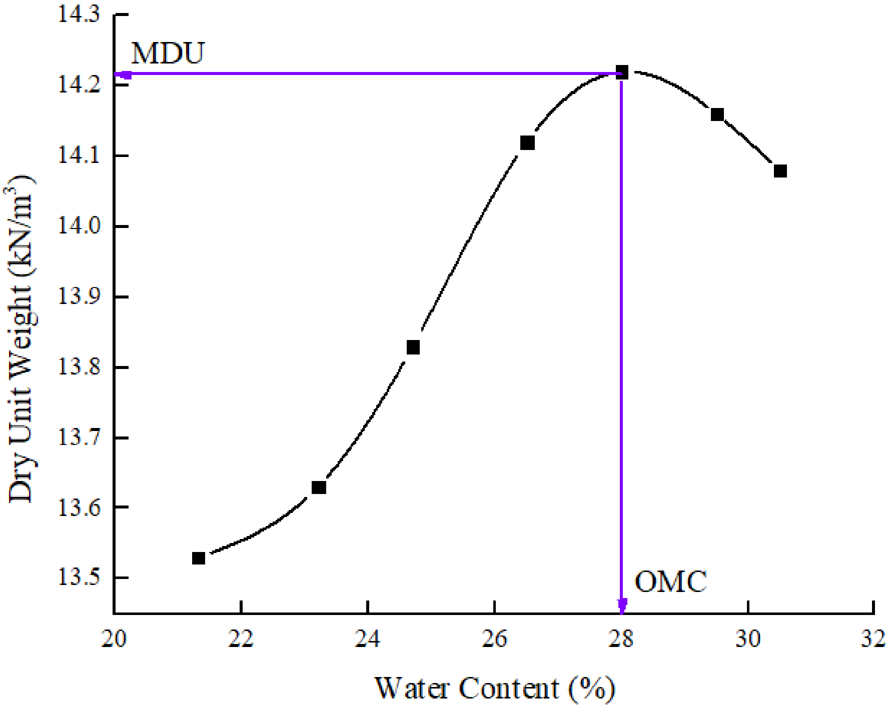

Fig. 4.6 Plot of maximum dry unit weight Vs water content for lithomargic clay

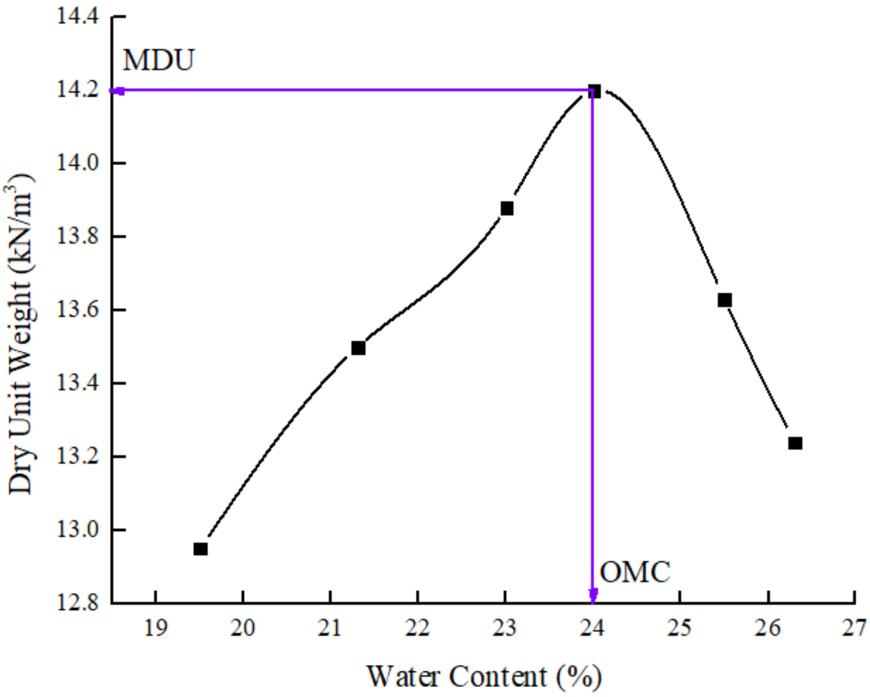

Fig. 4.7 Plot of maximum dry unit weight Vs water content for lithomargic clay with 25% GBFS replacement

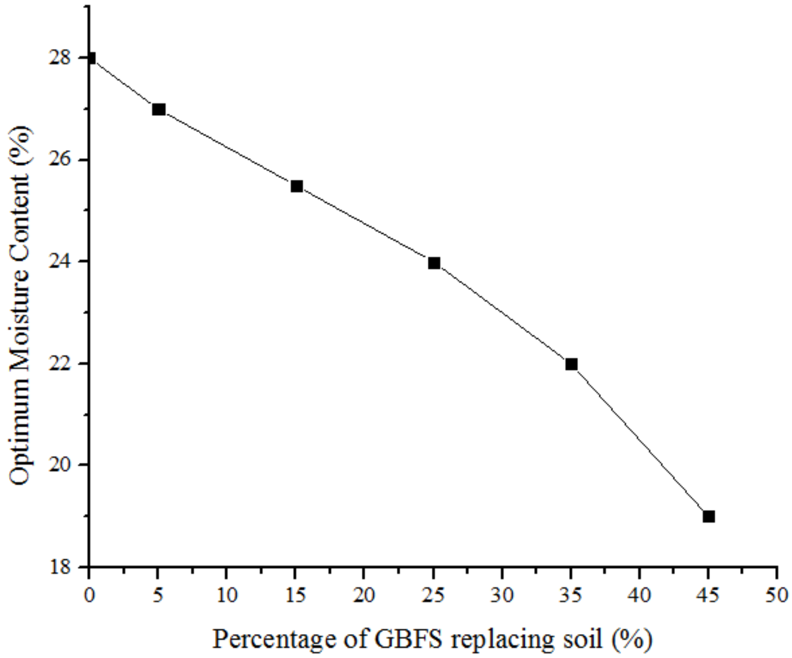

Fig. 4.8 Variation of OMC with increasing percentage of GBFS replacing soil

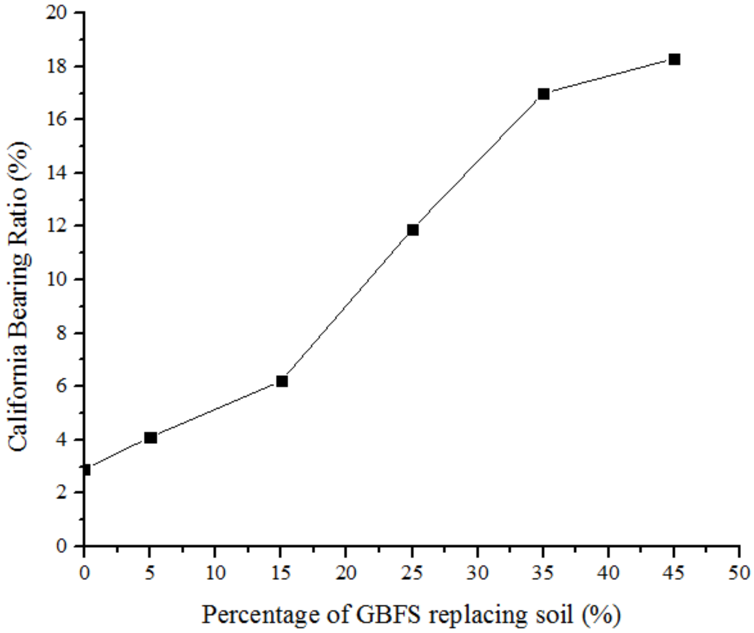

Fig. 4.9 Variation of soaked CBR with increasing percentage of GBFS replacing soil

Table 4.4 Variation of UCS with percentage of GBFS replacing soil

Curing period	UCC Strength(kPa)					
	Percentage of GBFS replacing soil					
	0%	5%	15%	25%	35%	45%
0 days	232	249	242	238	211	161
7 days	232	322	334	385	308	252
14 days	232	393	411	523	443	338
28 days	232	581	604	712	638	477

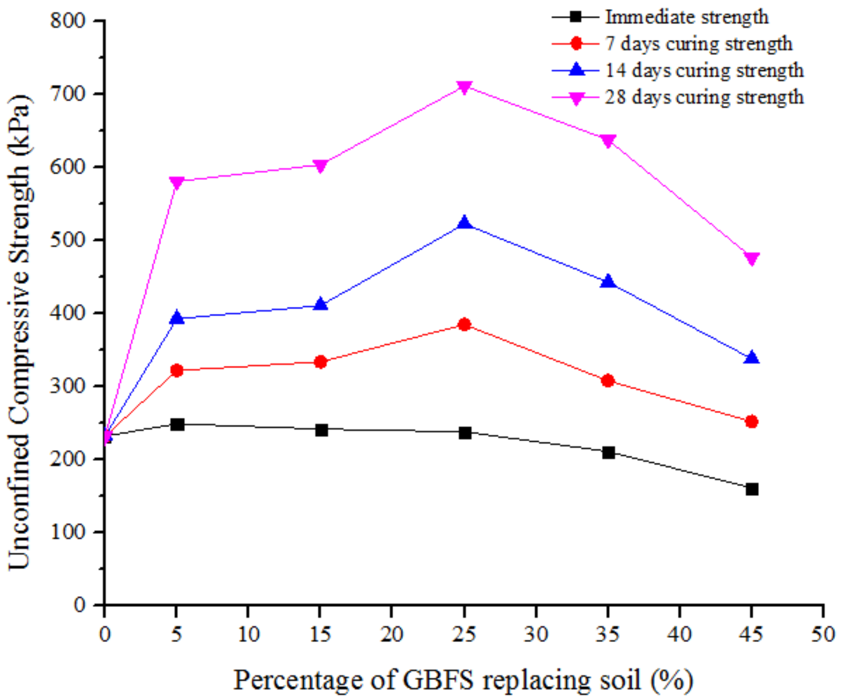

Fig. 4.10 Variation of UCS with increasing percentage of GBFS replacing soil

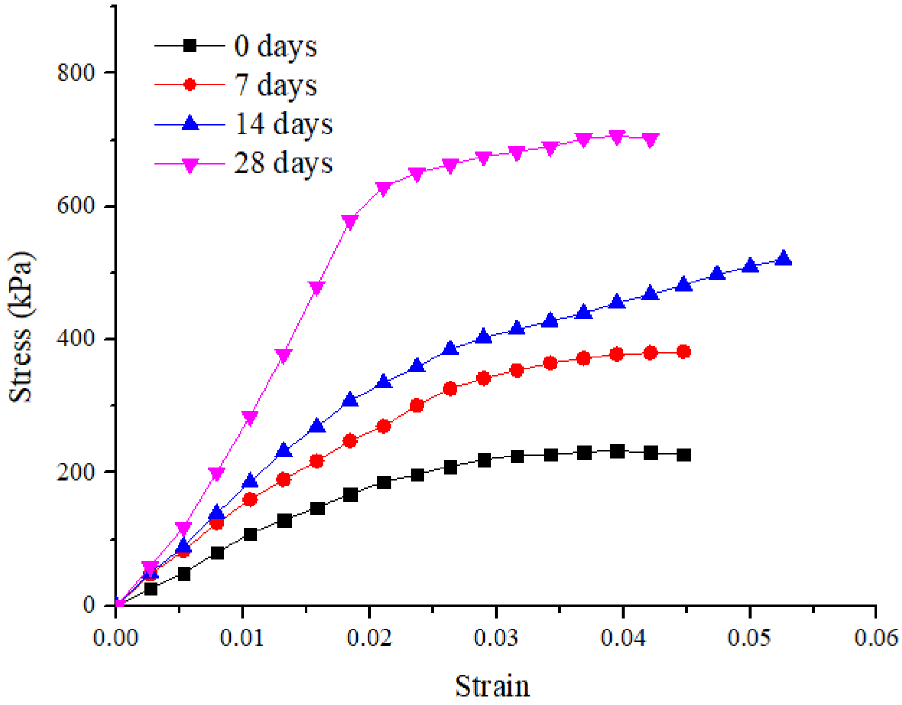

Fig. 4.11 Plot of stress versus strain for lithomargic clay with 25% GBFS replacement for varying curing periods.

Variation of UCS with different percentage of GBFS replacing soil (for curing period of 0,7,14 and 28 days) is shown in Table 4.4 and figure 4.10. From figure 4.10, it is observed that, for immediate testing, there is no significant improvement in UCS value with the replacement of soil by GBFS up to 25% and for 35% and 45% replacement, the UCS value decreased considerably. This is due to the fact that, adding a coarser (non-cohesive) material (GBFS) will reduce the fines content in the soil-GBFS mix and the strength decreased due to the lack of confinement. With 7 days curing, the strength increased considerably until 25% replacement. This is due to the occurrence of pozzolanic reactions like CSH gel formation, caused by soil and slag. At 25% of slag replacing the soil, the strength increased from 232kPa to 385kPa (increase of 65.9%) at 7 days. Beyond 25% replacement of soil by GBFS, the UCS decreased. A similar trend was observed for higher period of cured samples (i.e. 14 days and 28 days). This is because although with increase in GBFS content, the availability of pozzolanic material increases, the water available for pozzolanic reactions becomes less due to decreasing water content. The maximum UCS is observed for 25% soil replaced by GBFS which is 712kPa at 28 days curing period. An increase of 206.9% (232kPa to 712kPa) is observed in UCS at 28 days curing for lithomargic clay replaced by 25% GBFS. Stress-strain curves for lithomargic clay with 25% GBFS replacement at varying curing periods are shown in figure 4.11. We can observe that the slope of the stress-strain curves becomes steeper with increasing curing periods. Thus lithomargic clay replaced by 25% GBFS is found to be optimized mix.

Table 4.5 Variation of c and φ with increasing percentage of GBFS replacing soil.

Properties obtained from triaxial UU test		Percentage of GBFS replacing soil					
		0%	5%	15%	25%	35%	45%
0 days	Cohesion c_{UU} (kPa)	23	22	21	20	19.5	18
0 days	Angle of internal friction ϕ_{UU} (degrees)	19	20	22	26	27.5	30.5
7 days	Cohesion c_{UU} (kPa)	23	29	43	59	53	42
7 days	Angle of internal friction ϕ_{UU} (degrees)	19	21	24	27.5	29	32
28 days	Cohesion c_{UU} (kPa)	23	40	61	83	75	64
28 days	Angle of internal friction ϕ_{UU} (degrees)	19	22.5	26	29	31	33

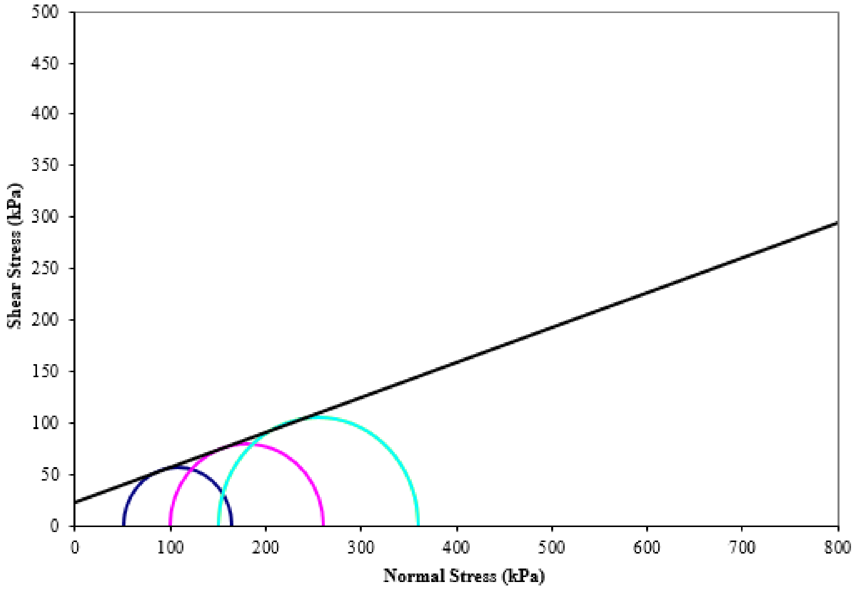

Fig. 4.12 Plot of Mohr circle and failure envelope for lithomargic clay

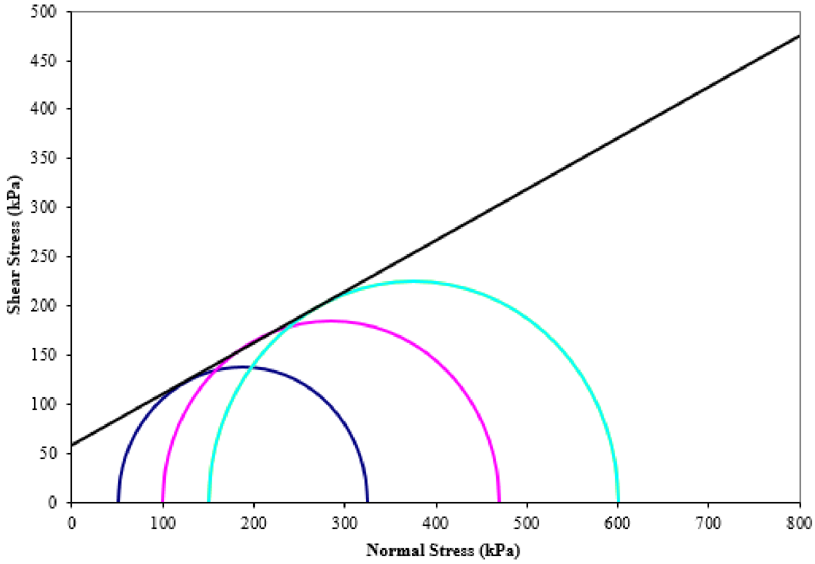

Fig. 4.13 Plot of Mohr circle and failure envelope for lithomargic clay with 25% GBFS replacement at 7 days curing period.

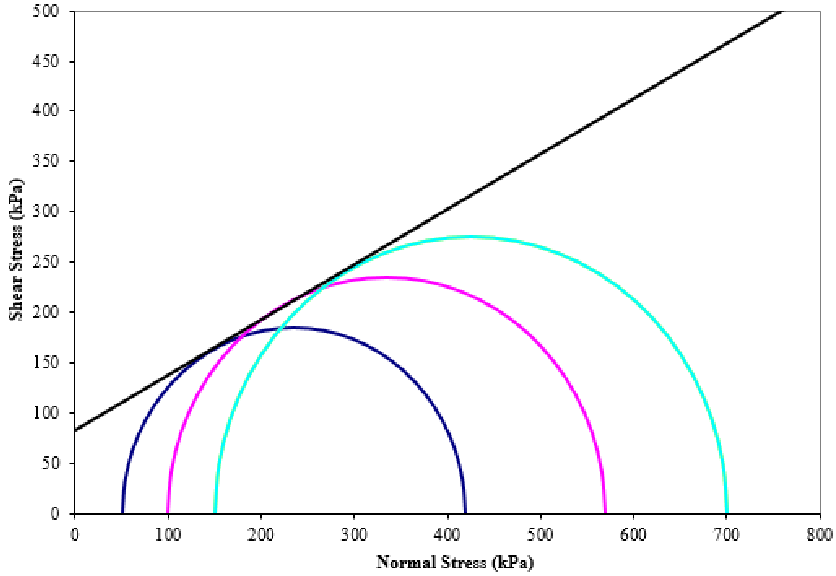

Fig. 4.14 Plot of Mohr circle and failure envelope for lithomargic clay with 25% GBFS replacement at 28 days curing period.

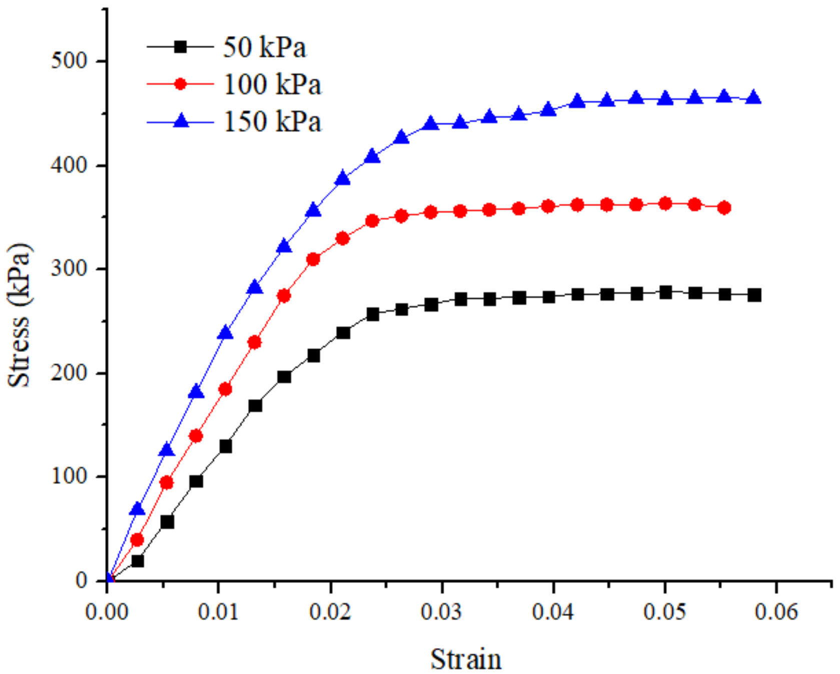

Fig. 4.15 Plot of stress versus strain for lithomargic clay with 15% GBFS replacement for different confining pressures

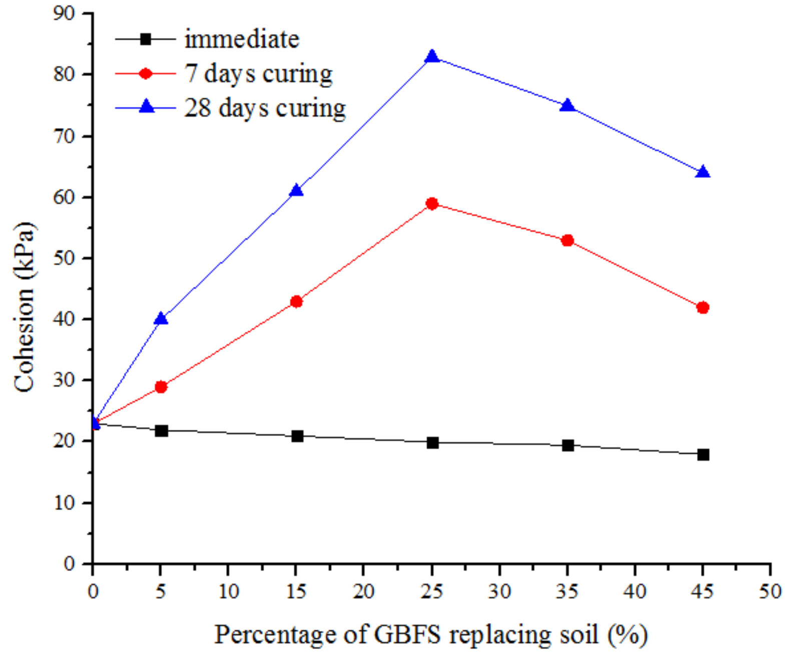

Fig. 4.16 Variation of cohesion with increasing percentage of GBFS replacing soil

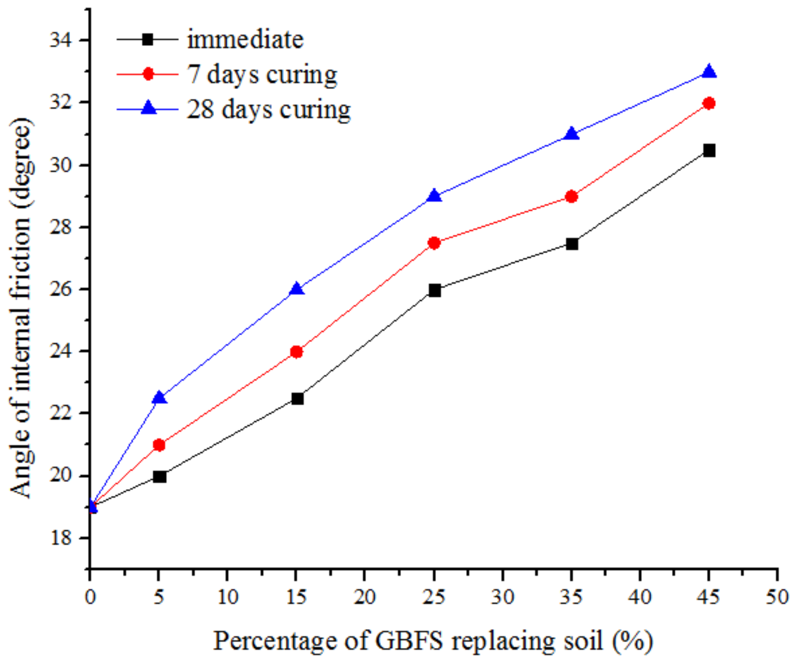

Fig. 4.17 Variation of frictional angle with increasing percentage of GBFS replacing soil.

Typical plots of Mohr circle with failure envelope are shown in figures 4.12-4.14. In addition, a typical plot of stress strain curves is shown in figure 4.15 for lithomargic clay with 15% GBFS replacement. From the figure 4.16, it is clear that at immediate testing cohesion reduced with increasing percentage of replacement of soil by GBFS. However, at 7 days and 28 days cured samples, the cohesion increased up to 25% replacement of soil by GBFS and further decreased at higher percentages (35% and 45%) of replacement of soil by GBFS. For 7 days cured samples, increase in cohesion when lithomargic clay is replaced by 25% GBFS is found to be 156.5% (i.e. 23kPa to 59kPa), whereas at 28 days curing, it is 260.9% (23kPa to 83kPa). This is due to the fact that pozzolanic reactions increase up to 25% and beyond this, the cohesion decreased because the water required for the hydration of the pozzolanic reactions is insufficient and hence the reduced reactions. The excess slag particles remains redundant (Yi et al 2014). From figure 4.17, we can see an increase in the frictional angle for all the combinations due to pozzolanic reactions. In addition, the granulated blast furnace slag particles have good interlocking and rough surface, which leads to the increase in friction angle. At 7 days curing frictional angle increased from $19^{\circ}$to $27.5^{\circ}$when soil is replaced by 25% GBFS and increase is found to be from $19^{\circ}$to 29° for 28 days cured samples.

4.5. GEOTECHNICAL INVESTIGATIONS OF LITHOMARGIC CLAY BLENDED WITH CEMENT

Cement is universally used admixture to improve the properties of the soil. Cement stabilization has attained a wider acceptance all over the world. The lithomargic clay was blended with cement as an additive in which cement was added in different percentages such as 2%, 4%, 6% and 8% by dry weight of the soil. These soils mixed with cement were tested to get various geotechnical properties. The summary of the laboratory test results are presented in Table 4.6 and figures 4.18 - 4.22.

Table 4.6 Geotechnical properties of lithomargic clay blended with cement

Sl. No.	Properties	Percentage of cement added to soil				
		0%	2%	4%	6%	8%
1	Liquid limit w_L (%)	47	46	45	43.7	42.4
2	Plastic Limit w_p (%)	31	31.5	32.3	33.2	34
3	Plasticity Index I_p (%)	16	14.5	12.7	10.5	8.4
4	Max Dry Unit Weight γ_d (kN/m^3)	14.2	14.2	14.3	14.4	14.6
5	Optimum Moisture Content (%)	28	26	26	25.8	25.8
6	Unconfined Compressive Strength (kPa)	232	343	512	623	779
7	Soaked CBR (%)	2.9	22.4	53.1	65	79

A typical plot of liquid limit determination by Casagrande's method is shown in figure 4.18. With increase in addition of cement to lithomargic clay, liquid limit reduces and plastic limit increased thereby reducing the plasticity index of the soil-cement mix (figure 4.19). There is a reduction in plasticity index by 47.5% (i.e. 16% to 8.4%). A typical plot of water content versus dry unit weight is shown in figure 4.20. There is negligible variation in maximum dry unit weight, with increase in cement to the mix. This is because the proctor test was carried out before the commencement of hydration reactions. Stress-strain curves for lithomargic clay stabilized with varying percentage of cement is shown in figure 4.21. From Table 4.6 and figure 4.22, it is observed that there is an improvement in unconfined compressive strength with increasing percentage of cement after 7 days curing period. It was observed that as the cement content increased the Young's modulus increased. The 7 days cured UCS specimens and CBR test samples showed significant strength improvement with increasing addition of cement. Addition of 2% cement to lithomargic clay increased the compressive strength by 47.8% (from 232kPa to 343kPa). Further, at 4% cement addition, UCS increased from 232kPa to 512kPa (i.e. an improvement by 120.7%). This is due to the cement hydration, cation exchange, pozzolanic reaction, and carbonation. Hydration process is a process under which cement reaction takes place. The process starts when cement is mixed with water and other components for a desired application resulting into hardening phenomena. Cement hydration is a complex process affected by the presence of foreign matter or impurities, water cement ratio, curing temperature, specific surface of the mixture. Calcium silicates, C_3S and C_2S are the two main cementitious properties of ordinary Portland cement responsible for strength development. Calcium hydroxide is another hydration product of Portland cement that further reacts with pozzolanic materials available in stabilized soil to produce further cementitious material. $Ca(OH)_2$ in the soil water reacts with the silicates and aluminates in the soil to form a soil-cement mixture known as calcium aluminate silicate hydrate (CASH). As the pozzolanic reaction progresses, calcium aluminate silicate hydrate (CASH) is slowly converted into, a crystalline stage to form calcium silicate hydrate (CSH) and calcium aluminum hydrate (CAH) (Gruskovnjak et al. 2006, Song et al. 2000, Yi et al. 2015, Kolias et al. 2005, Jaritngami et al. 2014, Dermatas and Meng 2003).

In the presence of water, the calcium ions reduce the thickness of double diffused layer through cation exchange and flocculation-agglomeration reactions. This is primarily responsible for improvement in workability through reduction of adsorbed water and decrease in plasticity index. In long-term, pozzolanic reactions occur between the calcium ions of the stabilizer and the silica and alumina of the clay minerals resulting in the formation of cementitious products. The reactions can be written as:

$Ca(OH)_2$ (*ionization of lime*) $\rightarrow Ca^{2+} + 2(OH)^-$

$Ca^{2+} + OH^- + SiO_2$ (*soluble clay silica*) $\rightarrow$ Calcium-Silicate-Hydrate (C-S-H)

$Ca^{2+} + OH^- + Al_2O_3$ (*soluble clay alumina*) $\rightarrow$ Calcium-Aluminate-Hydrate (C-A-H)

These reactions contribute to the reduced plasticity and increase in shear strength properties

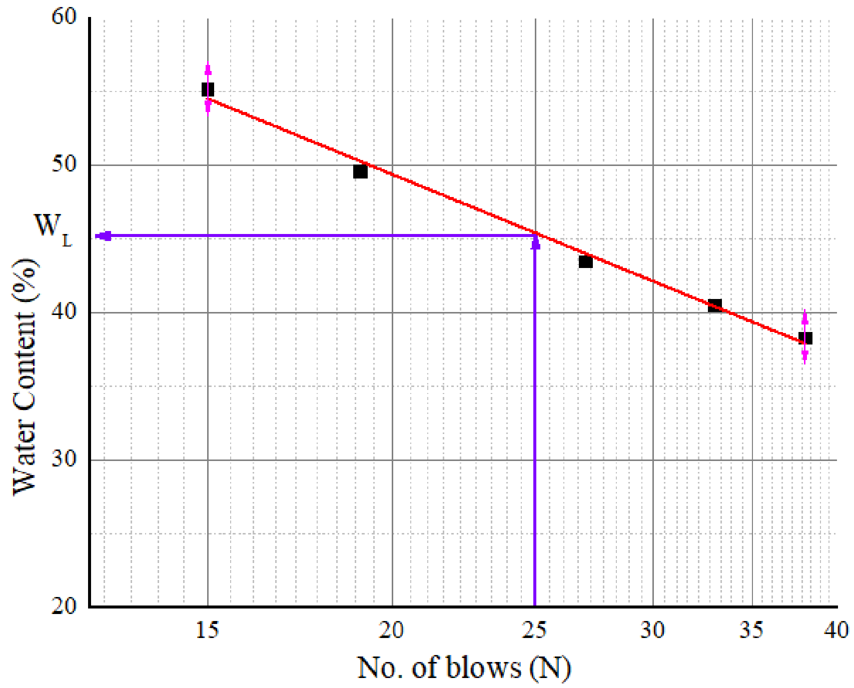

Fig. 4.18 Plot of water content versus no. of blows for lithomargic clay with 4% cement.

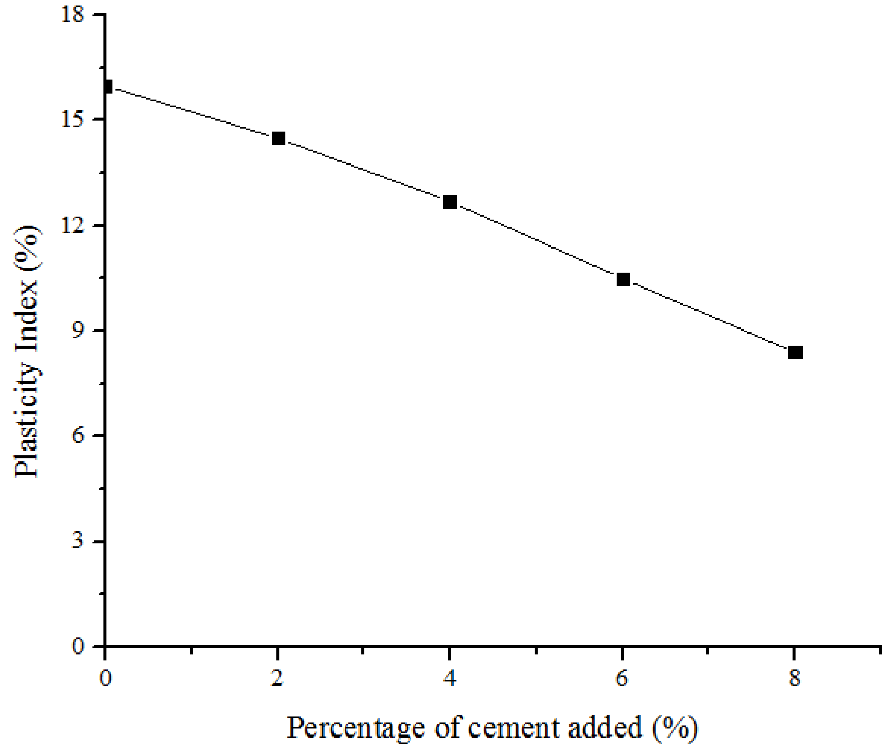

Fig. 4.19 Variation of plasticity index with increasing percentage of cement added.

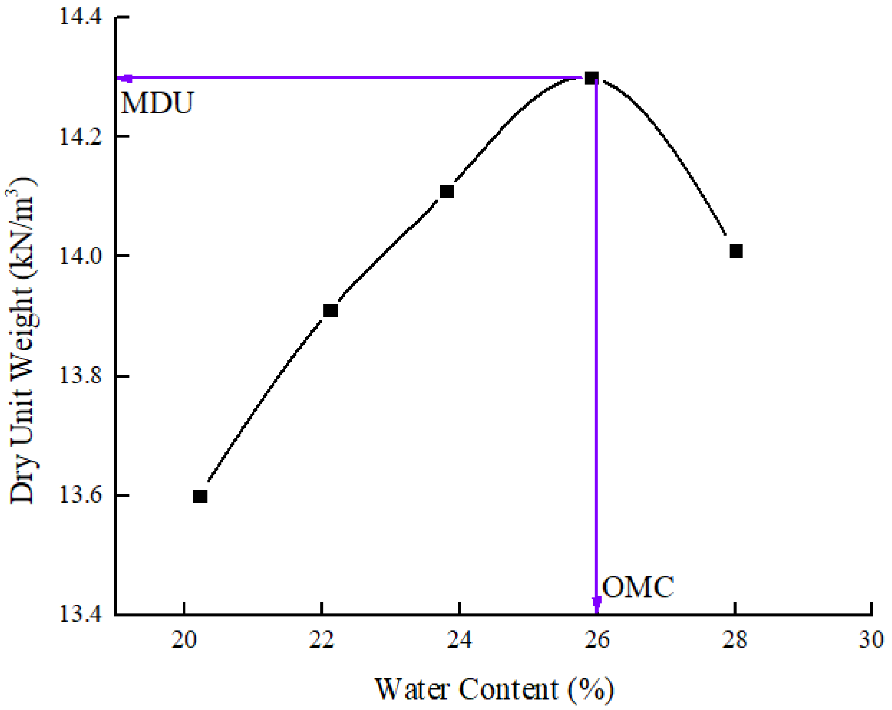

Fig. 4.20 Plot of dry unit weight versus water content for lithomargic clay with 4% cement.

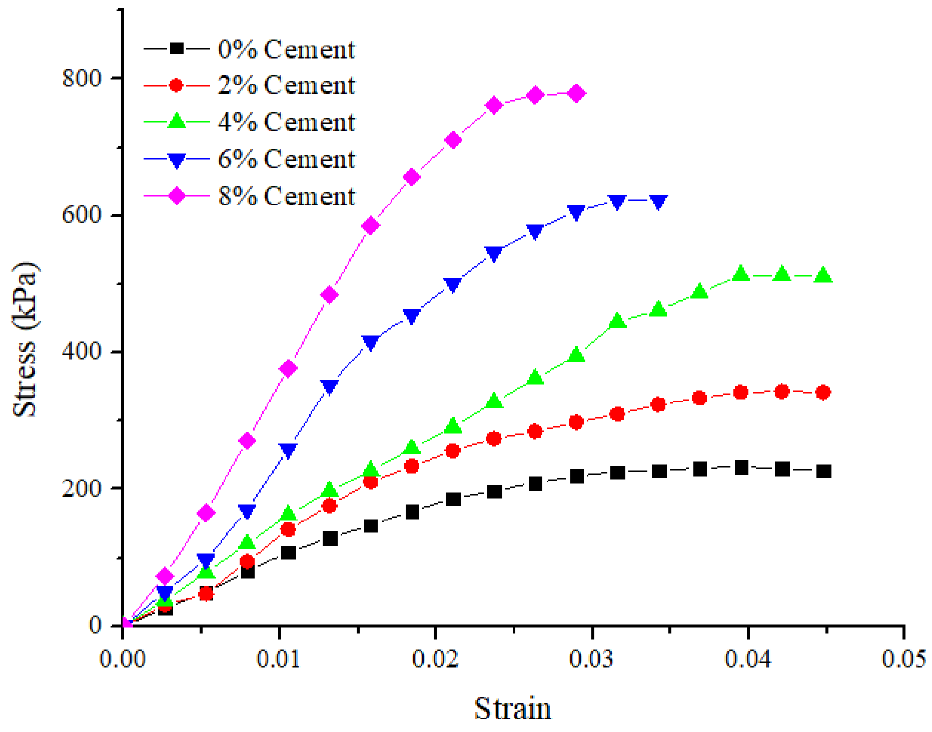

Fig.4.21 Plot of stress versus strain for lithomargic clay with varying cement content

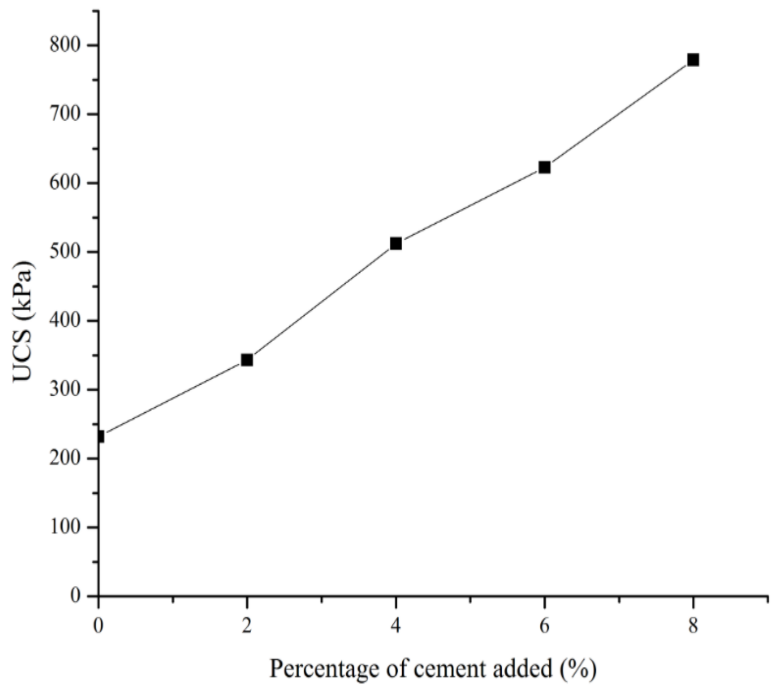

Fig. 4.22 Variation of UCS with different percentage of cement for 7 days curing period

4.6 EFFECT OF GBFS AND CEMENT ON PROPERTIES OF LITHOMARGIC CLAY

Triaxial compression tests and unconfined compression tests were conducted on 75% soil + 25% GBFS mix by adding 2% and 4% cement. Table 4.7 shows the variation of cohesion, frictional angle and UCS with the addition of 2% and 4% cement to the optimized combination of soil replaced with 25% GBFS for 7 and 28 days curing.

Table 4.7 Variation of shear strength parameters with different percentage of cement to optimized mix.

Curing days	Shear strength parameters from triaxial UU test	75% soil + 25% GBFS + varying percentage of cement		
		0% cement	2% cement	4% cement
7 days	Cohesion c_{UU} (kPa)	59	219	343
7 days	Angle of internal friction ϕ_{UU} (degrees)	27.5	40	42
7 days	UCS (kPa)	385	656	932
28 days	Cohesion c_{UU} (kPa)	83	246	383
28 days	Angle of internal friction ϕ_{UU} (degrees)	29	42	43
28 days	UCS (kPa)	712	1024	1378

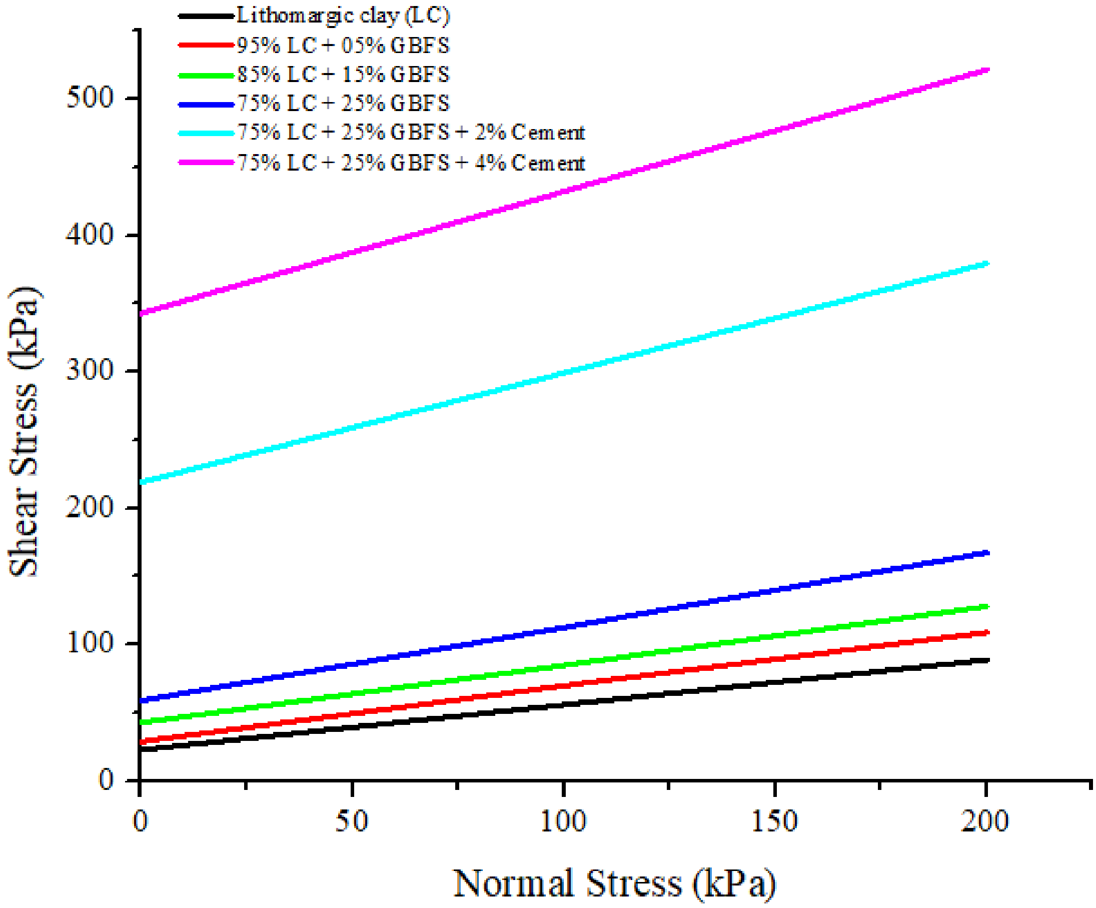

Fig. 4.23 Plot of failure envelopes for different stabilized soil combinations

Addition of cement to slag stabilized soils causes the alkalinity to activate and accelerate the pozzolanic properties. With 4% addition of cement to the soil-GBFS optimum mix, cohesion and angle of friction values are found to be 343kPa and 42° respectively at 7 days curing, whereas at 28 days curing, cohesion and angle of friction values are obtained as 383kPa and 43° respectively. Thus by adding 4% cement to optimum mix, cohesion increased by 481.3% at 7 days curing (from 59kPa to 343kPa) and by 361% at 28 days curing (from 83kPa to383kPa). The friction angle also increased by 52.7% at 7 days curing (from 27.5° to 42°) and 48.2% at 28 days curing (from 29° to 43°) at 4% addition to the optimum GBFS mix. Similarly, at 7 days curing UCS increased from 385kPa to 932kPa for 4% cement addition whereas for the same combination increase is found to be from 712kPa to 1378kPa at 28 days curing. The strength gain with cement to the optimized GBFS mix is due to the self hydration of cement as well as the increased alkalinity producing more pozzolanic products. The addition of a little amount of cement increases the shear strength significantly in less time. In cement triggered GBFS systems, there is a breakdown and dissolution of the glassy structure, as $Ca(OH)_2$ in the system is consumed and further creation of Calcium Silicate Hydrate gel takes place (Yi et al. 2015). When a small quantity of cement is added to soil–slag mix, the pH of gets elevated. The changes in the morphological arrangement and the nature of Calcium Silicate Hydrate gel formation depend on the pH of the system. Usually, natural soil deposits have a pH in the range of 5–8. The pH of this study soil was about 5.3. The solubility of silica, alumina, present in GBFS particles and clay minerals are increased at higher pH levels, thus making them accessible for reaction with the CaO from cement and GBFS to form reaction products like Calcium Aluminate Hydrate (CAH) and Calcium Silicate Hydrate (CSH). In the presence of CaO the formation of Calcium Silicate Hydrate (CSH), known for binding properties, is enhanced thereby providing strength to the soil-slag-cement mix. The formation of Calcium Aluminum Silicate Hydrate (CASH) is mainly responsible for the high strength (Sharma and Sivapullaiah 2012, Sivapullaiah 2013, Kolias et al. 2005, Jaritngami et al. 2014, Dermatas and Meng 2003). Figure 4.23 shows the plot of failure envelopes for different mixtures of soil. We can observe that the failure envelope becomes steeper and higher with the increase in additive content, i.e. the improvement in friction angle is observed. Improvement in cohesion values can also be observed with the increase in contents of GBFS and cement.

CHAPTER 5

MICROSTRUCTURAL INVESTIGATIONS ON STABILIZED LITHOMARGIC CLAY

5.1 INTRODUCTION

There is good improvement in strength of lithomargic clay due to the replacement of soil by granulated blast furnace slag (GBFS) with/without addition of cement. The strength improvements are due to the production of hydraulic and pozzolanic compounds. The type and amount of compounds formed vary which also lead to microstructural changes. An attempt has been made in this chapter to identify the new compounds formed and consequent changes in the structure of the soil.

5.2 X-RAY DIFFRACTION ANALYSIS

One of the very well established methods for mineralogical characterization of fine-grained soils is by X-ray diffraction (XRD) analysis. Majority of the soil minerals are crystalline in nature and their structure is defined by a unique geometry. XRD identifies minerals based on this unique crystal structure. In XRD, characteristic X-rays of particular wave length are passed through a crystallographic specimen. When X-rays strike a crystal, they penetrate to a depth of several million layers before being absorbed. At each atomic plane a minute portion of the beam is absorbed by individual atoms and which oscillates as dipoles and radiate waves in all directions. When X-ray interacts with crystalline specimen, it gives a particular diffraction pattern, which is unique for a mineral with a particular crystal structure. The diffraction pattern of the soil specimen (according to its crystal structure), which is based on powder diffraction or polycrystalline diffraction, is then analyzed for the qualitative and quantitative assessment of minerals.

The small size of most soil particles prevents the study of single crystals and hence powder method is used. In the powder method, a small sample containing particles at all possible orientations is placed in a collimated beam of parallel X-rays and diffracted beams of various intensities are scanned by a Geiger, proportional, or scintillation tube and recorded automatically to produce a chart showing the intensity of diffracted beam as a function of range 2 Theta (2θ).

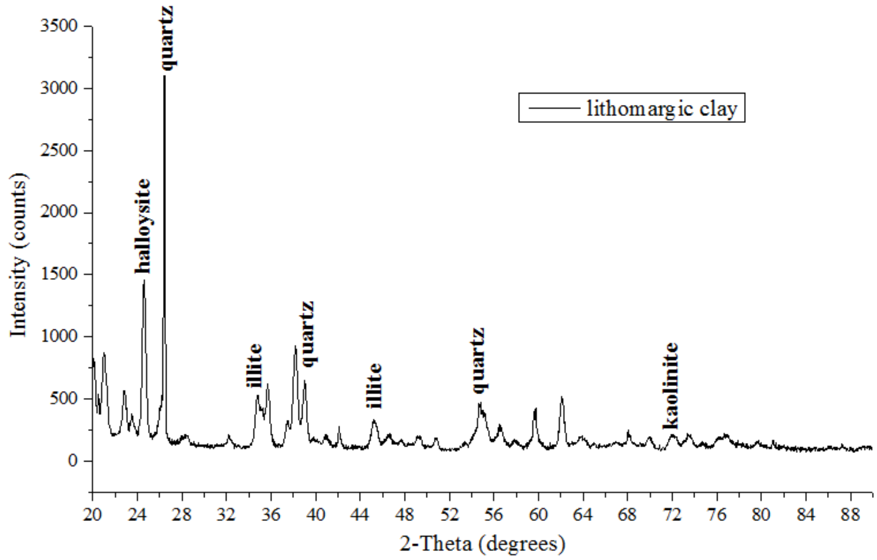

Fig. 5.1 X-ray diffraction spectrum for lithomargic clay (soil)

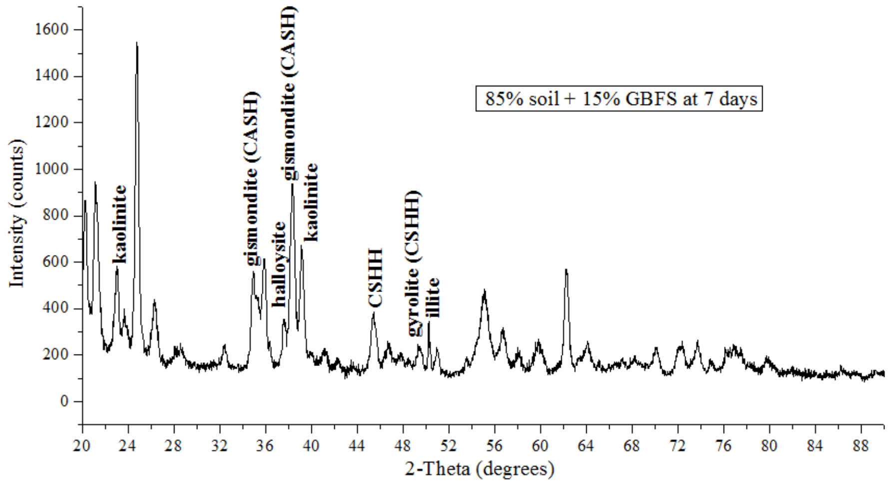

Fig. 5.2 XRD spectrum for 85% soil + 15% GBFS mix at 7 days curing

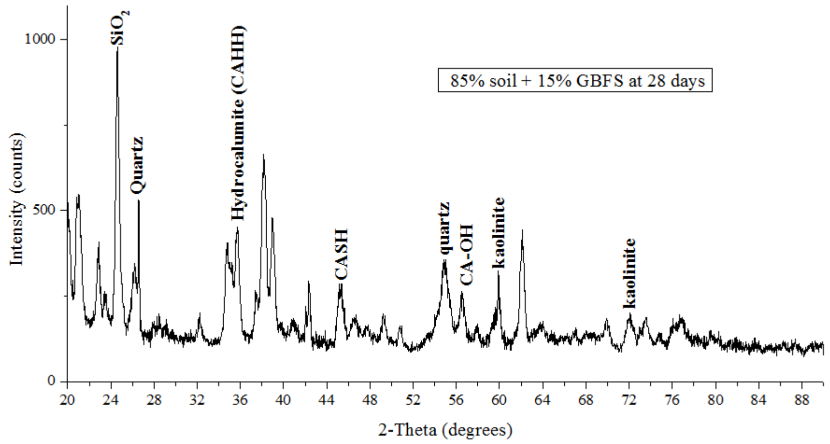

Fig. 5.3 XRD spectrum for 85% soil + 15% GBFS mix at 28 days curing

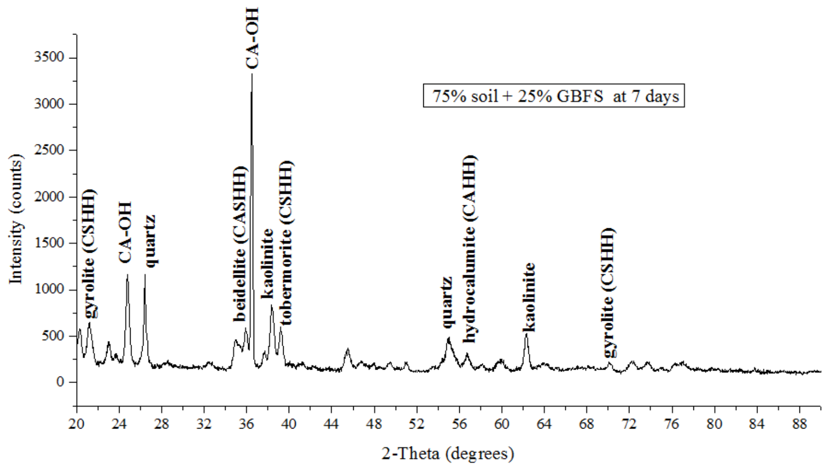

Fig. 5.4 XRD spectrum for 75% soil + 25% GBFS mix at 7 days curing

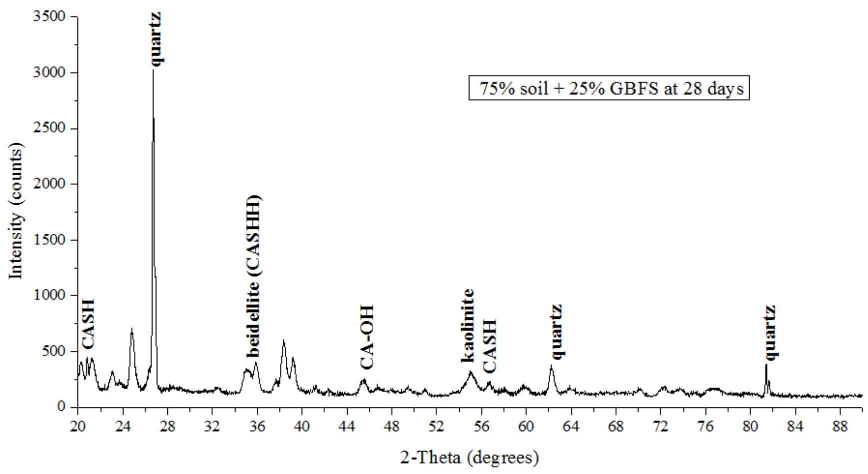

Fig. 5.5 XRD spectrum for 75% soil + 25% GBFS mix at 28 days curing

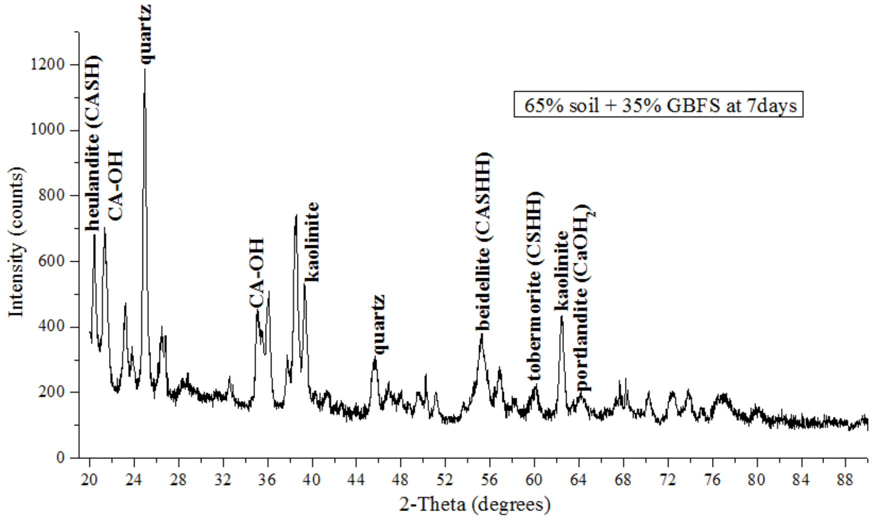

Fig. 5.6 XRD spectrum for 65% soil + 35% GBFS mix at 7 days curing

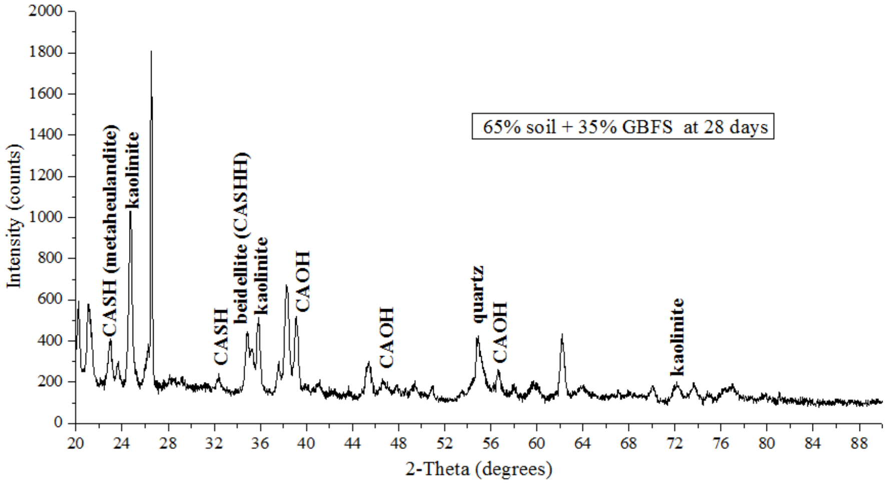

Fig. 5.7 XRD spectrum for 65% soil + 35% GBFS mix at 28 days curing

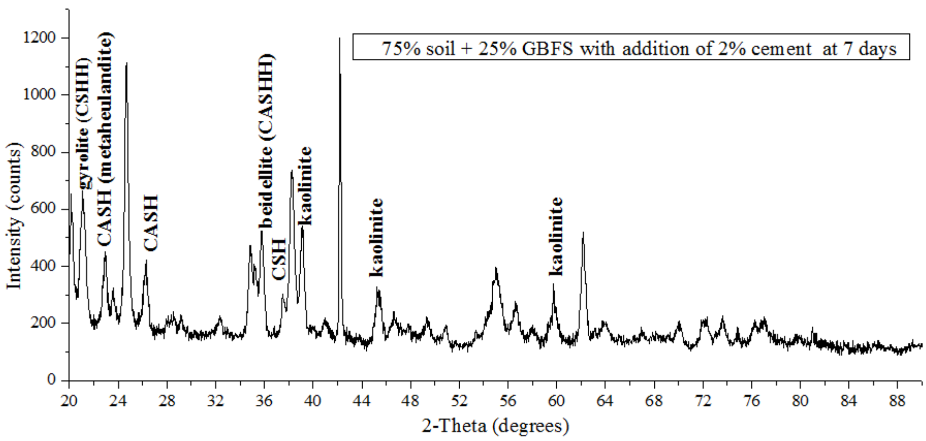

Fig. 5.8 XRD spectrum for 75% soil + 25% GBFS mix with addition of 2% cement at 7 days curing

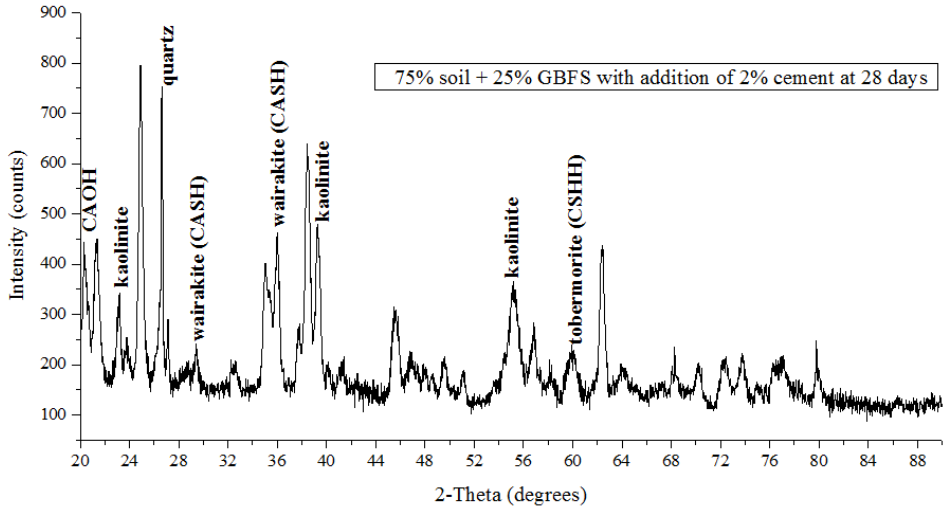

Fig. 5.9 XRD spectrum for 75% soil + 25% GBFS mix with addition of 2% cement at 28 days curing

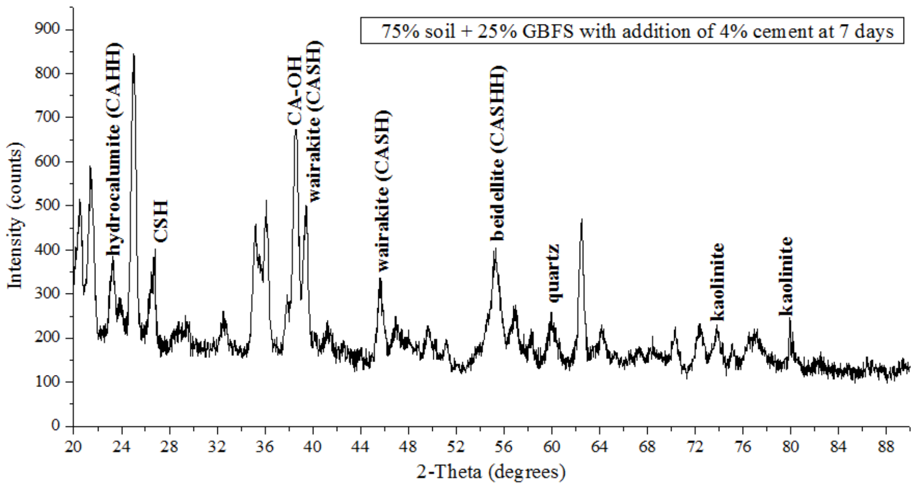

Fig. 5.10 XRD spectrum for 75% soil + 25% GBFS mix with addition of 4% cement at 7 days curing

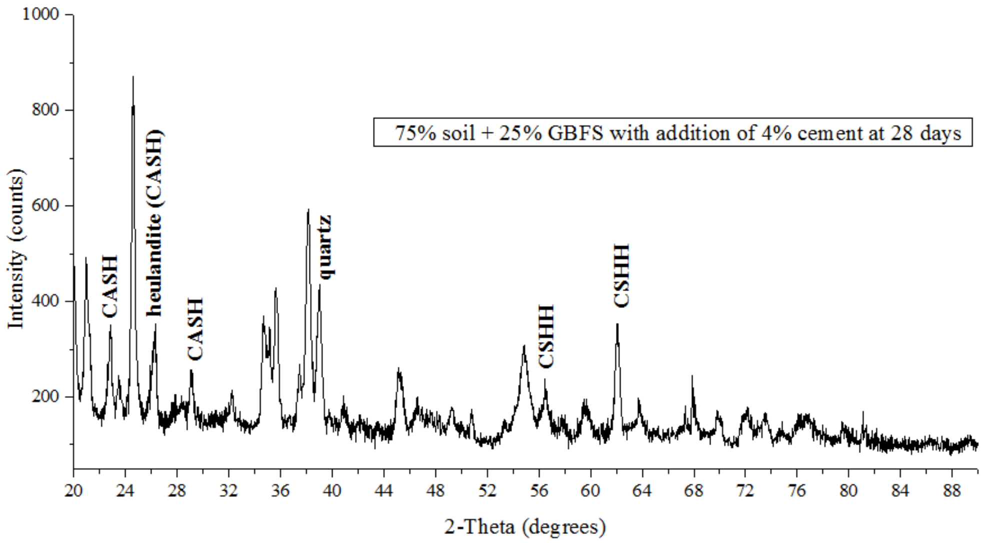

Fig. 5.11 XRD spectrum for 75% soil + 25% GBFS with addition of 4% cement at 28 days curing

Powder X-ray diffraction (XRD) tests were carried out through an X-ray diffractometer, Rigaku Miniflex 600 XRD analyzer or X-ray diffratometer (40Kv, 15mA) with Cu Kα radiation were used for the study. The instrument was based on the Bragg-Brentano geometry. X-Ray Diffraction analysis is done for lithomargic clay and stabilized lithomargic clay to know the effect of stabilization. Lithomargic clay samples stabilized with various percentage of GBFS with/without the addition of cement were subjected to XRD analysis. Samples cured for 7 and 28 days were taken and pulverised, passed through 75 microns IS sieve and powder samples were oven dried at 110°C for 24 hours and subjected to XRD analysis. Soil specimens were placed onto the X-ray diffractometer, and the readings were recorded for 2θ angle from 20˚ to 90˚, and at an angular speed of 2°/min with a step size of 0.02. Data obtained are shown in figures. 5.1-5.11. The mineralogical analysis of the stabilized soil is very important to determine the changes in the mineralogical phases due to pozzolanic reactions. These reactions depend on the chemical and the mineralogical composition of each soil and the additives. The changes in micro structural development of soils due to addition of additives play a significant role in the geotechnical properties and the mechanical behavior of these stabilized soils.

The XRD pattern obtained from powder samples of soil i.e. lithomargic clay is shown in figure 5.1. The major crystalline minerals present in lithomargic clay are kaolinite, illite, halloysite and quartz from the X-ray diffraction analysis. It can be observed that when lithomargic clay soil was replaced by various percentage of GBFS, the cementitious products are formed which are responsible for the increase in strength (figures 5.2-5.11). The free CaO from GBFS promoted the formation and development of pozzolanic reaction in the soil mix. Improvement when soil is replaced by GBFS is due to hydration and pozzolanic reactions in the mix with curing. The calcium hydroxide obtained from the hydration of free lime from GBFS dissociate in water. Hydroxyl ion concentration increases with increase in pH concentration. The increase in pH will accelerate the formation of cementitious compounds, which increases the strength of the soil. The hydrous alumina and silica from both soil and GBFS gradually react with the calcium ions from the hydrolysis of CaO to produce insoluble pozzolanic compounds like CSH, CASH. CSH and CASH are the main reaction products, contributing to the increase in the strength. The hydrated CaO is deposited as a separate crystalline solid phase. High strength and stiffness is achieved due to the elimination of large pores by bonding particles together. Compared with the lithomargic clay, few new peaks of low to moderate intensities are observed for soil + GBFS and soil + GBFS cement combinations indicating formation of new compounds. Among these, the major cementitious compounds are calcium silicate hydrate (CSH) [$Ca_2HO_{4.5}Si$], calcium aluminum silicate hydrate (CASH) [$Al_2Ca_{11}O_{23.5}Si_7$], calcium silicate hydroxide hydrate (CSHH) [$Ca_{4.5}H_7O_{20}Si_6$], calcium aluminum oxide hydrate (CAOH) [$Al_2Ca_4H_{38}O_{26}$], beidellite [$Al_2Ca_{0.2}H_{14}O_{18}Si_4$], gyrolite [$Ca_4H_8O_{20}Si_6$], wairakite [$Al_2CaH_4O_{14}Si_4$], gismondine [$Al_8Ca_4H_{34.42}O_{49.21}Si_8$], heulandite [$Al_2CaH_{12}O_{24}Si_7$] and tobermorite [$Ca_5H_{10}O_{22}Si_6$].

These new products bind together and form a hardened skeleton matrix. These compounds were observed in all the stabilized soil mixes and hence the improvement in strength properties is achieved. The other compounds present are quartz and kaolinite, which were originally present in the untreated soil. However, the peaks of these were reduced when treated with GBFS and cement indicating breakdown of kaolinite and quartz minerals. The formation of stronger cementitious peaks are observed more clearly with addition of cement. The intensity of peaks related with cementitious compounds appear broader with increase in curing periods.

The hydration and pozzolanic reactions occur at all percentages of replacement. However, at 25% replacement of soil by GBFS, UCS was found to be maximum. At lower percentages of replacement i.e. 15%, the dissolution of alumina and silica from the slag-soil mix is slightly less (figure 5.2). Thus, only few compounds like gismondine, gyrolite and CSHH are observed which resulted in little improvement in strength. At 25% replacement, the dissolution of alumina and silica is complete and hence all the ions participate in the hydration and pozzolanic reactions. The maximum strength is justified with the formation of gyrolite, hydrocalumite, tobermorite, beidellite and CAOH compounds (figure 5.4). At higher percentage of replacement i.e. 35%, the moulding water that is available for the hydration and pozzolanic reactions is less and the excess CaO remains redundant. This was observed with the occurrence of portlandite [$Ca(OH)_2$] (figure 5.6). Thus, the voids are filled by the unhydrated slag particles, which reduced the strength of the soil matrix.

5.3 SCANNING ELECTRON MICROSCOPY ANALYSIS

The Scanning Electron Microscope (SEM) is one of the most versatile instrument used to study the microstructure of soil particles. Secondary electrons are emitted from a sample surface and it appears as three-dimensional images. The SEM has x20 to x150, 000 magnification range and depth of field 300 times greater than that of the light microscope. In SEM, the sample is being irradiated with an electron beam and the electrons that are being emitted, scatter electrons from the sample under analysis. Moreover, by looking at these scattered electrons, by gathering them and subjecting them to appropriate analysis, we can get an idea about the topography of the surface. This has led to extensive use of SEM for the study of clay particles. SEM can provide shape analysis, size analysis, and texture of the specimens. The Scanning Electronic Microscope (SEM) study along with EDS (Energy Dispersive Spectroscope) can display all the elements present in the specimen as they are being collected and enables to perform rapid identification. It enables the composition of the specimen to be determined to an overall accuracy of about 99% and detection sensitivity down to 0.1%

by weight. A semiconductor material is used to detect the X-rays together with processing to analyze the spectrum. The energy dispersive spectroscope (EDS) analysis indicates the elements present in the soil samples which involves in the hydration and pozzolanic reactions.

Samples are prepared to their maximum dry density and OMC, cured for 7 and 28 days and subjected to SEM. Lithomargic clay and stabilized lithomargic clay are observed under microscope to magnification of 200x to 3000x, i.e. up to 3-micron level. The changes in texture and morphology are analyzed. Pieces of soil samples, sliced from the central region of the post-test specimen, were oven-dried. The dried soil samples were mounted in phenolic resin base and were coated with a thin layer of gold palladium to provide surface conductivity. The specimens thus prepared were placed in an SEM instrument and the photomicrographs were generated. Scanning electron microscope images show that, there is formation of various new compounds when lithomargic clay is stabilized using GBFS and cement.

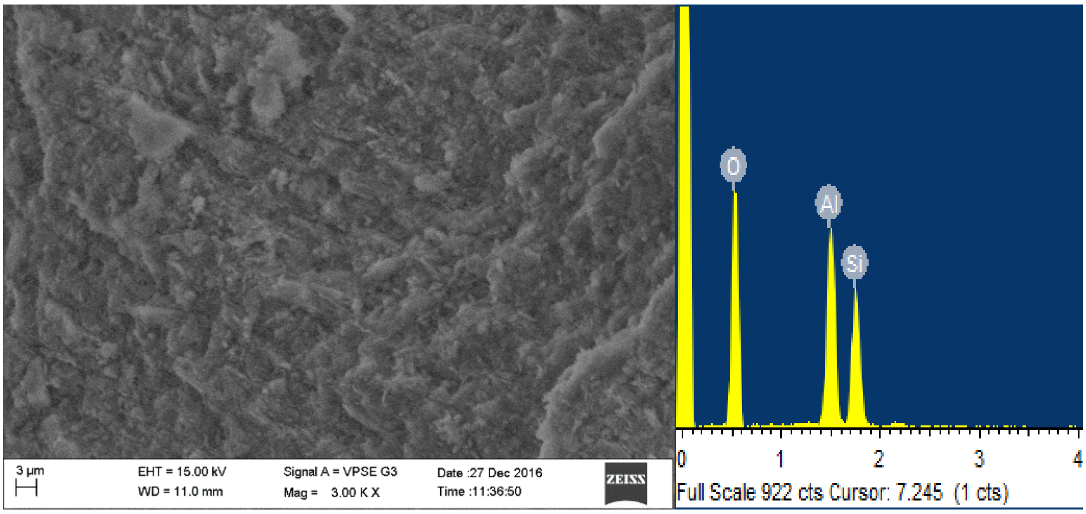

Fig. 5.12 SEM image and EDS micrograph of lithomargic clay (soil)

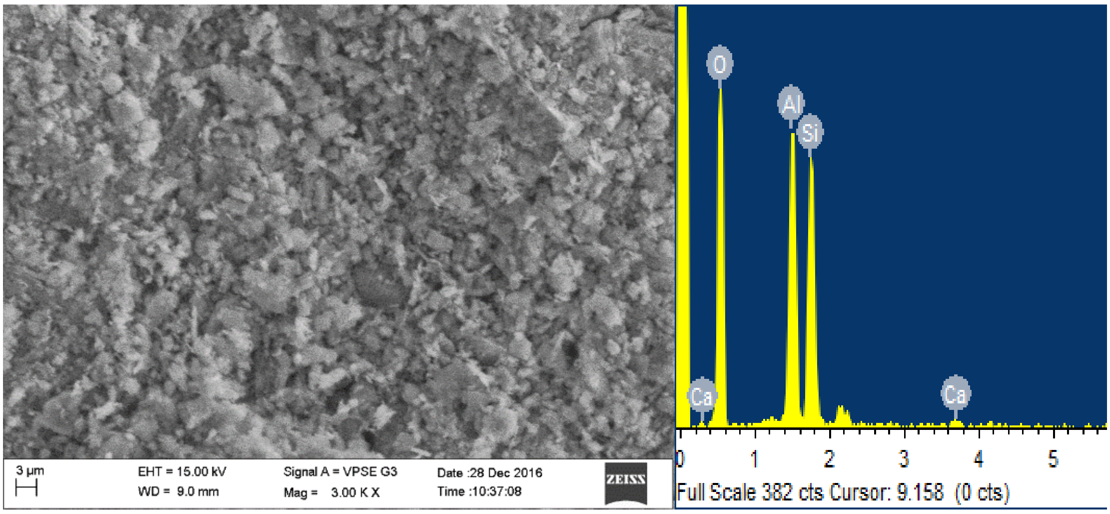

Fig. 5.13 SEM image and EDS micrograph of 85% soil + 15% GBFS mix at 7 days curing.

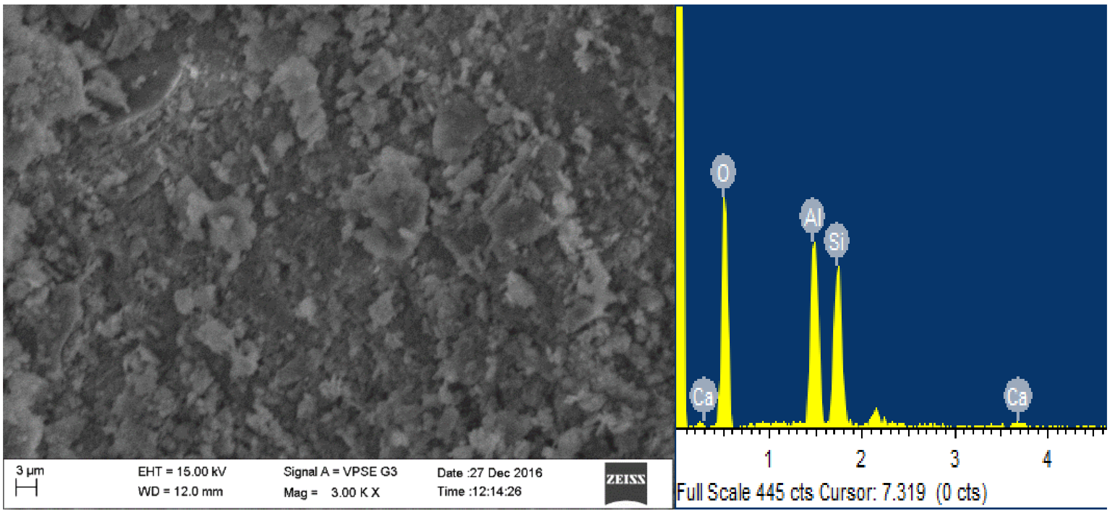

Fig. 5.14 SEM image and EDS micrograph of 85% soil + 15% GBFS mix at 28 days curing.

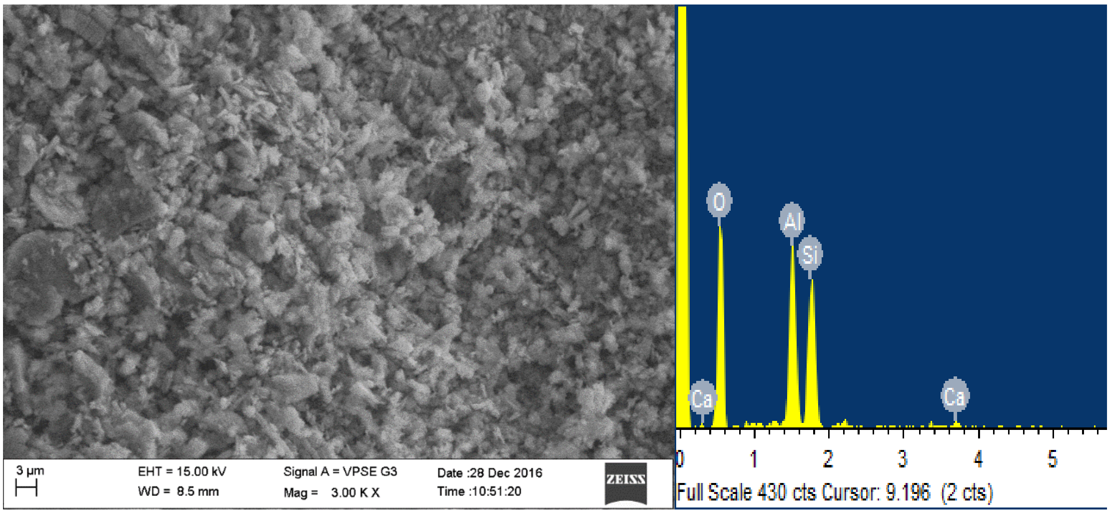

Fig. 5.15 SEM image and EDS micrograph of 75% soil + 25% GBFS mix at 7 days curing.

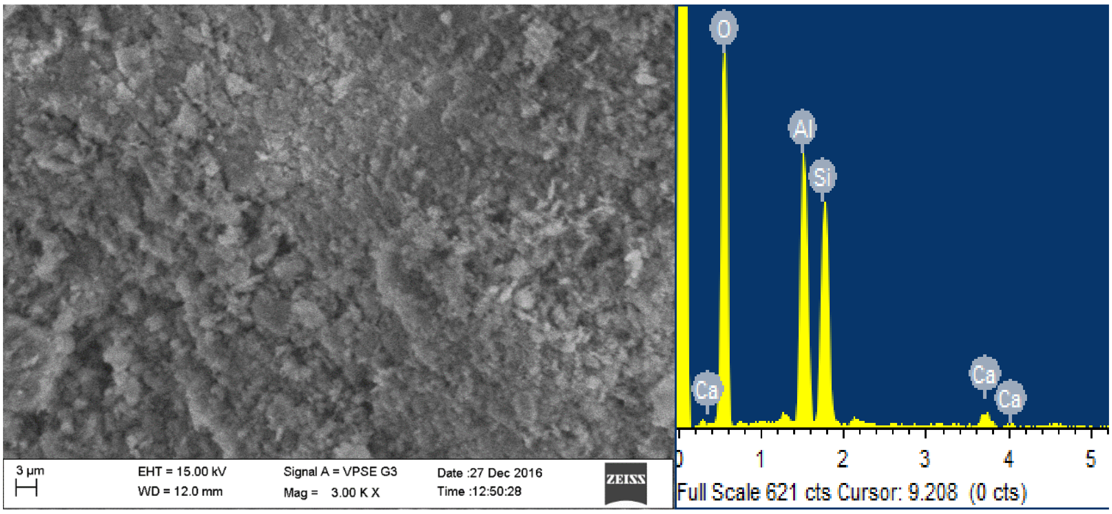

Fig. 5.16 SEM image and EDS micrograph of 75% soil + 25% GBFS mix at 28 days curing.

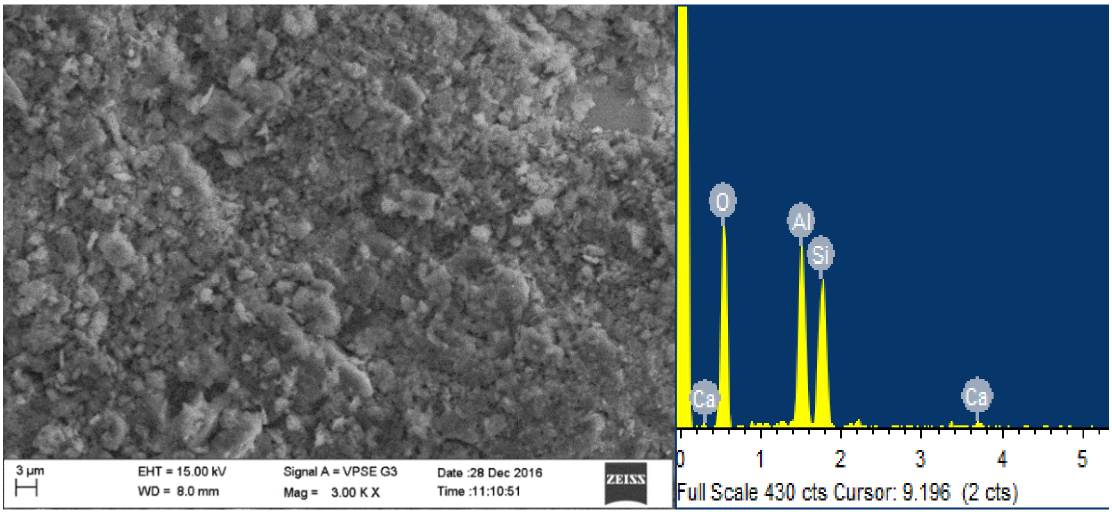

Fig. 5.17 SEM image and EDS micrograph of 65% soil + 35% GBFS mix at 7 days curing.

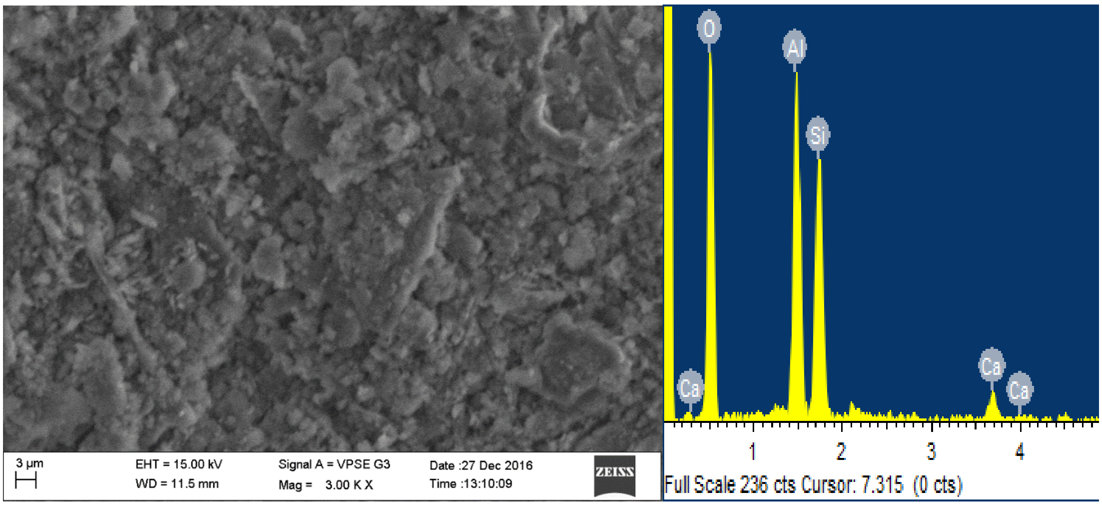

Fig. 5.18 SEM image and EDS micrograph of 65% soil + 35% GBFS mix at 28 days curing.

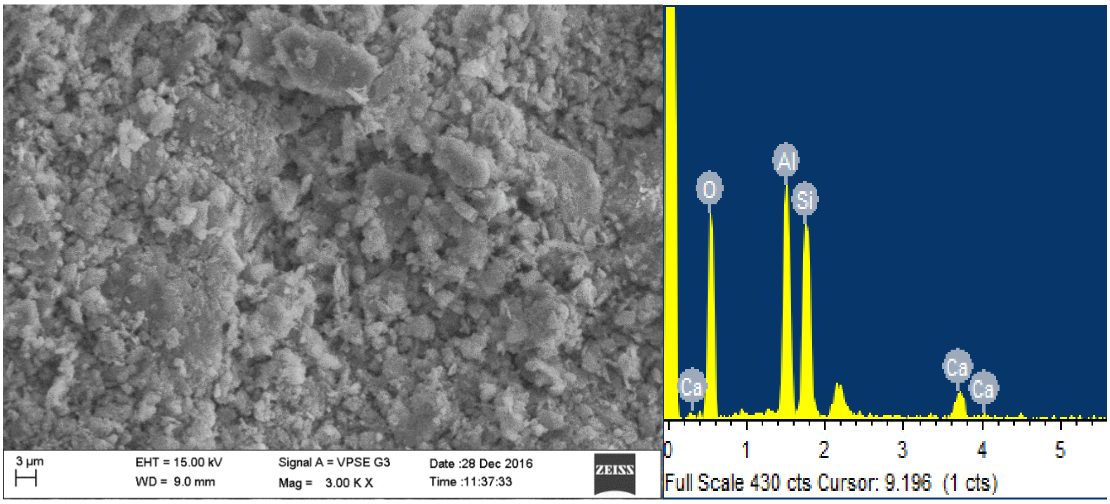

Fig. 5.19 SEM image and EDS micrograph of 75% soil + 25% GBFS mix with addition of 2% cement at 7 days curing.

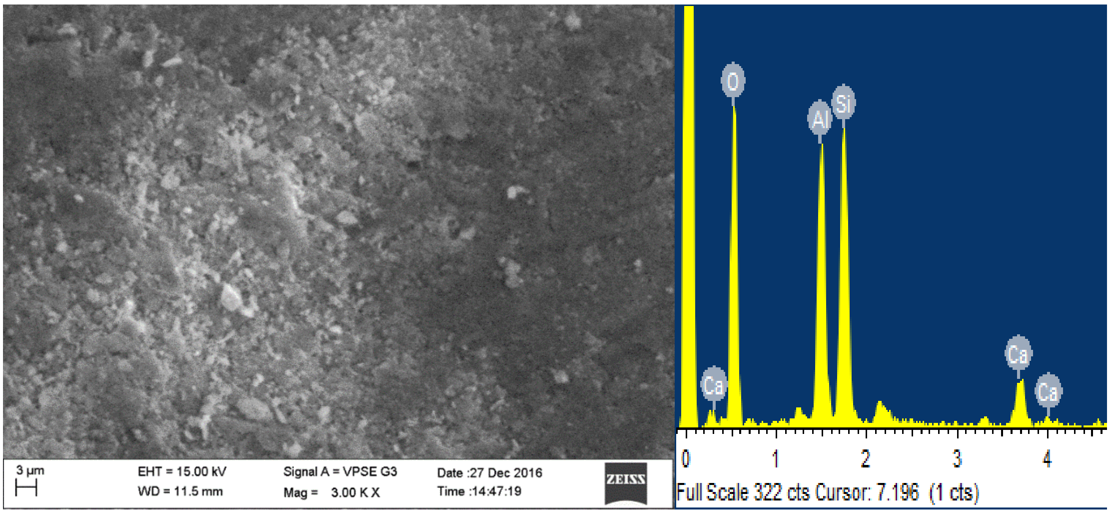

Fig. 5.20 SEM image and EDS micrograph of 75% soil + 25% GBFS mix with addition of 2% cement at 28 days curing.

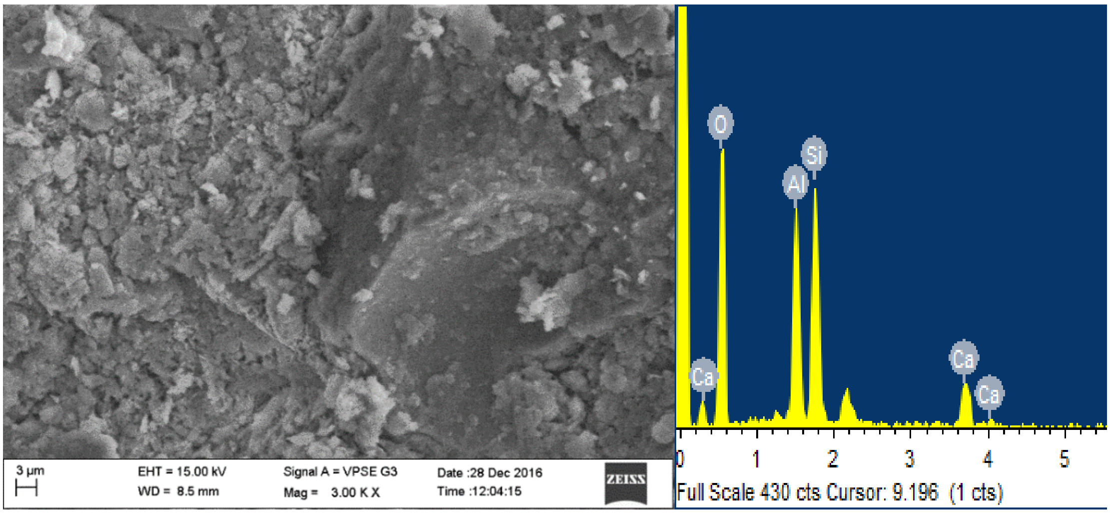

Fig. 5.21 SEM image and EDS micrograph of 75% soil + 25% GBFS mix with addition of 4% cement at 7 days curing.

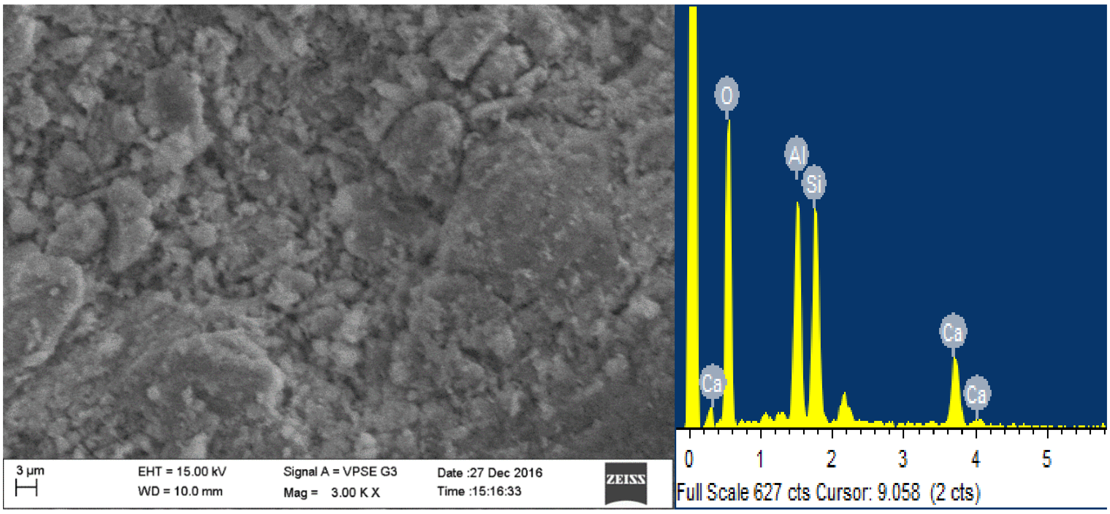

Fig. 5.22 SEM image and EDS micrograph of 75% soil + 25% GBFS mix with addition of 4% cement at 28 days curing.

The SEM images of lithomargic clay stabilized with GBFS and cement was taken at a magnification capacity of 3000 and a scale bar of 3μm. From figure 5.12, it is observed that the lithomargic clay had a smooth texture, wavy in nature with visibly larger void spaces. With the replacement of soil by GBFS, it was observed that the presence of GBFS has sufficient bonding agent to produce agglomerations (figures 5.13-5.22). The particles seem to be flocculated, physically the pore or air spaces have reduced and the mixture has gained strength. The development and maintenance of high strength and stiffness is achieved by the elimination of large pores by bonding of particles and flocculent particles arrangements. From figures 5.13-5.22, it is clear that, at higher curing periods, compacted matrix and reticulated structure with formation cementitious patch like compound are observed. This change in surface morphology in stabilized samples after curing is attributed to the formation of harden skeleton and cementitious matrix due to the inherent pozzolanic and cementitious property of lithomargic clay treated with GBFS and cement.

The formation of cementitious products leads to the reduction of voids/air spaces. This phenomenon occurs due to coating and binding of individual soil particles with cementitious gels resulting in the reduction in migration of ions into the pores resulting in a rigid structure. It is due to the fact that during early period of curing, thin layers of hydration products are formed on the surface of the particles due to pozzolanic reactions. The inner part of the thin layer consists of denser mass and the outer layer of the particle is of fine fibrous matter. It may be inferred that at early stages, the GBFS particles served as nucleation sites for hydration and pozzolanic reaction products. At higher curing periods, the SEM micrographs show dense gel-like mass covering all the composite particles completely and filling up the inter-particle spaces. The grain boundaries appear blurred and the dense gel acting as a binding substance is evenly distributed to form compact structure. It creates more contacts and greater cohesion in the soil composite mass that in turn contributes to higher rigidity and greater strength (Mishra and Karanam 2006).

5.4 QUANTITATIVE EDS ANALYSIS

Table 5.1 Quantitative EDS analysis of various samples at different curing periods.

Curing period	Combinations	Al (%)	Si (%)	Ca (%)	Al:Ca	Ca:Si
	Soil (Lithomargic clay)	19.22	19.93	-	-	-
7 days	85% soil + 15% GBFS	18.92	21.54	1.12	16.89	0.05
7 days	75% soil + 25% GBFS	18.56	19.68	1.68	11.04	0.08
7 days	75% soil + 25% GBFS with 2% cement	18.80	21.29	5.34	3.52	0.25
7 days	75% soil + 25% GBFS with 4% cement	14.92	18.35	9.04	1.65	0.49
28 days	85% soil + 15% GBFS	19.92	20.63	1.88	10.60	0.09
28 days	75% soil + 25% GBFS	17.58	18.53	2.32	7.58	0.12
28 days	75% soil + 25% GBFS with 2% cement	15.02	21.26	6.21	2.42	0.29
28 days	75% soil + 25% GBFS with 4% cement	12.85	15.55	10.63	1.21	0.68

The quantitative analysis showed that with the replacement of soil by GBFS and addition of cement, the kaolinite and quartz mineral broke and new cementitious compounds (mainly CASH and CSHH). These compounds are responsible for the strength gain. EDS analysis for same samples was also done along with SEM imaging. The results are analyzed for the modification occurred due to replacement of soil by GBFS and addition of cement. The EDS micrographs of soil treated with GBFS and cement at 7 and 14 days of cured samples show distinct peaks of Al, Si, and Ca. These peaks shows that hydration and pozzolanic reactions occur in presence of GBFS and cement to form stabilizing compounds. From the EDS data collected, it was observed that the highest peaks were of silica, oxygen, followed with low peaks of calcium. The peaks of calcium suggests that hydration and pozzolanic reaction has taken place, which resulted in increase of strength. Consumption of amorphous silica further strengthened the formation of new compounds. Results of EDS analysis for various samples at different curing periods are presented in Table 5.1. From Table 5.1, it is observed that there is increase in Ca and decrease in Al for soil stabilized with GBFS and cement at 28 days curing. It confirms the formation of more cementitious compounds with slow process of pozzolanic reaction [CSH and CAH]. These are responsible for increase in strength. From the EDS micrographs, it is observed that the intensity peaks of Ca, S, Al and O become relatively stronger with increase in curing period. Increase in the ratio of Ca:Si with cement content and curing period may be attributed to the formation of pozzolanic reaction products (Jha and Sivapullaiah 2014).

The percentage of Ca increases with the increase in GBFS and cement content, furthermore with increase in curing period. The ratios of Al:Ca and Ca:Si brought out the changes in the composition of particle surfaces due to coating of new minerals/compounds formed due to reactions that occur between the soil particles and additives. Strength improvement due to pozzolanic reactions is achieved with increase in Ca:Si and decrease in Al:Ca ratios (Table 5.1). Moreover, the increase in calcium percentage in soil stabilized with GBFS and cement confirms that role of C_2S and C_3S is more pronounced.

CHAPTER 6

TYPICAL STUDIES ON APPLICATIONS OF STABILIZED SOIL USING PLAXIS 2D

6.1 GENERAL

PLAXIS is a finite element software, which covers all aspects and applications of geotechnical engineering simulation using a user-friendly interface with the power of finite element. It is intended for 2 Dimensional and 3 Dimensional analysis of deformation and stability of soil structures, as well as groundwater and heat flow, in geo-engineering applications such as excavations, foundations, embankments and tunnels.

PLAXIS 2D is a two-dimensional finite element program, developed for the analysis of deformation, stability and groundwater flow in geotechnical engineering. The simple graphical input procedures enable a quick generation of complex finite element models and the enhanced output facilities provide a detailed presentation of computational results. Geotechnical applications require advanced constitutive models for the simulation of the non-linear, time dependent and anisotropic behaviour of soils and rock. Special procedures are required to deal with hydrostatic and non hydrostatic pore pressure in the soil since it is a multiphase material. PLAXIS is equipped with features to deal with various aspects of complex geotechnical structures.

Laboratory test results obtained for lithomargic clay, lithomargic clay stabilized with GBFS and lithomargic clay stabilized using GBFS and cement are applied to following two typical cases using PLAXIS 2D.

i. Load settlement analyses of strip footings resting on lithomargic clay and stabilized lithomargic clay.
ii. Stability analyses of embankment slopes (soon after construction-UU condition).

Features of PLAXIS program, results obtained from load settlement analysis of strip footing and stability analysis of embankment slopes are discussed in the subsequent sections.

6.2. OVERVIEW OF THE ANALYSIS PROGRAM

The basic concept of finite element analysis is that a complicated model of a body or structure is divided into a number of smaller elements. These elements are then connected by nodes. By solving the values at the nodes the stresses and strains in every element can be calculated. In PLAXIS 2D real situations may be modelled either by a plane strain or an axisymmetric model. To carry out a finite element analysis, the user has to create a two dimensional geometry model composed of points, lines and other components in the X-Y plane and specify the material properties and boundary conditions. The geometry of the model can be easily defined in the soil and structures modes. When the geometry modelling process is complete, user can proceed with the calculations. This consists of generation of mesh and definition of the construction stages. The staged construction mode allows for simulation of construction and excavation processes by activating and deactivating soil clusters and structural objects. Finite element calculations can be divided into several sequential calculation phases. When the calculation phases have been defined and points for curves have been selected, then the calculation process can be executed. When it gets finished, the calculation list is updated. An extensive range of facilities exists within the PLAXIS 2D output programs to display the results of the analysis. The main output quantities of a finite element calculation are the displacements and the stresses. If the model involves structural elements, then the structural forces in them are also calculated.

6.3 LOAD-SETTLEMENT ANALYSES OF STRIP FOOTING RESTING ON LITHOMARGIC CLAY AND ON STABILIZED LITHOMARGIC CLAY

A series of two dimensional finite element analyses on strip footings of different widths were performed to understand the load settlement behaviour. The analyses were performed using the finite element program PLAXIS 2D software package.

The load settlement analyses of lithomargic clay, stabilized lithomargic clay with different percentages of granulated blast furnace slag (GBFS) replacing the soil and a combination of soil-GBFS and cement was performed made for strip footings of width 1 m, 1.5 m and 2 m.

Details of the footing:

Width of the footing – 1 m, 1.5 m and 2 m

Depth of footing – 1.2 m below GL

Thickness of footing – 0.6 m

Footing properties:

Modulus of elasticity (E) – 2.2 x 10^7 kPa

Poisson's ratio (μ) – 0.15

Unit weight (γ) – 25 kN/m^3

The properties of the lithomargic clay and the stabilized soil (with different percentages of GBFS and GBFS-cement combination) that were used as input in the analyses are shown in Table 6.1.

Table 6.1 Input parameters for lithomargic clay, GBFS stabilized lithomargic clay and GBFS + cement stabilized lithomargic clay

Properties	Max. dry unit weight (kN/m^3)	OMC (%)	Modulus of elasticity E (MPa)	Cohesion c (kN/m^2)	Angle of internal friction ϕ°	Coefficient of permeability k (m/day)	Poisson's ratio μ
Lithomargic clay (LC)	14.20	28	2.25	23	19	0.0032	0.33
95% LC + 5% GBFS	14.30	27	3.9	40	22.5	0.0398	0.34
85% LC + 15% GBFS	14.30	25.5	5.6	61	26	0.0569	0.35
75% LC + 25% GBFS	14.20	24	7.3	83	29	0.0714	0.37
75% LC + 25% GBFS with addition of 2% cement	14.20	24	9.2	246	42	0.00015	0.34
75% LC + 25% GBFS with addition of 4% cement	14.20	24	9.6	383	43	0.00017	0.33

6.3.1 Steps followed for settlement analyses of strip footing resting on lithomargic clay and stabilized lithomargic clay using PLAXIS 2D

Step 1: In the new window of PLAXIS 2D a suitable project title and description is to be give followed by defining the geometry of footing to be modelled. In this particular study, the geometry of 8B width and -4B depth is fixed on observing no considerable changes in the settlement after running few trial settlement analyses (figure 6.1).

Step 2: A bore hole is placed in the model which enables to define the soil properties like unit weight, permeability, modulus of elasticity and shear strength parameters (c and ϕ). Total depth of the soil model (4B) is also defined in the bore hole.

Step 3: Strip footing of 0.6 m thickness is placed at -1.2m depth and concrete properties are assigned to it.

Step 4: A uniformly distributed load is placed on the footing.

Step 5: 2D meshes are generated.

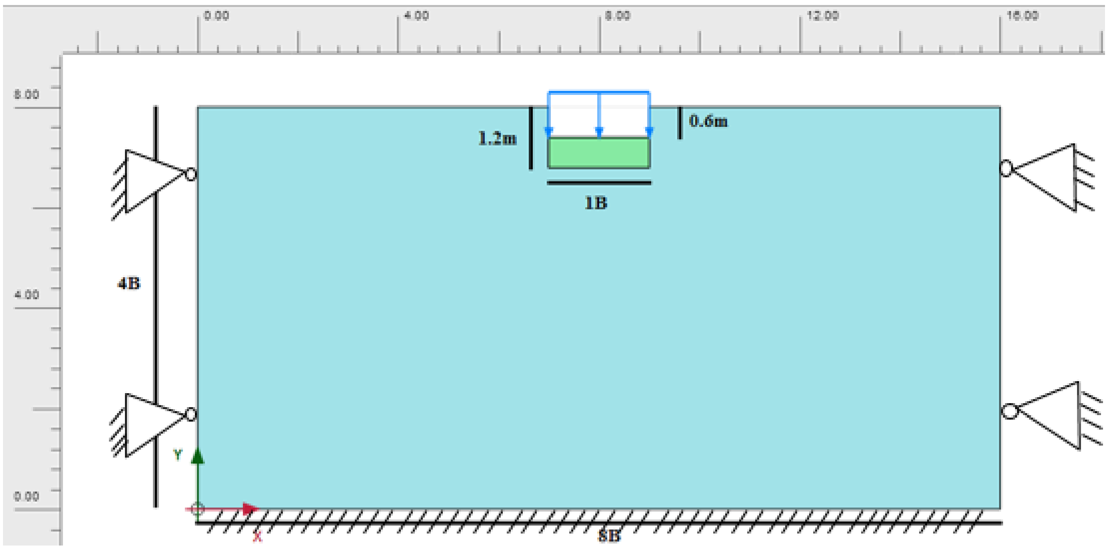

Fig. 6.1 Finite element model for 2m wide (B=2m) strip footing on lithomargic clay

Step 6 (Calculation Phase): The applied soil and concrete properties and loadings are activated as follows:

Phase 1: Excavation- By default a soil layer will be activated above the footing placed at -1.2m. Hence the soil layer above the footing is turned off to deactivate it.

Phase 2: The footing is activated. Then the load intensity is activated and a load intensity of 10kPa is applied.

Step 7: Calculations are started.

Step 8: In the output tab, the settlement values are shown with a figure showing the settlement contours.

Step 9: Step 6 is repeated with increased loading intensity of 50kPa, 100kPa, 150kPa, etc. up to 400kPa and the corresponding settlements are found out.

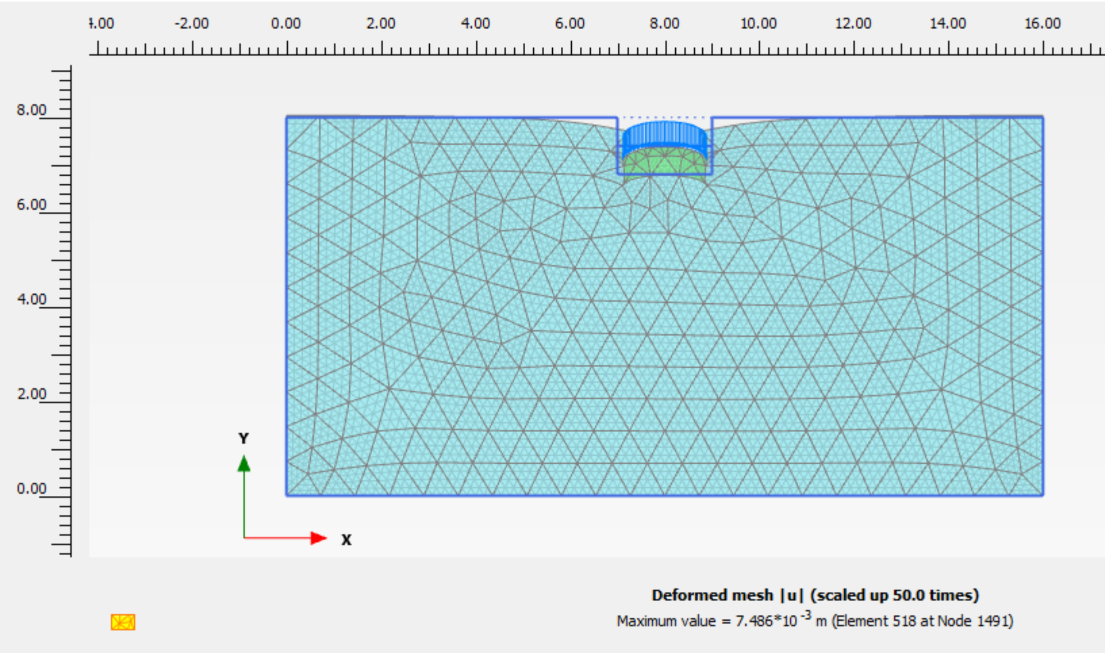

Fig. 6.2 Typical output showing deformed mesh.

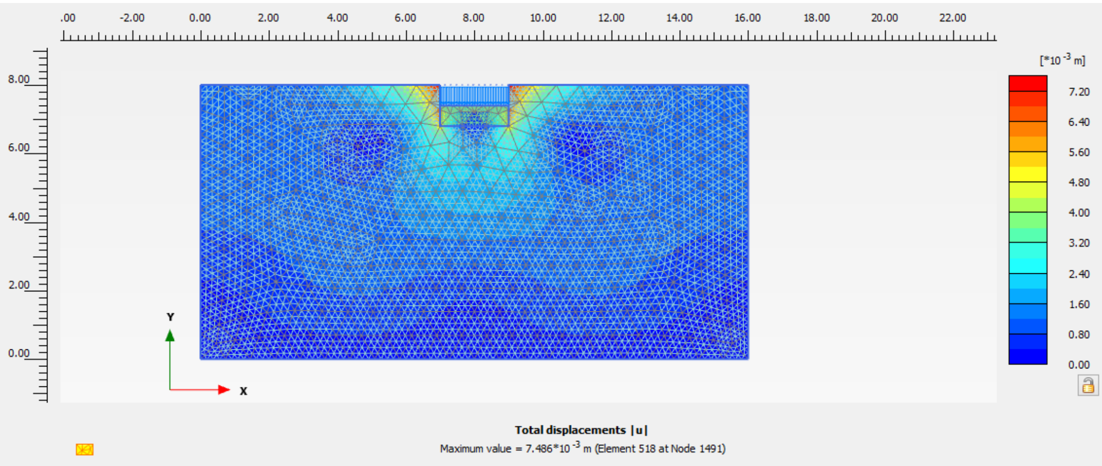

Fig. 6.3 Typical output showing settlement contours.

Typical output showing deformed mesh and settlement contours are shown in figure 6.2 and figure 6.3 respectively.

6.3.2 Different cases considered in the analyses

Three cases have been run in PLAXIS 2D software with varying combinations of lithomargic clay, GBFS and cement. In case 1, the footing width is varied as B=1m, 1.5m, 2m and load intensity is varied as 10kPa, 50kPa, 100kPa, 150kPa, 200kPa, 300kPa, 350kPa and 400kPa for footing resting on lithomargic clay. In case 2, footing is resting on 2B width and 1B depth of stabilized soil. In case 3, footing is resting on 3B width and 2B depth of stabilized soil. In all the cases the footing widths of 1m, 1.5m and 2m and loading intensities of 10, 50, 100…..400kPa have been considered.

Case 1: Footing resting on lithomargic clay (figure 6.4)

Case 2: Footing resting on stabilized soil having width 2B and depth 1B (figure 6.5)

Case 3: Footing resting on stabilized soil having width 3B and depth 2B (figure 6.6)

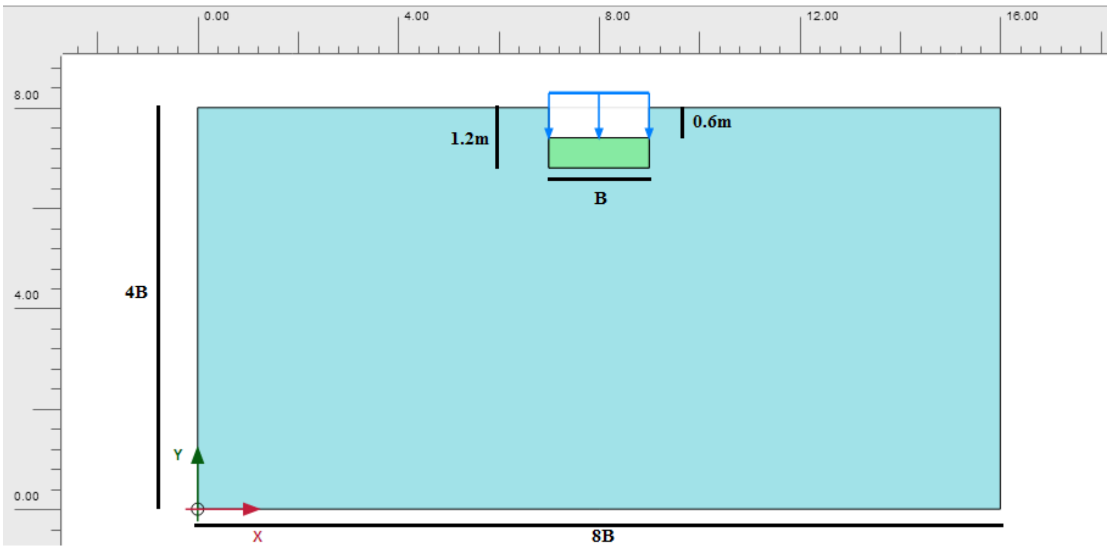

Fig. 6.4 Case 1: Footing resting on lithomargic clay

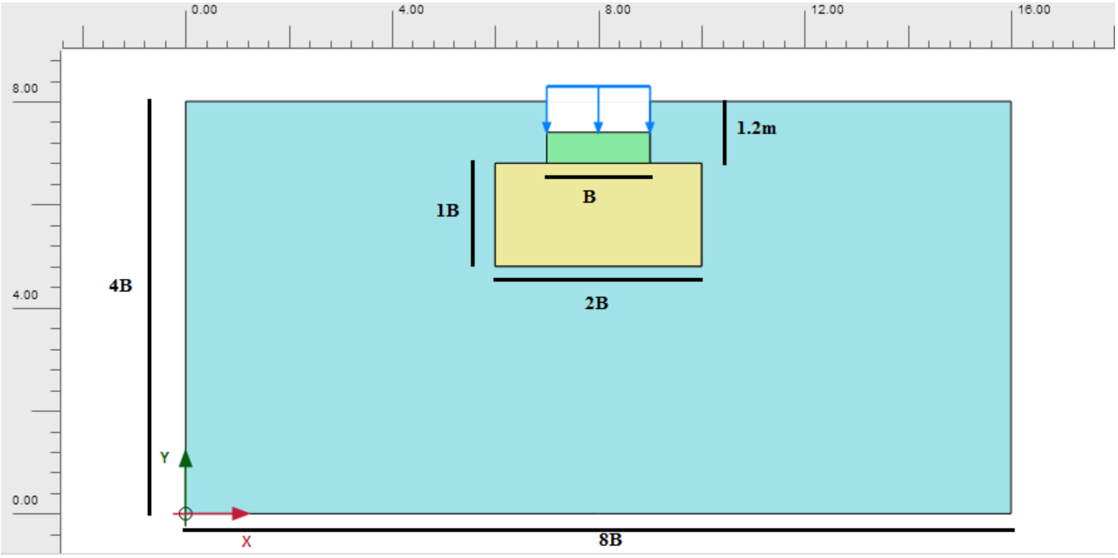

Fig. 6.5 Case 2: Footing resting on stabilized soil having width 2B and depth 1B

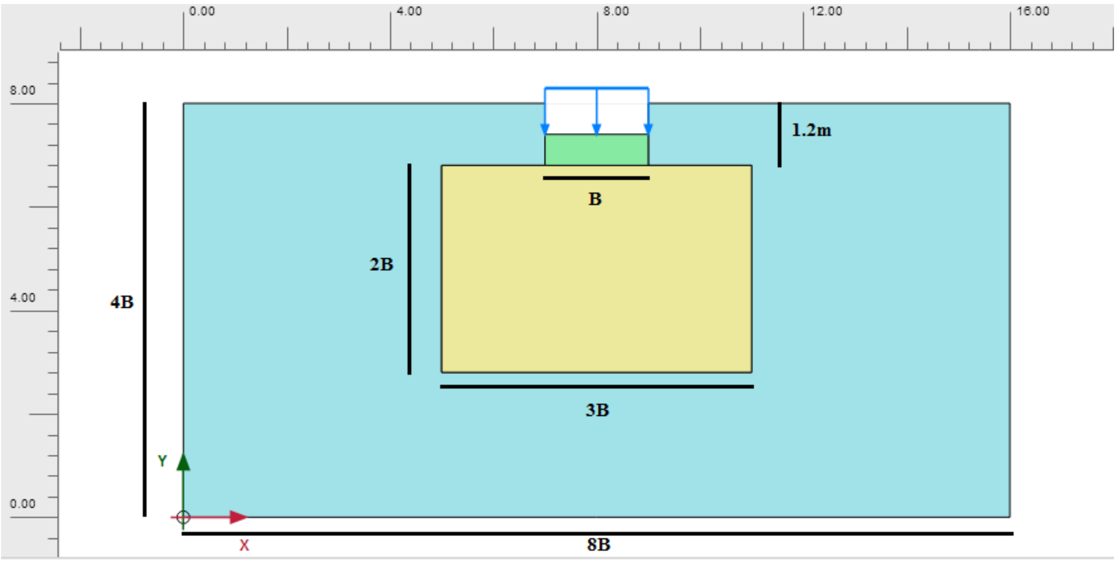

Fig. 6.6 Case 3: Footing resting on stabilized soil having width 3B and depth 2B

Case 1: Load settlement behaviour of strip footing resting on lithomargic clay

The results of the load-settlement analyses for 1m, 1.5m and 2m wide strip footing resting on lithomargic clay as obtained from the PLAXIS 2D software are presented in Table 6.2and figure 6.7. From the figure 6.7, it is clear that, with the increase in width of the footing the settlement increases. In addition, the settlement increases with the increase in load intensities. For load intensity of 200kPa, settlement for 1m wide strip footing is 77.38mm, which increases to 167.4mm for footing width of 2m (increase of 116.3%).

Table 6.2 Settlement of strip footings of different widths resting on lithomargic clay

Load intensity (kPa)	Settlement (mm)		
	1m wide strip footing	1.5m wide strip footing	2m wide strip footing
10	5.32	6.28	7.59
50	14.94	20.10	22.80
100	26.71	32.43	41.51
150	40.90	57.81	76.41
200	77.38	121.80	167.40
250	165.90	295.00	428.50

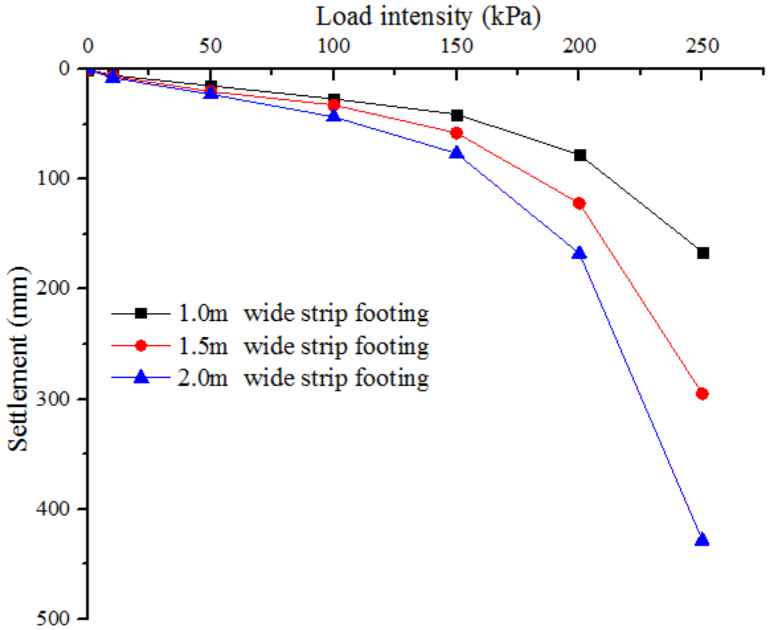

Fig. 6.7 Settlement vs load intensity for 1m, 1.5m and 2m wide strip footing resting on lithomargic clay.

Case 2: Load settlement behaviour of strip footing resting on stabilized soil of width 2B and depth 1B considering various soil mixtures

PLAXIS 2D analyses were carried out for width of footing 1m, considering 95% soil + 5% GBFS, 85% soil + 15% GBFS, 75% soil + 25% GBFS, 75% soil + 25% GBFS with addition of 2% cement and 75% soil + 25% GBFS with addition of 4% cement. Results are provided in Table 6.3 for B=1m. Figure 6.8 shows the graphical representation of load settlement response (B=1m).

From Table 6.3 and figure 6.8, it is observed that with the increase in load intensities, the settlement increases for lithomargic clay. Upon stabilization with GBFS and cement the settlement decreases. For a particular load intensity of 200kPa, settlement for footing on lithomargic clay is 77.38mm which reduces to 37.6mm when soil is replaced by 25% GBFS. Reduction in settlement is 51.4%. On addition of 4% cement to the same 75% soil + 25% GBFS mix, settlement reduced to 28.93mm (total reduction of 62.6%). Similar trends are observed for all load intensities and for all mixes. For footing width of 1.5m, Table 6.4 and figure 6.9 and for footing width of 2m, Table 6.5 and figure 6.10 shows results obtained from similar analyses.

Table 6.3 Settlement of B=1m wide strip footing resting on stabilized soil of width 2B and depth 1B considering various soil mixtures.

Settlement (mm)						
Load intensity (kPa)	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25%GBFS with addition of 4% cement
10	5.32	4.17	3.97	3.83	2.92	2.12
50	14.94	12.76	12.06	11.81	7.91	7.19
100	26.71	23.52	22.71	20.69	14.51	12.28
150	40.90	31.90	30.08	28.13	24.62	21.20
200	77.38	42.36	39.86	37.60	30.28	28.93
250	165.90	71.80	55.16	49.32	43.32	38.52
300	-	141.20	75.19	63.78	56.92	51.13
350	-	-	120.60	87.42	77.66	68.15
400	-	-	-	113.20	101.90	89.39

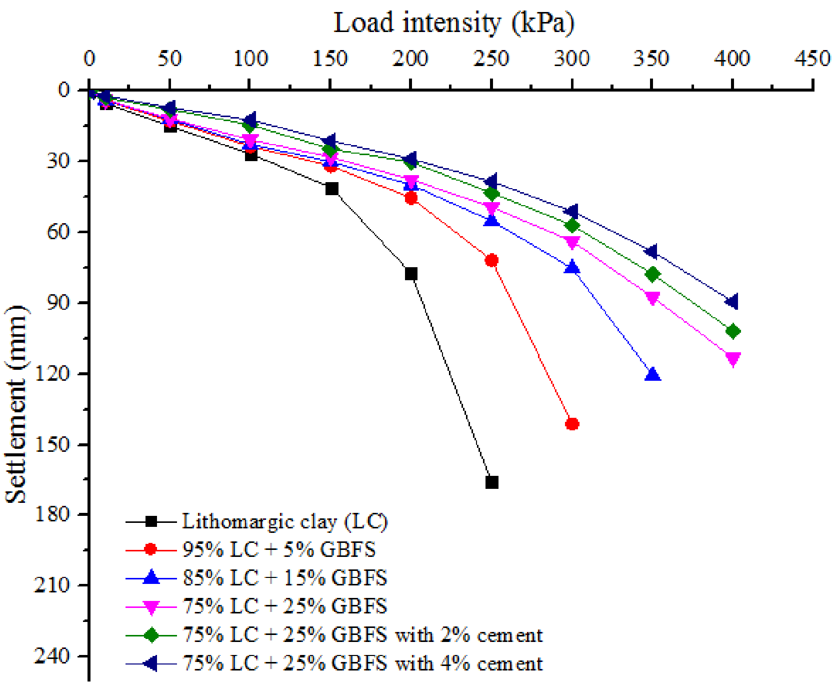

Fig. 6.8 Settlement vs load intensity for different soil combinations for strip footing of width B=1m resting on stabilized soil of width 2B and depth 1B.

Table 6.4 Settlement of B=1.5m wide strip footing resting on stabilized soil of width 2B and depth 1B considering various soil mixtures.

Settlement (mm)						
Load intensity (kPa)	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25%GBFS with addition of 2% cement	75% LC + 25%GBFS with addition of 4% cement
10	6.28	5.76	5.08	4.68	4.47	4.21
50	20.12	18.16	15.90	13.45	12.62	11.13
100	32.43	29.10	28.10	25.31	22.11	19.81
150	57.81	48.01	42.21	38.53	31.32	28.01
200	121.81	80.66	59.09	51.29	48.97	46.06
250	295.00	151.60	94.52	68.17	61.99	58.99
300	-	301.10	153.40	91.86	78.73	73.46
350	-	-	328.30	139.21	100.90	90.47
400	-	-	-	234.00	129.31	120.52

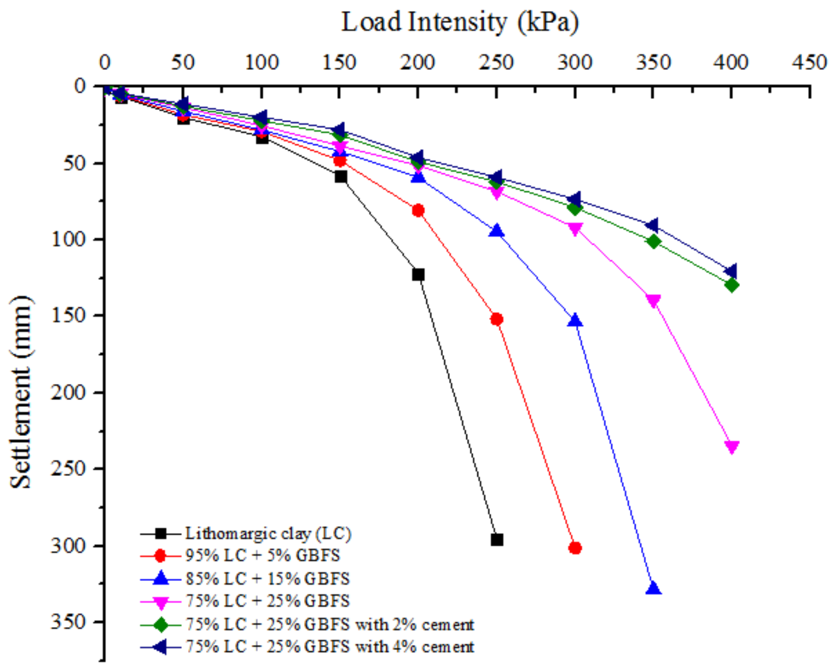

Fig. 6.9 Settlement vs load intensity for different soil combinations for strip footing of width B=1.5m resting on stabilized soil of width 2B and depth 1B

Table 6.5 Settlement of B=2m wide strip footing resting on stabilized soil of width 2B and depth 1B considering various soil mixtures.

Settlement (mm)						
Load intensity	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25%GBFS with addition of 2% cement	75% LC + 25%GBFS with addition of 4% cement
10	7.59	6.61	5.48	5.15	4.65	4.31
50	22.81	19.12	18.21	16.53	14.51	11.98
100	41.53	37.24	35.22	31.81	26.23	22.51
150	76.41	58.92	55.12	51.16	45.32	41.51
200	167.40	94.32	72.53	70.13	63.31	57.63
250	428.56	151.62	101.10	92.92	81.83	77.89
300	-	349.20	176.93	120.01	96.66	92.14
350	-	-	361.66	159.20	127.14	119.43
400	-	-	-	276.91	168.73	157.31

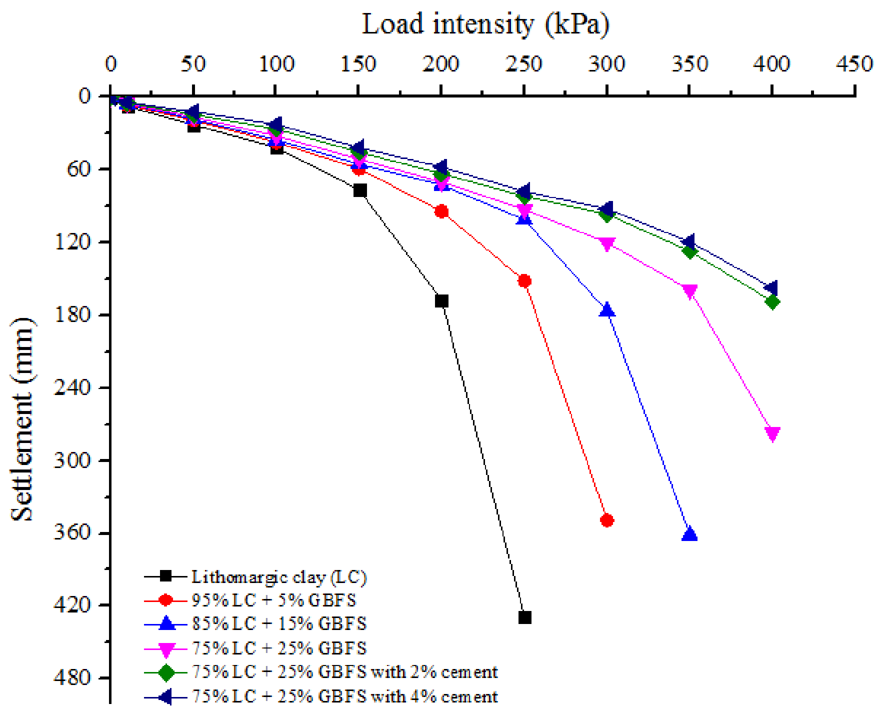

Fig. 6.10 Settlement vs load intensity for different soil combinations for strip footing of width B=2m resting on stabilized soil of width 2B and depth 1B

Case 3: Load settlement behaviour of strip footing resting on stabilized soil of width 3B and depth 2B considering various soil mixtures

Load settlement analyses using PLAXIS 2D were carried out for varying width of footing (i.e. 1m,1.5m and 2m), considering different soil-GBFS proportions and soil-GBFS-cement mixtures. Results are provided in Table 6.6 and figure 6.11 shows the graphical representation of load settlement response.

From Table 6.6 and figure 6.11, it is observed that with the increase in load intensities, the settlement increases for lithomargic soil. Upon stabilization of soil with GBFS the settlement reduces and furthermore reduces when cement is added. For a particular load intensity of 200kPa, settlement for footing resting on lithomargic clay is 77.38mm which reduces to 30.10mm when soil is replaced by 25% GBFS (61.1% reduction). On addition of 4% cement to the same 75% soil + 25% GBFS mix, settlement reduced to 17.16mm (total reduction of 77.82%). Similar trends are observed for all load intensities and for all mixes. For footing width of 1.5m, Table 6.7 and figure 6.12 and for footing width of 2m, Table 6.8 and figure 6.13 shows similar results obtained. It was also observed that as the depth of stabilized zone increased the settlement reduces. The percentage reduction in settlement is more in this case compared to case 2.

Table 6.6 Settlement for B=1m wide strip footing resting on stabilized soil of width 3B and depth 2B considering various soil mixtures.

Settlement (mm)						
Load intensity (kPa)	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25%GBFS with addition of 4% cement
10	5.32	3.97	3.81	3.42	2.74	2.22
50	14.94	12.17	9.83	7.61	4.18	2.94
100	26.71	21.51	18.73	15.21	9.96	7.01
150	40.90	30.03	26.53	21.32	15.32	12.83
200	77.38	43.79	37.57	30.10	20.16	17.16
250	165.91	59.33	48.32	42.78	25.12	21.82
300	-	89.88	69.66	52.41	35.51	30.86
350	-	-	99.36	62.37	44.29	40.39
400	-	-	-	74.06	55.40	47.43

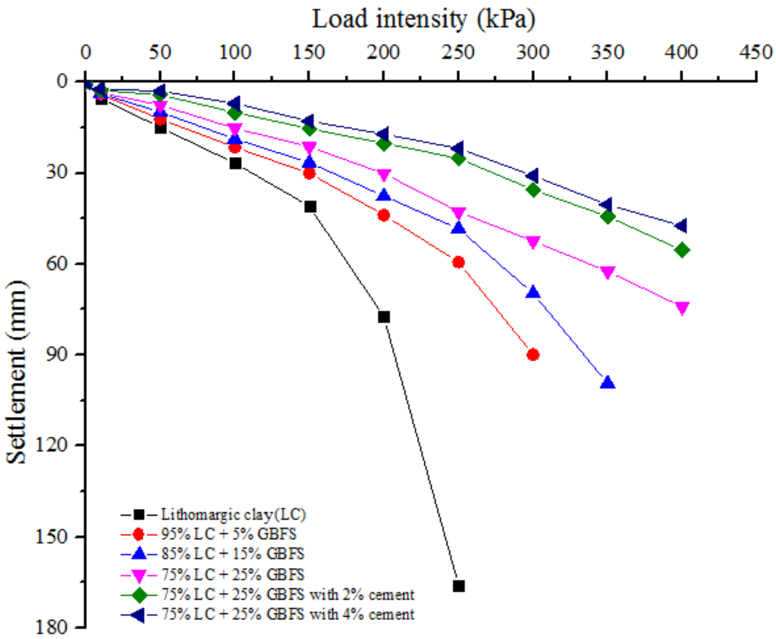

Fig. 6.11 Settlement vs load intensity for different soil combinations for strip footing of width B=1m resting on stabilized soil of width 3B and depth 2B

Table 6.7 Settlement of B=1.5m wide strip footing on stabilized soil of width 3B and depth 2B considering various soil mixtures.

Settlement (mm)						
Load intensity (kPa)	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25%GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
10	6.28	5.10	4.87	3.50	3.29	3.03
50	20.12	16.77	14.02	11.04	7.11	6.75
100	32.40	27.12	23.91	20.32	14.33	12.36
150	57.81	45.31	35.41	27.91	19.39	15.95
200	121.83	71.93	53.75	39.66	26.91	24.21
250	295.12	141.76	73.45	52.48	42.53	36.32
300	-	238.25	115.03	64.90	56.92	52.74
350	-	-	188.16	78.57	68.38	64.69
400	-	-	-	102.22	83.10	75.82

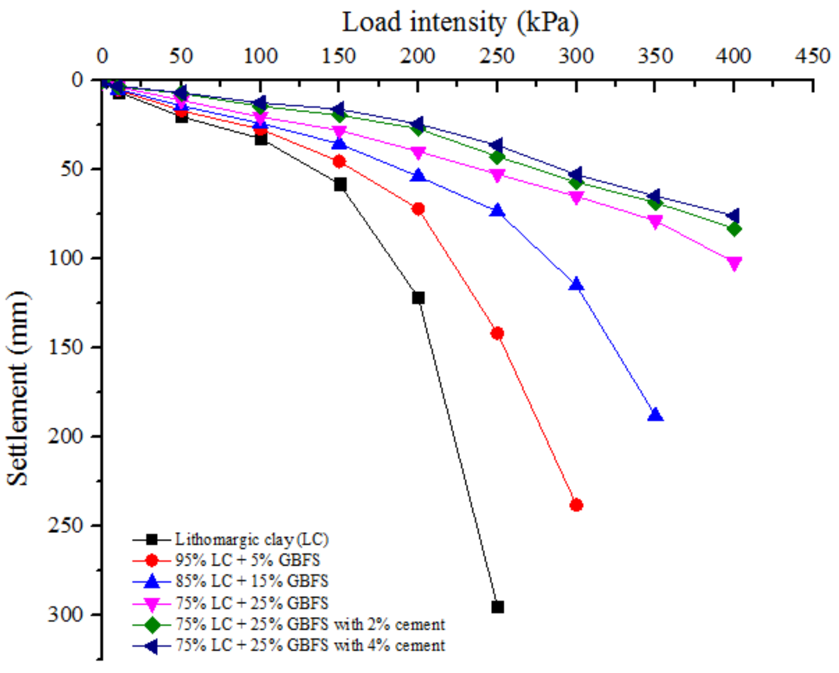

Fig. 6.12 Settlement vs load intensity for different soil combinations for strip footing of width B=1.5m resting on stabilized soil of width 3B and depth 2B

Table 6.8 Settlement of B=2m wide strip footing resting on stabilized soil of width 3B and depth 2B considering various soil mixtures.

Settlement (mm)						
Load intensity (kPa)	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
10	7.59	5.375	4.11	3.92	3.57	3.13
50	22.81	17.65	15.33	12.77	8.68	7.63
100	41.51	34.69	29.16	23.51	16.32	14.65
150	76.41	54.17	45.12	34.79	24.12	21.2
200	167.4	75.72	62.13	53.42	40.01	34.34
250	428.5	106.8	80.03	68.38	57.2	53.85
300	-	192.1	129.19	83.93	70.68	66.86
350	-	-	220.13	100.1	84.11	80.01
400	-	-	-	117.32	100.5	93.05

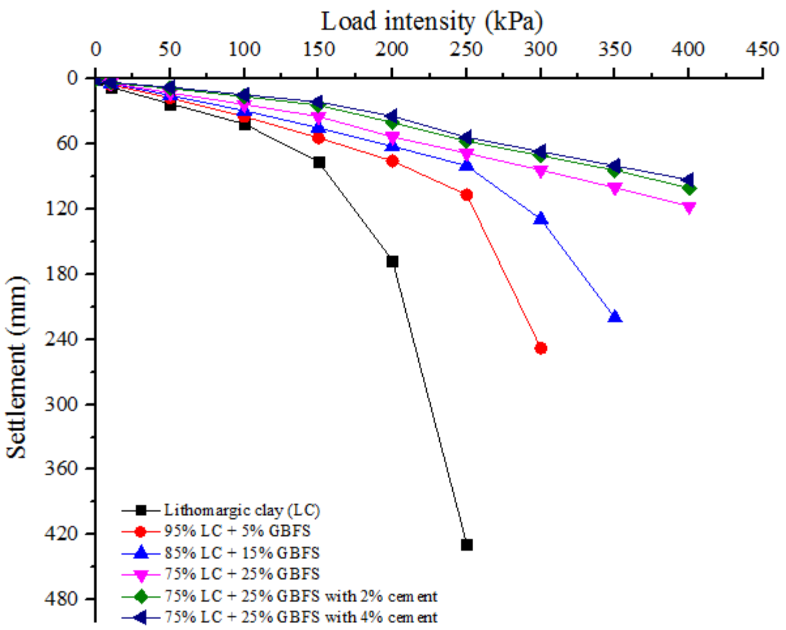

Fig. 6.13 Settlement vs load intensity for different soil combinations for strip footing of width B=2m resting on stabilized soil of width 3B and depth 2B

6.3.3 Increase in net allowable pressure for stabilized soil

Table 6.9 Net allowable pressure for various soil mixtures for footing resting on stabilized soil of width 2B and depth 1B for an allowable settlement of 25mm

Combinations	B=1m		B=1.5m		B=2m	
	Net allowable pressure (kPa)	% increase in net allowable pressure	Net allowable pressure (kPa)	% increase in net allowable pressure	Net allowable pressure (kPa)	% increase in net allowable pressure
Lithomargic clay (LC)	93	-	70	-	56	-
95% LC + 5% GBFS	109	17.2%	81	15.7%	67	19.6%
85% LC + 15% GBFS	116	24.7%	87	24.3%	70	25%
75% LC + 25% GBFS	129	38.7%	99	41.4%	78	39.3%
75% LC+ 25% GBFS with addition of 2% Cement	153	64.5%	116	65.7%	94	67.8%
75% LC + 25% GBFS with addition of 4% Cement	175	88.2%	132	88.6%	107	91.1%

For case 2, for various combinations of soil mixtures, values of net allowable pressure for allowable settlement of 25mm are tabulated in Table 6.9. Table 6.10 provides similar results for case 3.

Table 6.10 Net allowable pressure for various soil mixtures for footing resting on stabilized soil of width 3B and depth 2B for an allowable settlement of 25mm

Combinations	B=1m		B=1.5m		B=2m	
	Net allowable pressure (kPa)	% increase in net allowable pressure	Net allowable pressure (kPa)	% increase in net allowable pressure	Net allowable pressure (kPa)	% increase in net allowable pressure
Lithomargic clay (LC)	93	-	70	-	56	-
95% LC +5% GBFS	121	30.1%	90	28.6%	72	28.6%
85% LC +15% GBFS	140	50.5%	105	50%	85	51.8%
75% LC + 25% GBFS	171	83.9%	131	87.1%	107	91.1%
75% LC + 25% GBFS with addition of 2% cement	248	166.7%	187	167.1%	153	173.2%
75% LC + 25% GBFS with addition of 4% cement	268	188.2%	203	190%	165	194.6%

From Table 6.9 and Table 6.10, we see that the increase in net allowable pressure increases with increase in GBFS content and further more increases with addition of cement to 75% soil + 25%GBFS mix. The increase in net allowable pressure increases with increase in the depth of stabilized area below the footing. For a footing width of B=2m, the percentage increase in net allowable pressure is39.3% for 75% soil + 25% GBFS mix for 1B depth of stabilized soil (Table 6.9). The net allowable pressure increased by 91.1% for the same 75% soil + 25% GBFS mix for 2B depth of stabilized soil (Table 6.10). With addition of 4% cement to the optimized mix (75% soil + 25%GBFS), the net allowable pressure increased by 91.1% for 1B depth stabilized soil (Table 6.9) and 194.6% increase is found for 2B depth of stabilized soil when compared with footing resting on lithomargic clay (Table 6.10). Thus there is very good improvement in load settlement behaviour of strip footing when lithomargic clay is stabilized using GBFS and cement.

6.4 STABILITY ANALYSES OF AN EMBANKMENT SLOPE (SOON AFTER CONSTRUCTION) USING PLAXIS 2D

Slope instability is a major concern in the areas where failures causes catastrophic destruction. The failures might be triggered by internal or external factors that cause imbalance to natural forces. An internal triggering factor is the factor that causes failure due to internal changes, such as increasing pore water pressure and or imbalanced forces developed due to external load.

Short-term analyses refers to conditions such as soon after construction where in, there is no drainage occurring within this period. Short-term stability analysis of an embankment slope is important.

In slope stability analysis, the factor that is very often in doubt is the shear strength of the soil. The loading is known more accurately because usually it consists of the self-weight of the slope. The Factor of Safety (FoS) is taken as a ratio of the available shear strength to that required to keep the slope stable. For highly unlikely loading conditions, accepted factors of safety can be as low as 1.2-1.25.e.g., for dam situations based on seismic effects, or where there is rapid drawdown of the water level in a reservoir. Generally, accepted factor of safety is 1.5 for undrained analysis and 1.3 for combined or drained analysis.

An analysis of slope stability begins with the hypothesis that the stability of a slope is the result of downward or motivating forces (i.e., gravitational) and resisting (or upward) forces. The resisting forces must be greater than the motivating forces for a slope to be stable. The relative stability of a slope (or how stable it is at any given time) is typically conveyed by geotechnical engineers through a FoS defined as

$$FoS = \frac{\sum R}{\sum M} \tag{6.1}$$

Where, $\sum R$ is the total resisting moment and $\sum M$ is the total motivating or driving moment.

These analyses provide two useful results: (i) the normal stress on the shear surface and (ii) the shear stress required for equilibrium. The factor of safety is the ratio of the shear strength of the soil divided by the shear stress required for equilibrium. The normal stresses along the slip surface are needed to evaluate the shear strength.

The factor of safety in PLAXIS is computed using phi-c reduction at each case of slope modeling. In this type of calculation the incremental multiplier, Msf is used to specify the increment of the strength reduction of the first calculation step. The strength parameters are reduced successively in each step until all the steps have been performed. The final step should result in a fully developed failure mechanism, if not the calculation must be repeated with a larger number of additional steps. Once the failure mechanism is reached or at the failure stage of the slope, the FoS is given by,

$$FoS = RF_{at\ failure} = \frac{Available\ strength}{Strength\ at\ \ failure} = \sum Msf\ \ value\ of\ Msf\ at\ \ failure \tag{6.2}$$

The c−φ method is based on the reduction of the cohesion (c) and the tangent of the friction angle (tanφ) of the soil. The parameters are reduced in steps until the soil mass fails. PLAXIS uses a factor (RF) to relate the reduction in the parameters during the calculation at any stage with the input parameters, which is given by,

$$RF = \frac{\tan\phi_{input}}{\tan\phi_{reduced}} = \frac{c_{input}}{c_{reduced}} \tag{6.3}$$

Where, *RF* = the reduction factor at any stage during calculations, $\tan\phi_{input}$ and c_{input} are the input parameters of the soil, $\tan\phi_{reduced}$ and $c_{reduced}$ are the reduced parameters calculated by the program.

In embankment slope analyses, though there are many situations such as long term stability, steady seepage condition, effect of water table, live load on embankment etc., this work is limited to stability analyses soon after construction and obtaining improvement in factor of safety values due to stabilization of lithomargic clay. Results obtained from the analyses are discussed in the subsequent sections.

6.4.1 Various cases considered in the analyses

For the stability analyses of embankment slopes (soon after construction), following three cases are considered.

Case 1: Embankment is made up of lithomargic clay and resting on lithomargic clay. Slope heights 4m, 6m, 8m, 10m, and 12m and embankment slope angles of 30°, 45°, 60°, and 75° were considered [figure 6.14(a)].

Case 2: Embankment with stabilized soil resting on lithomargic clay. Slope heights: 4m, 6m, 8m, 10m and 12m. Embankment slope angles: 30°, 45°, 60° and 75° [figure 6.14(b)].

Case 3: Embankment with stabilized soil and soil below the embankment, stabilized for a depth of half the embankment height. Embankment heights: 4m, 6m, 8m, 10m and 12m. Embankment slope angles: 30°, 45°, 60° and 75° [figure 6.14(c)].

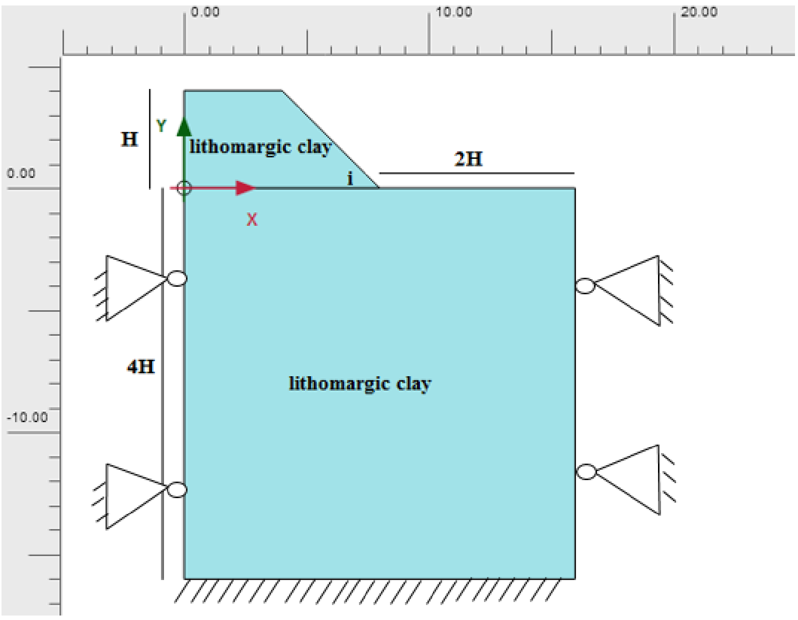

Fig. 6.14 (a) - Case 1

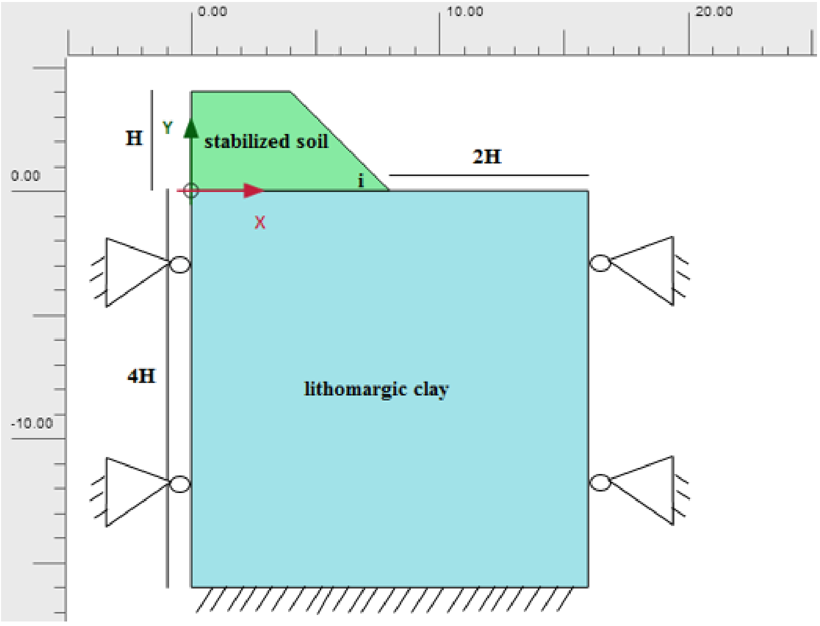

Fig. 6.14 (b) - Case 2

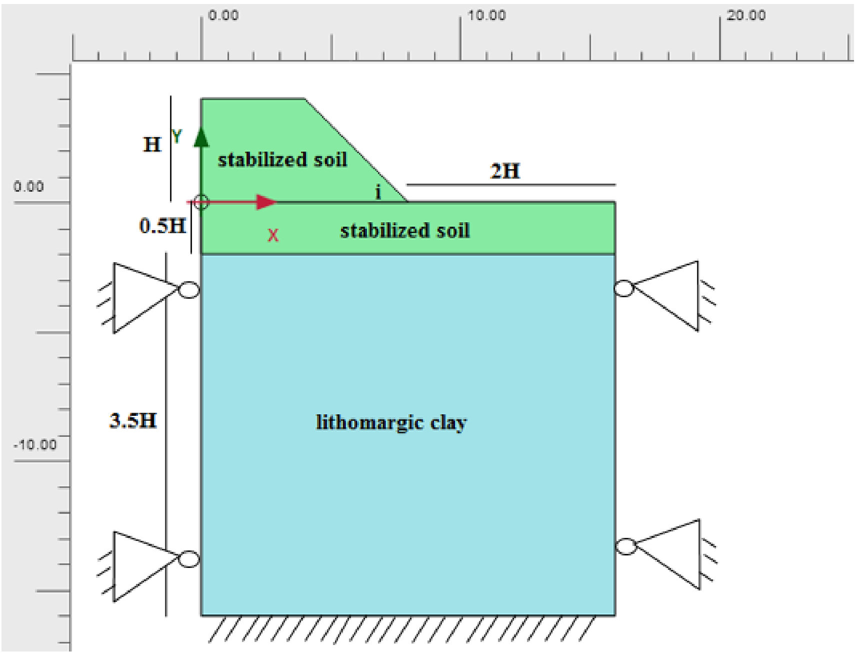

Fig. 6.14 (c) - Case 3

Fig. 6.14 Various cases considered for the analyses.

6.4.2 Results of slope stability analyses for embankment made up of lithomargic clay and resting on lithomargic clay (Case 1)

A series of finite element analyses using PLAXIS 2D on stability of embankment slope were performed and values of factor of safety were established for different heights and slope angles. Results are provided in Table 6.11(a) and figure 6.15. Analysis using limit equilibrium method (Nayak and Padmaja 2006) were also done for typical cases to validate the results. In addition, the percentage difference were recorded and presented in Table 6.11(b). Difference of 0.32% to 1.87% for slope angle of 30 degrees, 8.1% to 13.4% for slope angle of 45 degrees and 19.85% to 22.8% for slope angle of 60 degrees were observed between limit equilibrium method and finite element method.

Table 6.11(a) Variation of FoS for different embankment heights (Case 1)

Factor of Safety				
Embankment slope angle, i				
Height	30°	45°	60°	75°
4m	3.10	2.78	2.64	2.47
6m	2.42	2.15	1.93	1.70
8m	1.98	1.75	1.54	1.32
10m	1.60	1.47	1.31	1.10
12m	1.31	1.24	1.06	-

Table 6.11(b) Comparision of FoS obtained from PLAXIS and limit equilibrium method (LEM)

Height	30°			45°			60°		
	PLAXIS	LEM	% diffe-rence	PLAXIS	LEM	% diffe-rence	PLAXIS	LEM	% diffe-rence
4m	3.10	3.09	0.32	2.78	2.51	9.71	2.64	2.05	22.35
6m	2.42	2.40	0.83	2.15	1.86	13.49	1.93	1.49	22.80
8m	1.98	1.95	1.51	1.75	1.54	12.00	1.54	1.22	20.78
10m	1.60	1.57	1.87	1.47	1.35	8.16	1.31	1.05	19.85

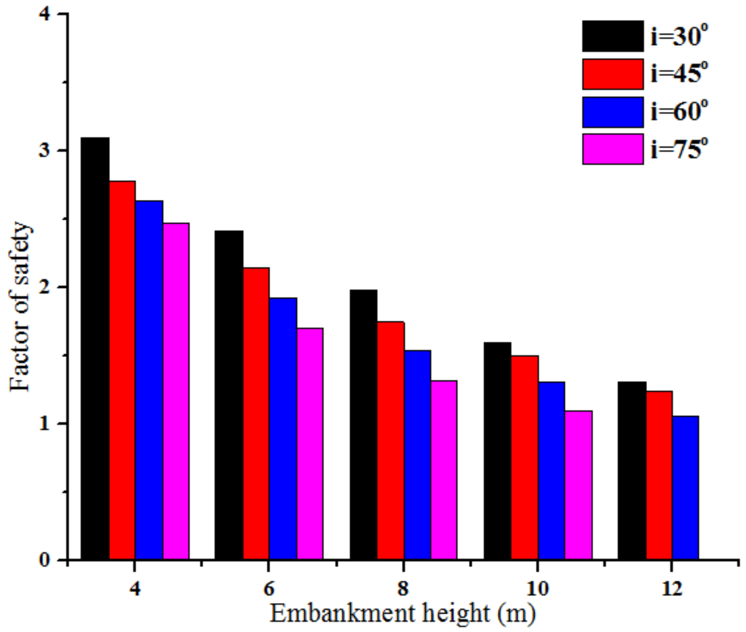

Fig. 6.15 Variation of factor of safety with embankment constructed with lithomargic clay with varying embankment slope angle and varying embankment height.

It is noticed from Table 6.11 and figure 6.15 that increase in height of the embankment resulted in decrease in values of factor of safety. As expected, with the increase in slope angles, the FoS decreases.

6.4.3 Results of slope stability analyses for embankment with stabilized soil resting on lithomargic clay (Case 2)

Considering slope heights of 4m, 6m, 8m, 10m and 12m, for various combinations of soil mixes, analyses were carried out and the results are provided in Table 6.12 for slope angle of 30°. Graphical representation of the same is shown in figure 6.16. Similar analyses were carried out for slope angles of 45°, 60° and 75° and results are presented in Tables 6.13, 6.14 and 6.15 respectively. Figures 6.17, 6.18 and 6.19 provides variation of factor of safety for slope angles of 45°, 60° and 75° considering various soil mix combinations.

It is evident from Table 6.12 and figure 6.16 that for an embankment slope angle of 30°, the FoS decreases with increase in embankment height. With the increase in the GBFS replacing the soil in the embankment, the FoS increases. The improvement in FoS is larger when embankment is stabilized with 25% optimum GBFS content and cement.

Addition of cement to soil-GBFS mix increases the factor of safety considerably, compared to soil stabilized with GBFS alone. Similar trend is observed for slope angles of 45°, 60°and 75° [Tables 6.13-6.15 and figures 6.17-6.19]

Table 6.12 Variation of FoS for slope angle of 30° (Case 2)

Height	Factor of Safety for i=30°					
	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
4m	3.10	3.33	3.6	3.86	5.13	5.61
6m	2.42	2.58	2.78	2.99	4.08	4.32
8m	1.98	2.11	2.27	2.46	3.46	3.76
10m	1.60	1.75	1.87	2.01	3.00	3.35
12m	1.31	1.56	1.73	1.86	2.71	3.02

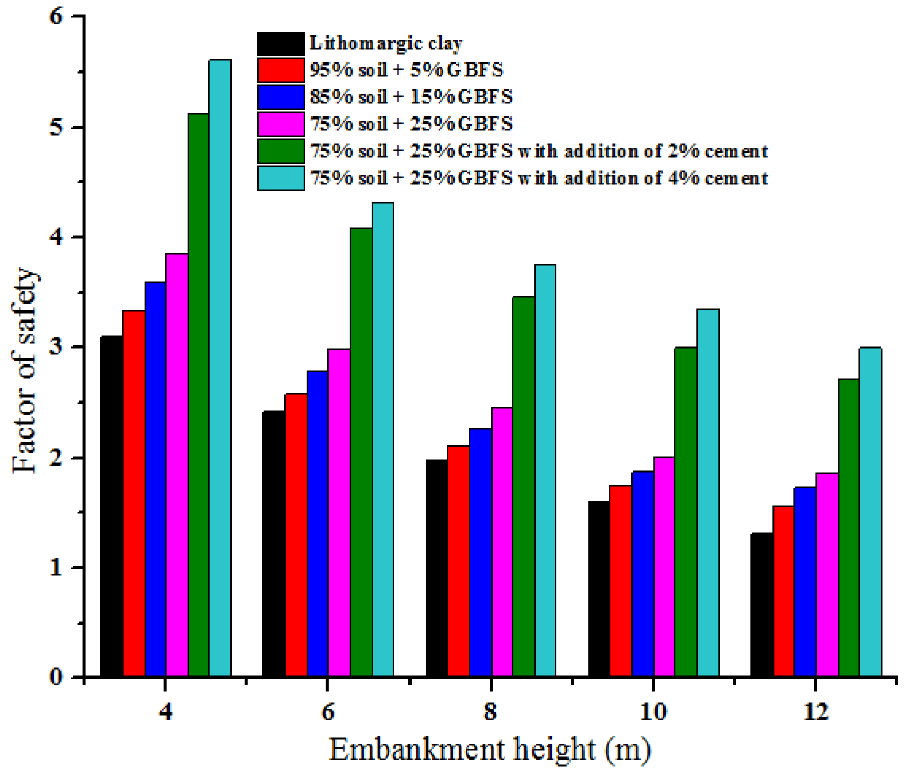

Fig.6.16 Variation of FoS for slope angle of 30° (Case 2)

Table 6.13 Variation of FoS for slope angle of 45° (Case 2)

Height	Factor of Safety for i=45°					
	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
4m	2.78	3.00	3.23	3.68	4.76	4.99
6m	2.15	2.29	2.49	2.70	3.9	4.16
8m	1.75	1.87	2.05	2.25	3.34	3.62
10m	1.50	1.61	1.78	1.97	2.96	3.27
12m	1.24	1.42	1.50	1.71	2.55	2.96

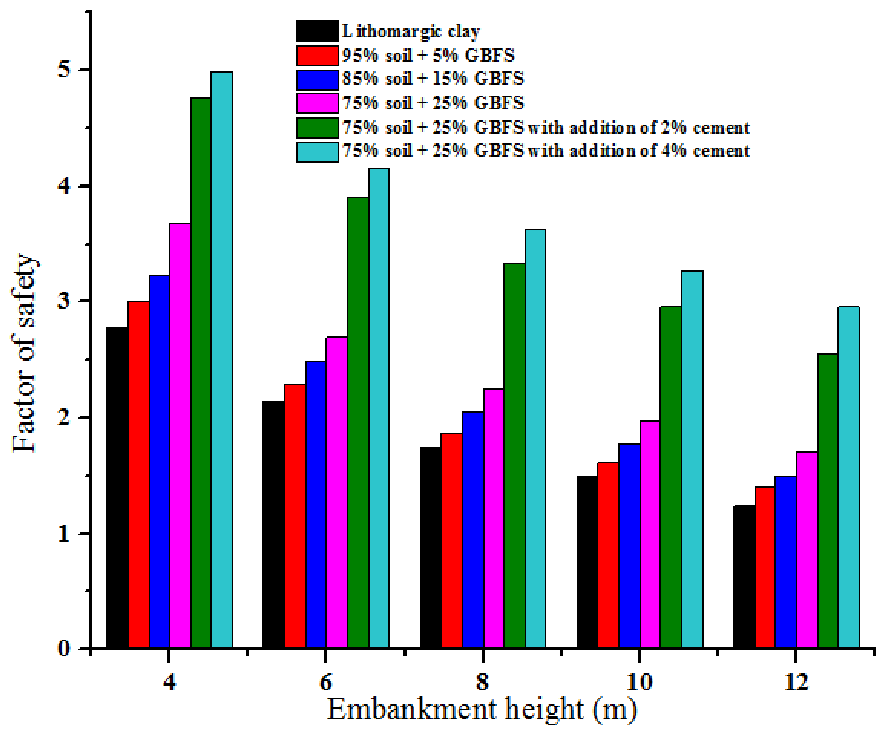

Fig.6.17 Variation of FoS for slope angle of 45° (Case 2)

Table 6.14 Variation of FoS for slope angle of 60° (Case 2)

Height	Factor of Safety for i=60°					
	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
4m	2.64	2.91	3.18	3.45	4.34	4.59
6m	1.93	2.10	2.34	2.61	3.35	3.60
8m	1.54	1.72	1.95	2.18	2.83	3.07
10m	1.31	1.49	1.67	1.87	2.51	2.74
12m	1.09	1.27	1.4	1.56	2.20	2.38

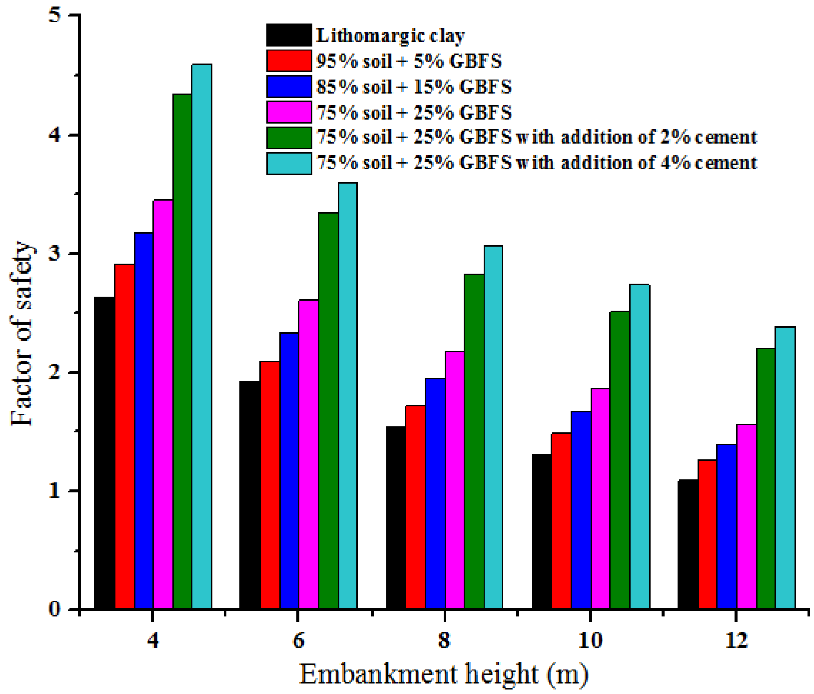

Fig.6.18 Variation of FoS for slope angle of 60° (Case 2)

Table 6.15 Variation of FoS for slope angle of 75° (Case 2)

Height	Factor of Safety for i=75°					
	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
4m	2.47	2.75	3.05	3.26	3.87	4.09
6m	1.70	1.95	2.26	2.50	2.92	3.20
8m	1.32	1.56	1.82	2.02	2.41	2.67
10m	1.10	1.34	1.54	1.74	2.10	2.24
12m	-	1.10	1.23	1.40	1.75	1.90

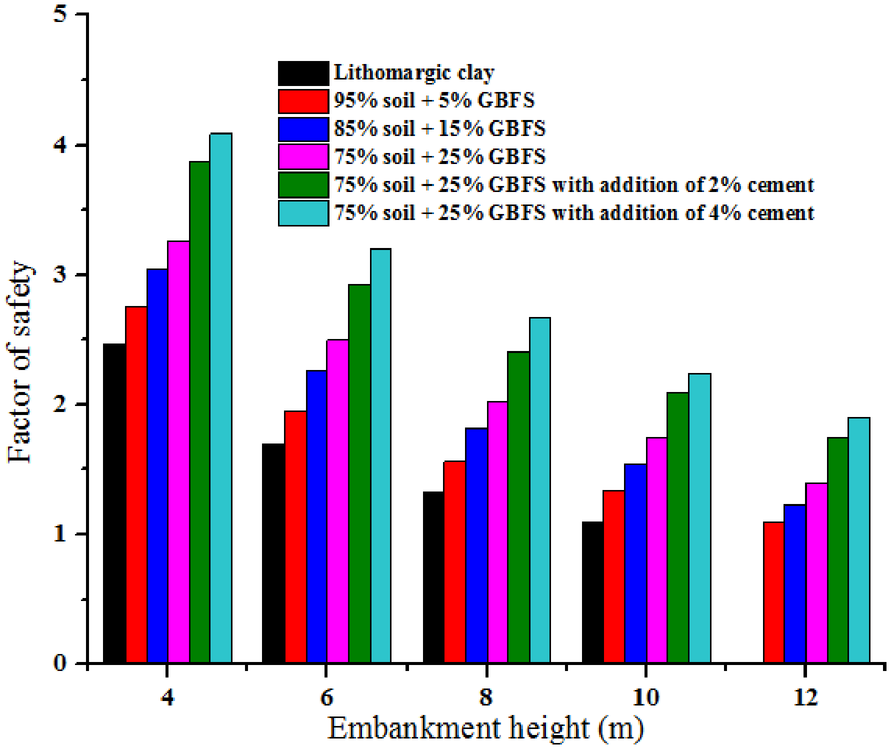

Fig.6.19 Variation of FoS for slope angle of 75° (Case 2)

6.4.4 Results of slope stability analyses for embankment with stabilized soil and soil below embankment stabilized for a depth of half the embankment height (Case 3)

Table 6.16 provides the results from the analyses, carried out for different slope heights of 4m, 6m, 8m, 10m and 12m considering various combinations of soil mixes, for a slope angle of 30°. Graphical representation of the same is shown in figure 6.20. Similarly, analyses were carried out for slope angles of 45°, 60° and 75° and results are presented in Tables 6.17, 6.18 and 6.19 respectively. Figures. 6.21, 6.22 and 6.23 provides variation of factor of safety for slope angles of 45°, 60° and 75° considering various soil mix combinations.

From Table 6.16 and figure 6.20, for a slope inclination of 30°, the FoS reduces with the increase in embankment height. It further increases with the increase in the GBFS replacing the soil in the embankment. The FoS also increased as the soil below the embankment is stabilized for a depth of half the embankment height. The improvement in FoS is higher when embankment is stabilized with 25% optimum GBFS content and cement. Addition of cement to the soil-GBFS mix increases the factor of safety considerably, compared to soil stabilized only with GBFS. Similar trend is observed for slope angles of 45°, 60°and 75° [Tables 6.17-6.19 and figures 6.21-6.23]

Table 6.16 Variation of FoS for slope angle of 30° (Case 3)

Height	Factor of Safety for i=30°					
	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
4m	3.13	3.63	4.24	4.64	7.19	8.52
6m	2.42	2.86	3.05	3.73	6.01	6.82
8m	1.98	2.42	2.89	3.23	5.66	6.25
10m	1.60	2.14	2.44	2.82	4.95	5.74
12m	1.31	1.87	2.25	2.58	4.45	5.20

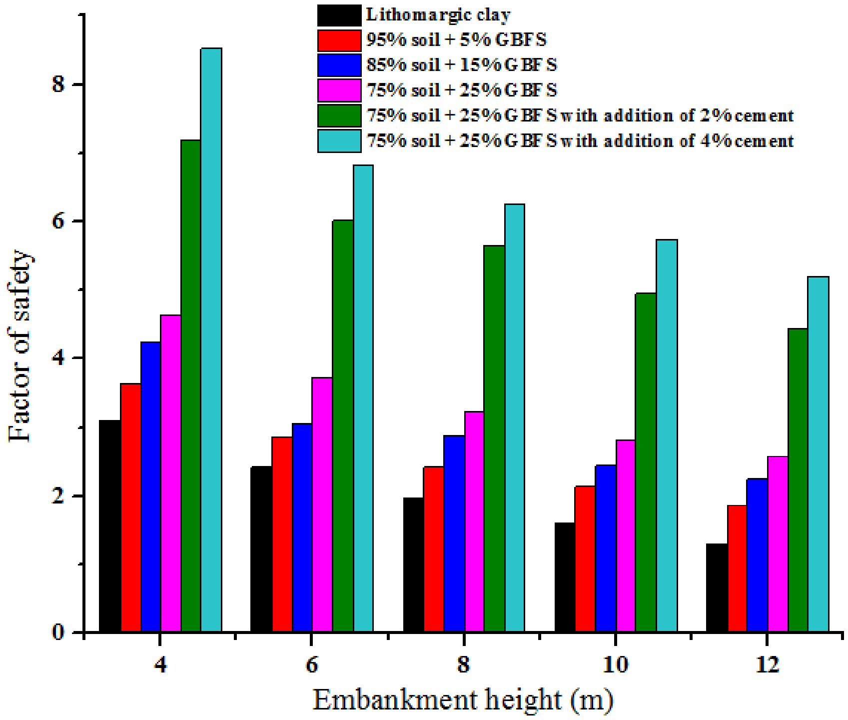

Fig.6.20 Variation of FoS for slope angle of 30° (Case 3)

Table 6.17 Variation of FoS for slope angle of 45° (Case 3)

Height	Factor of Safety for i=45°					
	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
4m	2.78	3.39	4.01	4.49	7.09	8.17
6m	2.15	2.68	3.21	3.61	5.80	6.60
8m	1.75	2.23	2.70	3.16	5.31	6.03
10m	1.53	1.91	2.41	2.72	4.84	5.50
12m	1.24	1.62	2.09	2.31	4.21	5.02

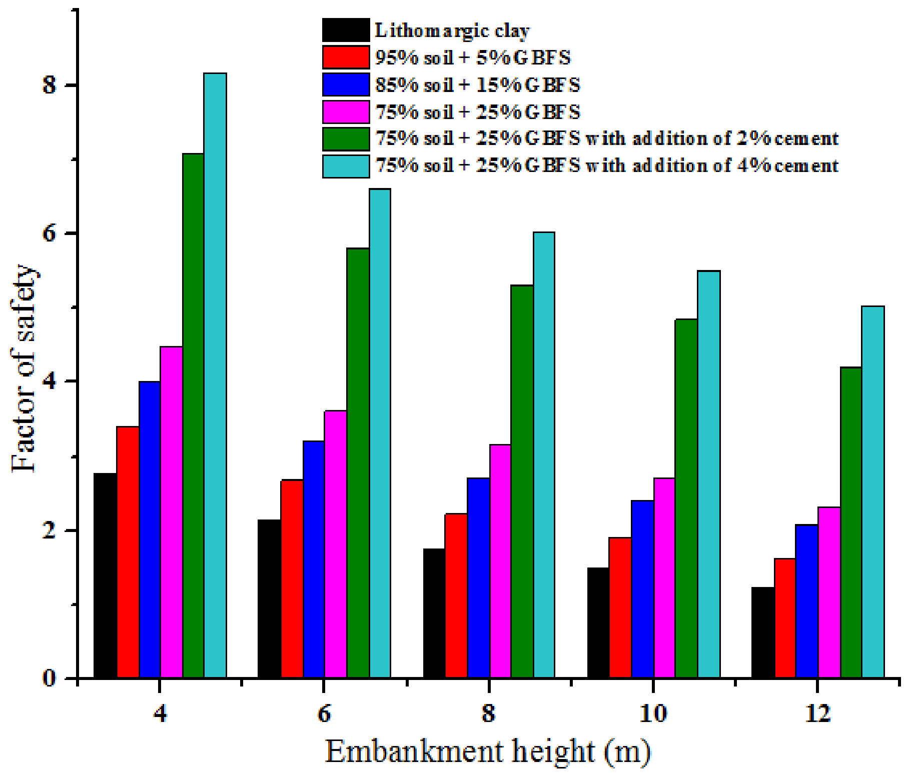

Fig. 6.21 Variation of FoS for slope angle of 45°(Case 3)

Table 6.18 Variation of FoS for slope angle of 60° (Case 3)

Height	Factor of Safety for i=60°					
	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
4m	2.64	3.34	3.97	4.35	6.81	7.77
6m	1.93	2.43	3.1	3.54	5.71	6.53
8m	1.54	1.93	2.59	3.1	5.12	5.86
10m	1.31	1.64	2.24	2.6	4.73	5.47
12m	1.09	1.35	1.66	2.09	3.92	4.87

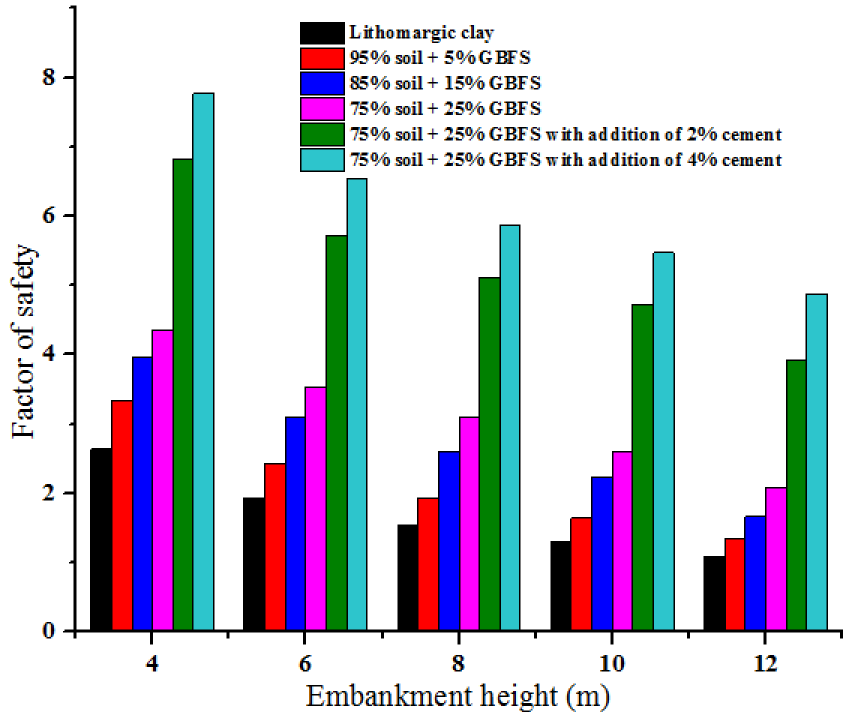

Fig.6.22 Variation of FoS for slope angle of 60°(Case 3)

Table 6.19 Variation of FoS for slope angle of 75° (Case 3)

Height	Factor of Safety for i=75°					
	Lithomargic clay (LC)	95% LC + 5% GBFS	85% LC + 15% GBFS	75% LC + 25% GBFS	75% LC + 25% GBFS with addition of 2% cement	75% LC + 25% GBFS with addition of 4% cement
4m	2.47	3.16	3.87	4.19	6.53	7.52
6m	1.70	2.13	2.90	3.24	5.44	6.19
8m	1.32	1.64	2.24	2.99	5.07	5.67
10m	1.11	1.38	1.89	2.52	4.66	5.42
12m	-	1.25	1.41	1.88	3.70	4.63

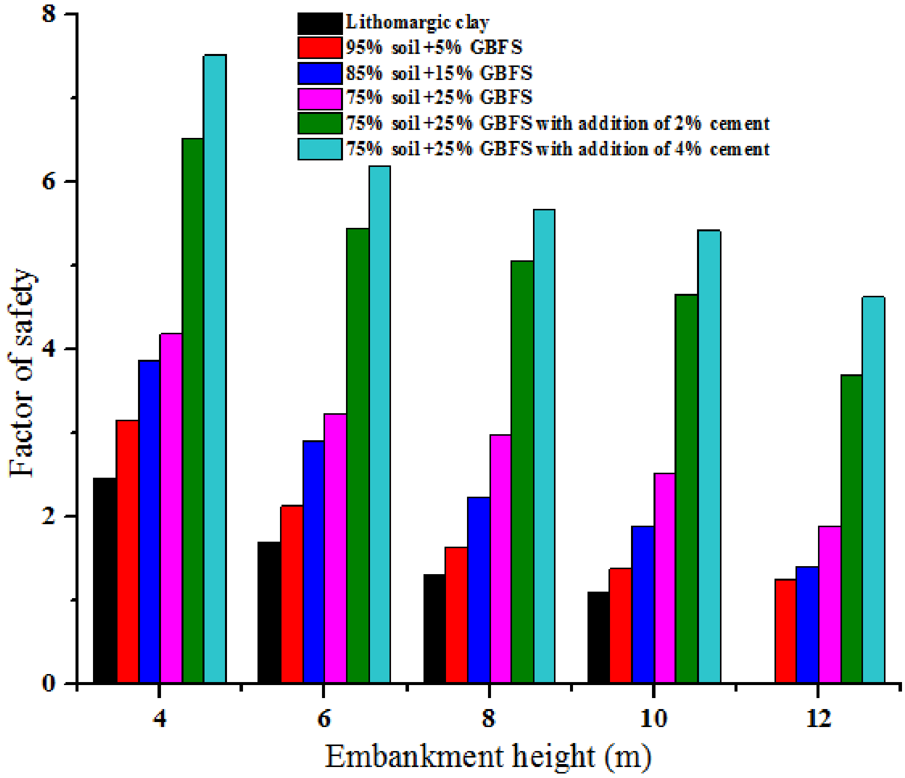

Fig.6.23 Variation of FoS for slope angle of 75° (Case 3)

6.4.5 Effect of stabilization on Factor of Safety (FoS)

The values of factor of safety obtained for case 1, case 2 and case 3 are compared to understand the effect of stabilization on FoS obtained for embankment slopes (soon after construction). For a typical case of H=10m, FoS values for all the three cases are presented in Table 6.20. Graphical representation is provided in figure 6.24 and figure 6.25.

Table 6.20. Effect of stabilization on FoS for H=10m

Factor of safety (FoS)					
		i=30°	i=45°	i=60°	i=75°
Case 1	Lithomargic clay (LC)	1.60	1.47	1.31	1.10
Case 2	75% LC + 25% GBFS	2.01	1.97	1.87	1.74
Case 2	75% LC + 25% GBFS + 2% cement	3.00	2.96	2.51	2.10
Case 3	75% LC + 25% GBFS	2.82	2.72	2.60	2.52
Case 3	75% LC + 25% GBFS with addition of 2% cement	4.95	4.84	4.73	4.66

Case 1: Embankment of lithomargic clay resting on lithomargic clay

Case 2: Embankment with stabilized soil and resting on lithomargic clay.

Case 3: Embankment and soil below embankment, stabilized for a depth of half the embankment height.

From Table 6.20 and figure 6.24 and figure 6.25, it is clear that there is good improvement in FoS values from case 1 to case 2. It further improves for case 3. For example, considering i=60°for H=10m, for case 1, FoS is 1.31, which increases to 1.87 when embankment is made up of 75% lithomargic clay + 25% GBFS (increase by 42.7%). By adding 2% cement to optimum mix, FoS increases to 2.51 (total increase is 91.6%). For case 3 (embankment and soil below embankment stabililzed for depth of half the embankment height), considering i=60°and H=10m, FoS value for 75% soil + 25% GBFS mix is 2.60 (increase by 98.5% compared to case 1) and by adding 2% cement, FoS value increases to 4.73 (increase by 261% compared to case 1).

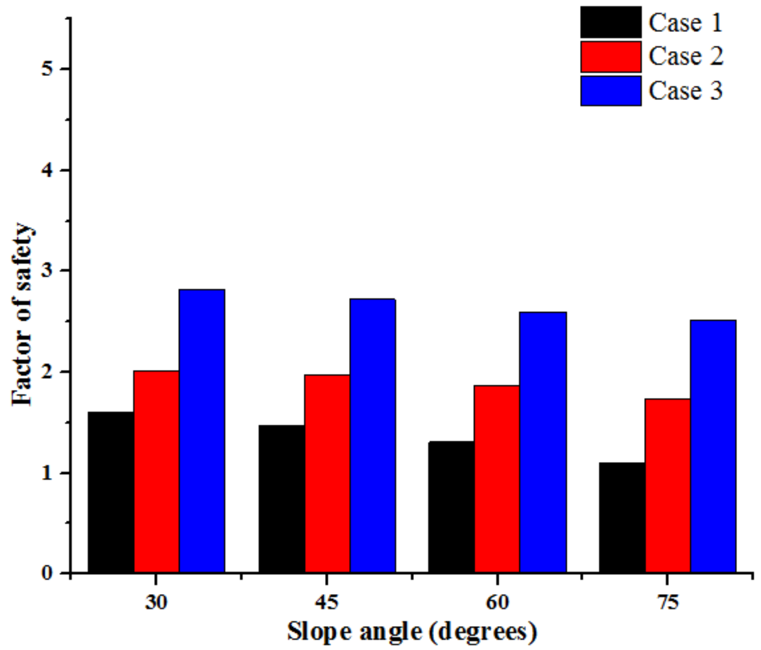

Fig. 6.24 Variation of FoS for different cases for 75% LC + 25% GBFS

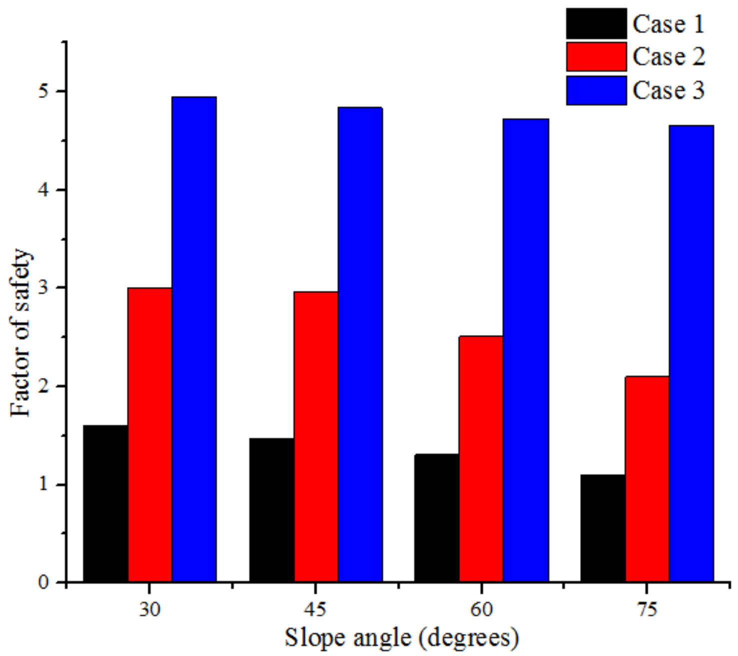

Fig. 6.25 Variation of FoS for different cases for 75% LC + 25% GBFS with addition of 2% cement

CHAPTER 7

MANUFACTURE AND TESTING OF COMPRESSED STABILIZED EARTH BLOCKS

7.1 INTRODUCTION

Stabilization is a process of mixing admixtures with soil to improve its volume stability, strength, permeability and durability. Amongst the variety of soil stabilizers used, cement has been the most popular stabilizer in the manufacture of compressed stabilized earth blocks (CSEBs). Compressed earth block or pressed soil block is a building material made primarily from damp soil compressed at high pressure to form blocks. These blocks uses a mechanical press to form blocks from inorganic and non-expansive soil. If the blocks are stabilized with cement, they are called as compressed stabilized earth block (Nagarajet al. 2014).

Attempts to utilize GBFS in combination with cement as a stabilizer to achieve desirable properties of CSEBs have not been reported. As GBFS is known to impart strength in the long term, its utilization in some proportions as a replacement to cement may be beneficial. Utilization of granulated blast furnace slag (GBFS) not only helps in the bulk utilization in production of CSEBs due to its latent pozzolanic properties but also helps in the increase in the rate of disposal of these slags. Since the rate of hydration and pozzolanic reactions is slow in GBFS, it can be improved with the addition of cement. Hence, an attempt has been made to understand the role of slag in combination with cement as a stabilizer for improving the long term properties of CSEBs to optimize the use of stabilizer and improve the strength of blocks. This work is aimed at contributing towards improvising the existing technology of manufacture of unfired earth blocks and in the reduction of cement utilization.

7.2 MATERIALS USED

Lithomargic clay, lateritic soil, granulated blast furnace slag and ordinary Portland cement are used in the manufacture of compressed stabilized earth blocks**.**

7.2.1 Soils: Two different types of soils were used in the current study, as these were the locally available soils. One is lateritic soil which is reddish/brownish in colour having hard and porous nature, while the other being lithomargic clay, which is whitish, pinkish/yellowish in colour, mainly consisting of silt/sand particles. Lithomargic clay is found below hard lateritic soil at depths varying from 2-5m throughout the southwestern belt of India. The properties of these soils obtained from laboratory testing are provided in Table 7.1 and their particle distribution curves are presented in figure 7.1.

Table 7.1 Properties of lithomargic clay and lateritic soil

Sl. No.	Properties	Lithomargic clay	Lateritic soil
1	*Particle size distribution*		
	Gravel size (%)	02	04
	Sand size (%)	10	60
	Silt size (%)	59	20
	Clay size (%)	29	16
2	*Atterberg's limits*		
	Liquid limit, w_L (%)	47	44
	Plastic limit, w_p (%)	31	32
	Shrinkage limit, w_s (%)	28	30
	Plasticity index, I_P (%)	16	12
3	IS Classification	MI	SM-SC
4	Specific Gravity	2.52	2.62
5	*Compaction characteristics*		
	(Standard Proctor Test)		
	Maximum dry unit weight γ_d (kN/m^3)	14.2	16.9
	Optimum moisture content (%)	28	21
6	Unconfined compressive strength (kPa)	232	406

7.2.2 Granulated Blast Furnace Slag (GBFS): The slag obtained from Kirloskar Ferrous Industries Limited was used. Its properties are reported in chapter 4.

7.2.3 Ordinary Portland Cement (OPC): 53 grade cement was used for the study.

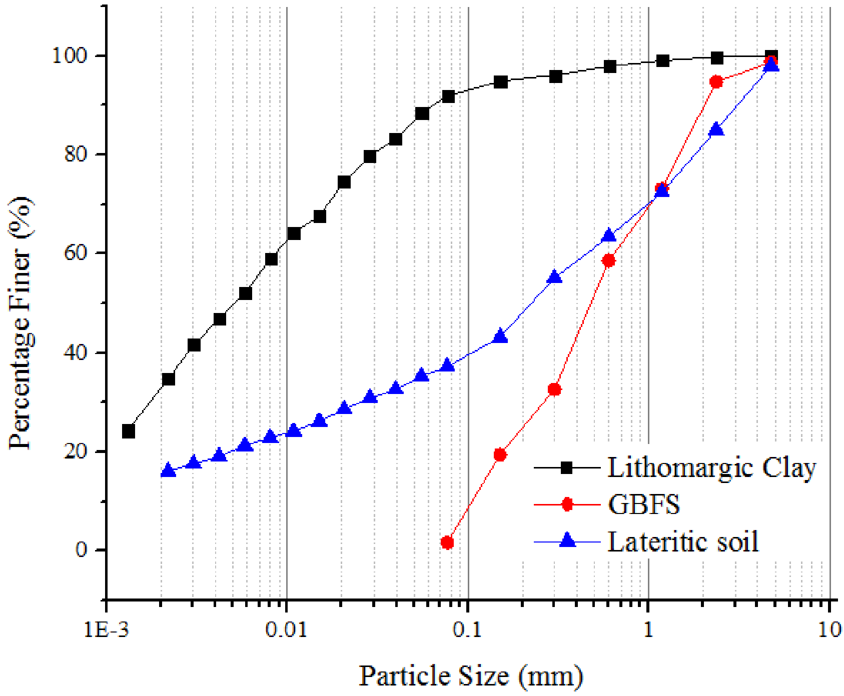

Fig. 7.1 Grain size distribution of lateritic soil, lithomargic clay and GBFS

7.3 MANUFACTURE AND TESTING

Screening: The soil was loosened in a uniform manner by removing any roots, leaves, twigs etc. i.e. any undesirable materials.

Drying: The soil was dried in the oven for 24 hours at 110 +/- 5°C to remove any moisture.

Pulverizing: The soil was broken down to a size passing through 4.75mm sieve, since the particles retained on them were considered as not suitable for block preparation.

Mixing: The requisite quantities of the materials i.e. soil, GBFS and cement was calculated. Weigh batching was done to control the block density. Mixing was done properly to ensure good quality blocks. Dry materials were mixed thoroughly to get a homogeneous mix. The amount of water that was required to obtain a good quality block was determined by making a good intact ball without sticking to the hand from initial trials. Requisite quantity of potable water was added to this mix by spraying and then turned over many times until all the required water was added. The process is repeated until all the particles were uniformly wetted. Figure 7.2 shows the cleaning and lubricating the block mould. Dry mixing of all the constituents and addition of water to the dry mix are shown in figures 7.3 and 7.4 respectively.

Compression: Block making machine was used for compressing the block. Generally the single acting ram generates a compaction pressure in the range of 2-3MPa.This process consists of compressing the wet mix after it has been placed in the mould, through static compression to get constant volume blocks. The compressed block was then removed from the machine. The ejected samples were weighed and labeled according to their mix. The size of the blocks obtained is of 30.5 cm x 14.3 cm x 10.5 cm. Figure 7.5 shows the method of compressing the block by pulling down the lever arm. After compression process is complete, soil blocks were ejected as shown in figure 7.6

Curing: Blocks were cured for 28 days. Figure 7.7 shows the storage and curing of the prepared blocks.

Testing: Compression tests were carried out in a compression testing machine. Both wet and dry compression tests were carried out. After 26 days of curing, the blocks were immersed in water for 2 days (6 blocks for each combination) and then the blocks were removed from water. The surfaces of the blocks were wiped dry and their mass and dimensions were measured. Dry compressive strength test were conducted on another set of similar samples after 28 days of curing (without immersion in water). The samples were then placed in the compression testing machine and loaded. Iron plates of 10mm thick were placed on either surfaces of the block before the application of load, as shown in figure 7.8.

Tests for water absorption were carried out on another set of similar samples (six blocks for each combination). The blocks were kept in oven for drying (figure 7.9) and their weight were taken accurately. Then the blocks were submerged in clean water for one day. After 24 hours, the surface were wiped dry and the blocks were weighed again. The increase in weight was noted for determining the water absorption of the blocks.

Fig. 7.2 Cleaning and lubricating the block mould

Fig. 7.3 Dry mixing of all the constituents

Fig. 7.4 Addition of water to the dry mix

Fig. 7.5 Compressing the block by pulling down the lever arm

Fig. 7.6 Ejecting the soil block after compression process

Fig. 7.7(a)

Fig. 7.7(b)

Fig. 7.7(c)

Fig. 7.7. Storage and curing of stabilized blocks [figures 7.7(a), 7.7(b) and 7.7(c)]

Fig. 7.8 Testing for compressive strength in a compression testing machine

Fig. 7.9 Placement of soil blocks in hot air oven for water absorption test.

7.4 RESULTS AND DISCUSSION

Results of various tests related to manufacture and testing of CSEBs are presented and discussed in the subsequent section.

7.4.1 Effect of GBFS on properties of lateritic soil

Various tests were conducted on lateritic soil and lateritic soil + GBFS mixes to decide the optimum percentage of GBFS replacing the lateritic soil. Results of various tests are presented in Table 7.2

Table 7.2. Properties of lateritic soil when replaced by GBFS

Sl. No.	Properties	Percentage of GBFS replacing lateritic soil					
		0%	10%	15%	20%	25%	30%
1	Liquid limit, w_L (%)	44	42	40.2	39.2	38.4	37.1
2	Plastic limit, w_p (%)	32	30	28.6	NP	NP	NP
3	Plasticity Index, I_P (%)	12	12	11.6	-	-	-
4	Specific gravity, G	2.62	2.53	2.49	2.45	2.41	2.37
5	Maximum dry unit weight, γ_d (kN/m^3)	16.9	16.8	16.8	16.7	16.5	16.3
6	Optimum moisture content (%)	21	20	19.5	19	18.4	18
7	UCS at 7 days curing (kPa)	406	480	551	589	549	511
8	UCS at 28 days curing (kPa)	406	772	869	938	899	786

From Table 7.2, it is observed that the soil mixture becomes non plastic beyond 15% replacement of soil by GBFS. Variation of UCS with percentage of GBFS replacing lateritic soil is shown in figure 7.10 for both 7 days and 28 days cured samples. From figure 7.10, it is observed that at 20% replacement of lateritic soil by GBFS, the Unconfined Compressive Strength (UCS) is maximum and hence this percentage of replacement is taken as optimum for manufacture of soil blocks. Laboratory test on lithomargic clay and lithomargic clay + GBFS mixes are already presented in Chapter 4.

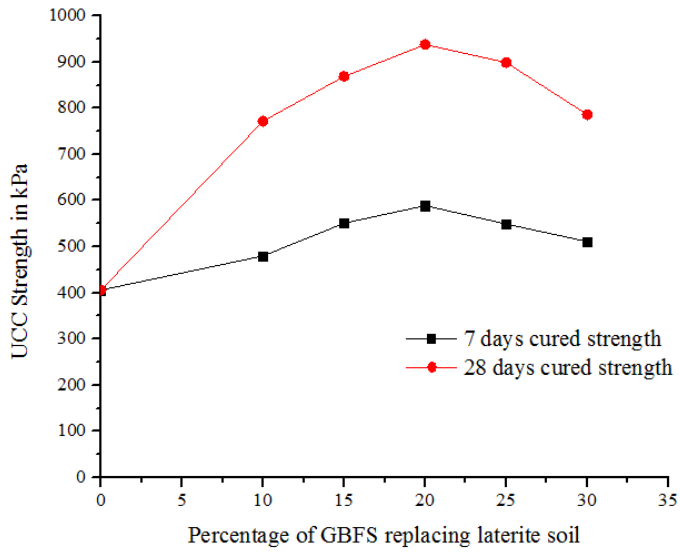

Fig. 7.10. Variation of UCS with different percentage of GBFS replacement.

7.4.2 Test results of compressed stabilized lithomargic clay blocks

Table7.3.Compressive strength and water absorption test results for stabilized lithomargic clay blocks

Series	28 days cured samples		
	Dry compressive strength (MPa)	Wet compressive strength(MPa)	Water absorption (%)
Lithomargic clay	1.05	-	-
75% Lithomargic clay + 25% GBFS	1.61	-	-
75% Lithomargic clay + 25% GBFS with addition of 6% cement	3.48	2.74	13.9
75% Lithomargic clay + 25% GBFS with addition of 8% cement	4.75	3.20	13.4
75% Lithomargic clay + 25% GBFS with addition of 10% cement	5.15	3.63	12.96
75% Lithomargic clay + 25% GBFS with addition of 12% cement	5.55	3.94	12.51

Results of compressive strength and water absorption tests on compressed stabilized lithomargic clay blocks are presented in Table 7.3. Dry compression tests were conducted on lithomargic clay blocks and on 75% lithomargic clay + 25% GBFS blocks. Wet compression tests and water absorption tests could not be conducted, since upon immersion in water there was complete disintegration of the block materials. However, when soil is replaced by 25% GBFS, we see an improvement in dry strength from 1.05MPa to 1.61MPa due to the hydration and pozzolanic reactions between the alumino silicates of soil and the CaO from GBFS forming a strong bond between the soil mixtures. With the addition of cement to the mix, a larger improvement in both the wet and dry compressive strength is observed. Increase in cement content increases the compressive strength of the blocks. This is due to the hydration products of cement, which fills the pores of the soil matrix and enhances the stiffness of its structure forming large bonds connecting the soil particles (Ramirez et al. 2012, Walker and Stace 1997, Reddy et al. 2007).

The wet compressive strength is about 21.3% to 32.6% lower than that of dry compressive strength as seen in Table 7.3 and figure 7.11. This reduction is due to the large amount of finer particles present in the mix i.e. the weakening effect of the bond between the soil particles and the cement paste and may be because of the development of the pore water pressures and the liquefaction of the unstabilized soil particles (Ramirez et al. 2012). For various mixes, water absorption varies from 13.9% to 12.51% (Table 7.3). It is also observed that the water absorption slightly decreases with increase in the cement content. This is due to the reduction in void spaces between the soil particles, which are filled up by the gel formation from the pozzolanic and hydration products of GBFS and cement. Finally, it is concluded that the blocks made of 75% lithomargic clay + 25% GBFS with addition of 10% cement can be utilized in load bearing masonry construction as it fulfills the criteria in terms of wet compressive strength i.e. >3.5MPa and water absorption i.e. < 15% as per the Indian Standard (IS) specifications.

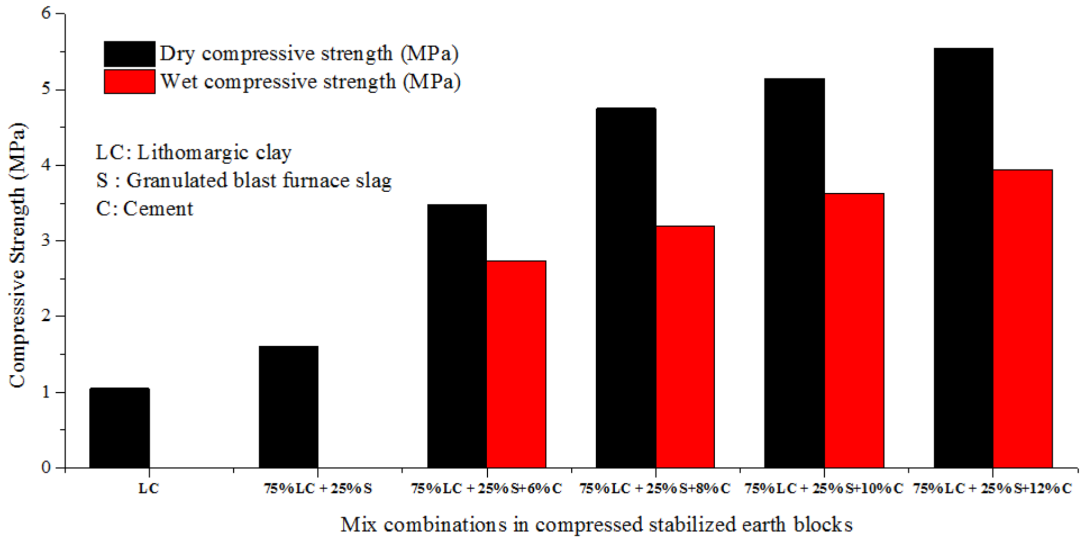

Fig. 7.11. Variation of dry and wet compressive strength of stabilized lithomargic clay blocks

7.4.3 Test results of compressed stabilized lateritic soil blocks

Table 7.4. Compressive strength and water absorption test results for stabilized lateritic blocks

Series	28 days cured samples		
	Dry compressive strength (MPa)	Wet compressive strength(MPa)	Water absorption (%)
Lateritic soil	1.52	-	-
80% Lateritic soil + 20% GBFS	2.13	-	-
80% Lateritic soil + 20% GBFS with addition of 2% cement	3.08	2.52	12.9
80% Lateritic soil + 20% GBFS with addition of 4% cement	3.91	3.09	12.4
80% Lateritic soil + 20% GBFS with addition of 6% cement	4.70	3.61	11.7
80% Lateritic soil + 20% GBFS with addition of 8% cement	5.25	4.19	10.9

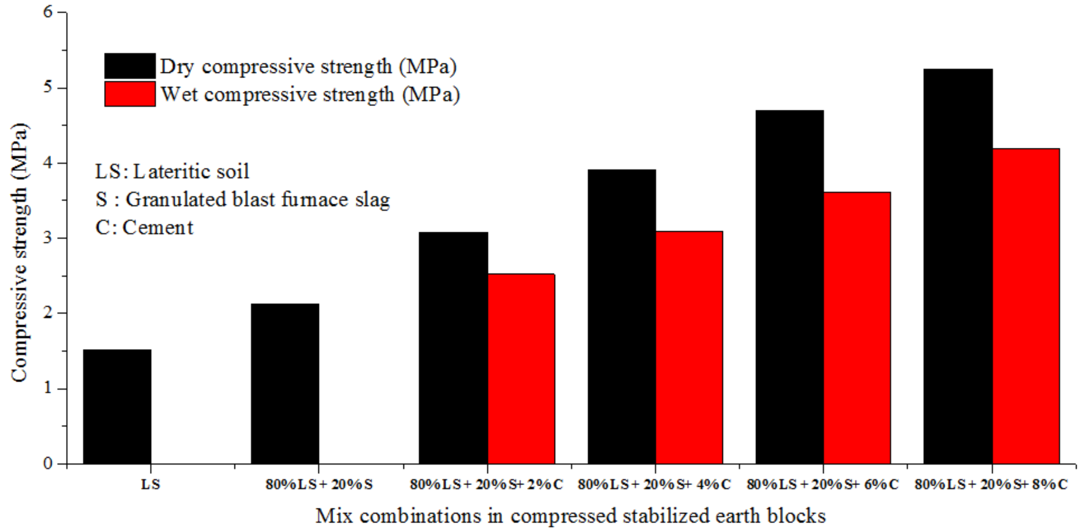

Fig. 7.12 Variation of dry and wet compressive strength of stabilized lateritic blocks

Compressive stabilized earth blocks made up of lateritic soil, GBFS and cement were tested for dry and wet compressive strength and water absorption. Results of compressive strength and water absorption tests on compressed stabilized lateritic soil blocks are presented in Table 7.4 and figure 7.12. Initially, when only the lateritic blocks were tested for their dry compressive strength, it gave a value of 1.52MPa, which is greater than that of lithomargic clay blocks (1.05MPa). The amount of sand fraction is more in lateritic soil and hence a good gradation helps it in the well interlocking of the particles with one another i.e. finer particles will fill the void of the coarser particles, thereby resulting in a denser and rigid soil matrix. The wet compression tests and water absorption tests could not be conducted as the soil block lost its shape after saturation. Similar trend was observed with blocks stabilized with GBFS. With the replacement of optimum percentage of GBFS (20%) to the soil mass, the dry compressive strength increased to 2.13MPa. GBFS contains CaO, it undergoes a slow pozzolanic reaction with aluminum silicates resulting in agglomeration of the soil particles and thus the improvement in strength is observed. This slow pozzolanic reaction may prove to be helpful in the long-term strength of the mix.

Addition of the cement accelerates the pozzolanic reactions within the soil mix. With the increase in cement content, the quantity of C_2S and C_3S increases which leads to the increase in CSH, CAH and CASH gels. These are the binding products of cement hydration, which reduces the porosity and increases the density of the soil matrix. The strength reduction after 2 days soaking was found to be in the range of 18.2%-23.2%. This lower percentage of reduction in strength was due to the lesser amount of finer particles in the soil mix and due to the effectiveness of cement for coarse grained particles. Water absorption values are also included in Table 7.4. Water absorption values for various combinations varies from 12.9% to 10.9%. Furthermore, the decrease in water absorption was due to the reduction in pore voids by the accumulation of the hydration and pozzolanic products between these voids resulting in more denser and lesser connectivity in these voids (Oti et al. 2009, Kaniraj and Havanagi 1999, Tripura and Singh 2015, Reddy and Kumar 2011). Blocks made of lateritic soil and optimum GBFS (20% replacement) with addition of 6% cement fulfills the criteria to be used in load bearing masonry construction. [i.e. wet compressive strength ≥3.5MPa and water absorption <15%].

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 SUMMARY

In this work, an attempt has been made to study the effect of granulated blast furnace slag (GBFS) and cement in stabilizing lithomargic clay and its use in few engineering applications.

Lithomargic clay during its initial characterization showed the need for its stabilization. GBFS is a material, which is an industrial by-product and available in large quantities. The main objective was to improve the strength properties of lithomargic clay using GBFS and cement. Experimental investigations were carried out to check the effectiveness of GBFS in stabilizing lithomargic clay. Lithomargic clay when stabilized with optimum percentage of GBFS, i.e. (75% lithomargic clay + 25% GBFS) gave good improvement in the strength properties. The pozzolanic reaction between granulated blast furnace slag and soil is due to the free CaO content in GBFS. Compared to cement stabilization, the strength gain in GBFS stabilized soil was lesser. To further improve the strength properties of soil-GBFS mix, small percentage (i.e. 2% and 4%) of cement was added. The improvement in strength properties was due to the pozzolanic and hydration reactions, which was observed through SEM and XRD analyses. Major cementitious compounds like CSH, CAH, CASH were responsible for binding the soil particles. Additionally, to apply the experimental results to engineering applications, PLAXIS software was used to study two problems namely, load settlement analyses of strip footings resting on lithomargic clay and stabilized lithomargic clay and the stability analyses of an embankment slope (soon after construction). It was found that the settlement decreased and load carrying capacity increased for a strip footing when resting on stabilized soil. The factor of safety for an embankment slope improved with use of GBFS and cement. Production of Compressed Stabilized Earth Blocks (CSEBs) requires large quantities of cement. Use of GBFS in the manufacture of CSEBs decreases the quantity of required cement. From the experimental investigation, it was found that two locally available soils namely, lateritic soil and lithomargic clay, stabilized by optimum GBFS and a small amount of cement can be effectively used in the manufacture of CSEBs. These CSEBs can be effectively used in construction as load bearing walls.

8.2 CONCLUSIONS

Based on the detailed study carried out in this work, following conclusions are drawn:

- The optimum percentage replacement of lithomargic clay by granulated blast furnace slag is 25%.
- The unconfined compressive strength increases by 65.9% and 206.9% when lithomargic clay is replaced by 25% GBFS for 7 days cured samples and 28 days cured samples respectively.
- For 7 days and 28 days cured samples, the cohesion increased up to 25% replacement of soil by GBFS and further decreased at higher percentages (35% and 45%) of replacement of soil by GBFS. For 7 days cured samples, increase in cohesion when lithomargic clay is replaced by 25% GBFS is found to be 156.5% (i.e. 23kPa to 59kPa), whereas at 28 days curing, it is 260.9% (23kPa to 83kPa).
- The friction angle increases for all percentages of replacement of lithomargic clay by GBFS. At 7 days curing, frictional angle increased from $19^{\circ}$to $27.5^{\circ}$when soil is replaced by 25% GBFS and increase is found to be from $19^{\circ}$to 29° for 28 days cured samples.
- Large improvement in unconfined compressive strength is observed due to cement stabilization. For lithomargic clay stabilized by the addition of 8% cement, percentage improvement in unconfined compressive strength is 235.7%.
- With addition of 4% cement to soil + optimum GBFS mix, the cohesion and frictional angle increases. At 28days curing period, the cohesion and friction angle values improved by 361% and 48.2% respectively with addition of 4% cement to 75% soil + 25% GBFS mix.
- From SEM studies, it is concluded that there is reduction of void/pore spaces due to the accumulation of cementitious matrix. This resulted in the change in

morphology of the material through pozzolanic and hydration reaction products. Hence, improvement in strength is observed.

- From EDS analysis, it is concluded that the strength improvement is achieved with the increase in Ca:Si and decrease in Al:Ca ratios when lithomargic clay is stabilized using GBFS and cement.
- XRD studies revealed that the compounds such as CASH, CSHH, CAOH, gyrolite, gismondine are responsible in binding the soil particles together to form a hardened skeleton matrix.
- From load settlement analysis, it is concluded that the net allowable pressure increases with increase in the depth of stabilized area below the footing. For 2m wide strip footing (B=2m), net allowable pressure increases by 91.1% when 1B depth soil below the footing is replaced by 75% soil + 25% GBFS mix with addition of 4% cement. Improvement is 194.6% when depth of stabilized area is 2B below the footing.
- From stability analysis of an embankment slope, improvement in FoS is observed (for all slopes angles and slope heights) by stabilization using GBFS and cement. For H=10m and i=60°, there is an improvement of 91.6% when embankment of stabilized soil (75% soil + 25% GBFS with addition of 2% cement) resting on lithomargic clay and 261% when embankment of stabilized soil resting on H/2 depth of stabilized soil (75% soil + 25% GBFS with addition of 2% cement).
- Compressed Stabilized Earth Blocks (CSEBs) prepared using 75% lithomargic clay + 25% GBFS with addition of 10% cement are suitable for the construction of load bearing walls.
- Compressed Stabilized Earth Blocks (CSEBs) prepared using 80% lateritic soil + 20% GBFS with addition of 6% cement are suitable for the construction of load bearing walls.

8.3 LIMITATIONS OF PRESENT WORK

In this study, the effect of stabilization was investigated considering curing periods as 28 days. Due to pozzolanic reactions, there is considerable improvement in strength properties up to curing periods of 28 days. However, beyond 28 days further possible increase in strength is not investigated.

8.4 SCOPE FOR FUTURE WORK

- To study the effect of GBFS in stabilization of different soils like marine clay, red earth etc.
- To study the activation of GBFS with other additives such as lime, alkali activators like NaOH etc.

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