GEOTECHNICAL STUDIES ON MARINE CLAY STABILIZED USING QUARRY DUST, GRANULATED BLAST FURNACE SLAG AND CEMENT AND ITS APPLICATIONS

Thesis

Submitted in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

by

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DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY KARNATAKA

SURATHKAL, MANGALORE -575025

SEPTEMBER 2021

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DECLARATION

by the Ph.D. Research Scholar

I hereby *declare* that the Research Thesis entitled "**Geotechnical Studies on Marine Clay Stabilized using Quarry Dust, Granulated Blast Furnace Slag and Cement and its Applications**" which is being submitted to the **National Institute of Technology Karnataka, Surathkal** in partial fulfillment 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.

Place: NITK, SURATHKAL

Date: 29/03/2021.

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CERTIFICATE

This is to certify that the ResearchThesis entitled **"Geotechnical Studies on Marine Clay Stabilized using Quarry Dust, Granulated Blast Furnace Slag and Cement and its Applications"** submitted by **PREETHAM H. K. (Register Number: 177062CV009)** as the record of research work carried out by him, *is accepted as Research Thesis submission* in partial fulfilment of the requirements for the award of degree of **Doctor of Philosophy**.

Nayak 29/9/2021

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ABSTRACT

Coastal region has vast existence of problematic clayey soil referred as marine clay. This problematic soil is characterized by its high consistency limits, high compressibility and low shear strength. In order to overcome these problems, an industrial by-product obtained from the rock crushers and iron industry i.e. Quarry dust (QD) and granulated blast furnace slag (GBFS) has been utilized to enhance the strength and index properties of marine clay. Marine clay was replaced by these granular by-products in various proportions and evaluated for its geotechnical performance. From the experimental data, it was observed that marine clay when replaced with 35% QD and 40% GBFS produced good improvement in UCC strength and shear strength and this proportion is termed as optimum combination. The research includes an experimental investigation on cement addition to optimum QD-marine clay and Optimum GBFS-marine clay combinations 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. In addition, the study also extends in evaluating the effectiveness of QD and GBFS along with cement in the production of compressed stabilized earth blocks (CSEB). From the current study, it is concluded that marine clay stabilized using QD and GBFS with/without cement can be effectively used in geotechnical applications, thereby increasing the rate of effective disposal of GBFS. Also, Study infers GBFS s more effective in stabilizing marine clay compared to quarry dust.

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NOMENCLATURE

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w_l	Liquid limit
w_p	Plastic limit
w_s	Shrinkage limit
I_p	Plasticity Index
c	Cohesion
ϕ	Angle of Internal Friction
$^\circ$	Degrees
R_i	Resistance to loss of strength on immersion

LIST OF ABBREVATIONS

IS	Indian Standard
CAH	Calcium Aluminum Hydrate
CSH	Calcium Silicate Hydrate
CAOH	Calcium Aluminum Oxide Hydrate
CASH	Calcium Aluminum Silicate Hydrate
QD	Quarry Dust
GBFS	Granulated Blast Furnace Slag
GGBFS	Ground Granulated Blast Furnace Slag
MDU	Maximum Dry Unit Weight
OMC	Optimum Moisture Content
OPC	Ordinary Portland Cement
pH	Potential of Hydrogen
UCS	Unconfined Compressive Strength
UU	Unconsolidated Undrained
CBR	California Bearing Ratio
CSEBs	Compressed Stabilized Earth Blocks

CHAPTER 1

INTRODUCTION

1.1 Background

Soil covers the major surface portion of the planet Earth. It is the most important natural resource, which is directly or indirectly needful for mankind in every way. Soil is a product obtained from the disintegration of the rock mass by many factors which include physical, biological, and chemical processes. Hence, the properties or behaviour of the soil depends on the type of its origin, placement, and influence.

Soil is the most important natural resource for civil engineering works which can be used directly for projects like the construction of backfills, embankments, fills, barriers for waste disposal and in the manufacture of tiles and bricks, cement production etc. Also, soil assists as an indirect medium in bearing loads from the superstructure. Hence, studies focussing on soil formation, its composition, and geotechnical properties are significant (Thomas et al. 2018).

Expeditious industrialization and rapid urbanization are taking place mostly in developing countries in the present scenario. Thus, Civil Engineers are faced with many new challenges to propose and execute a project in an optimized way.

Quality soil or best sites or ideal borrow pits in par with typical geotechnical definitions are seldom available. Though the presence of the above said soil might be available, transporting the same to the construction site may not be economical. Hence, there is a serious need to mitigate this problem of low Safe Bearing Capacity (SBC) and excessive settlement by improving the existing land by keeping the economy and environmental aspects under consideration. This process of upgrading the soil properties is known as Ground Improvement (Nayak and Sarvade 2012).

1.2 Soft soil

The tag "Soft soil" is given to those soils with high plasticity characteristics and poor geotechnical properties like low shear strength and high compressibility. It may also be organic. Soft soil comes under the group of problematic soil.

Soft soils are geologically young that attain equilibrium under their self-weight. Since soft soils have not undergone any significant delayed or secondary consolidation, right from the time of their formation, it is strong enough only to resist the overburden stress caused by it's self-weight. Thus it cannot bear additional stress due to external loads, resulting in colossal deformation of the particles (Chung et al. 2002).

Soft soils are normally identified with poor shear strength, low permeability and high compressibility character. As the shear strength of these soils is normally less than 40 kPa, these soils can be moulded by very little pressure (Rajasekaran et al. 1998). As the permeability of soft soil is too low, it affects the rate of consolidation, which is a serious issue. The particle readjustment and attainment of new equilibrium caused due to the applied loads distorts the loading geometry and causes uneven settlements. This results in the instability of the structure, leading to failure and loss of life. These soils are normally found below the groundwater table. The buoyancy effect which creeps in, accelerates the damage and excessive post-construction settlement of the structure (Ouhadi et al. 2014).

1.2.1 Marine clay

Marine clay is a type of soft consistent soil found in the coastal regions of the world. Marine clay is microcrystalline soils composed of clay minerals (kaolinite, montmorillonite, illite, and chlorite) and non-clay minerals (quartz and feldspar) (Rao et al. 1990). Marine clay is an uncommon type of soft consistent clay and is normally found below the water table. Marine clay are less sluggish, highly plastic, blackish silty-clay. It possess low shear strength and high compressibility due to high liquid limit, plastic limit, and plasticity index (Padmaraj and Chandrakaran 2017). This clay is sensitive with low density and high saturation. Due to the specific physicochemical structure, marine clays are vulnerable to volume change (Chew et al. 2004).

The poor geotechnical properties of marine clay create problems, such as cracking and breaking up of pavements, railways, highway embankments, building foundations, irrigation systems, underground pipelines, etc. The existence of geotechnical structures on soft marine clay leads to unpredictable changes over the duration of time and potential sudden collapse of the structure. The total loss or maintenance problems of the structures caused by the problematic nature of marine clays have shown the need for more reliable investigations and necessary methods to reduce or eliminate the ill effects on the structure by proper ground improvement (Phetchuay et al. 2016).

1.3 Soil stabilization

Soils/Sites possessing substandard engineering properties causing difficulties in construction operations were discarded or replaced by our ancestors. In the present scenario, due to the rapid industrialization and urbanization, these lands cannot be neglected. This paves the way for ground improvement. Infrastructure development is at its peak in coastal regions because of the massive movement of goods and raw materials. The coastal region is filled with problematic marine clay and thus, there is a significant need for stabilizing the soil (Abbey et al. 2021; Gravina et al. 2021).

Soil Stabilization is defined as enhancing the engineering properties of the existing soil mechanically or by the addition of suitable additives like special soil, cementing agents, or chemicals. This result in improved shear strength, reduced compressibility, and modified permeability of the treated soil. The traditional method of improving the friction angle is by adding coarser material and cohesion by the addition of cementing agents. Due to this improved cohesion and friction, improved shear strength, stability, and sustainability of the treated soil is achieved (C. Sekhar et al. 2017).

There are many techniques to modify the ground which are mechanical modification, hydraulic modification, physico-chemical modification and modification by inclusion and confinement. The selection of the technique depends on the type of soil, soil history, availability of equipment, zone and depth of treatment, availability of stabilizing materials, type and degree of improvement, the time required for improvement, type of problem (foundation soil, embankment soil, unstable slope, excavation etc.,), the economy of the project and its environmental impacts (Amulya et al. 2018).

Ground improvement techniques result in

- Enhanced shear strength, Bearing Capacity, Stability and Durability of soil
- Reduced compressibility and post-construction settlement
- Modified permeability
- The problem of shrinkage, swell and liquefaction potential is reduced

1.4 Industrial by-products

Innumerable factories, industries, mining plants are found around the world. These plants consume raw materials and produce finished goods. During the manufacturing process, some materials are discarded as useless and are termed as Industrial wastes. Some industrial wastes are decomposable by micro-organisms (food industry, textile mills, dairy, etc.) and some are not (Plastic Industries, fly ash, slag, quarry dust, etc.). The accumulation of these wastes is harmful to living organisms and also pollutes the environment (Alsaleh et al. 2021; Bahadori et al. 2019). Hence, the proper utilization of these wastes may reduce the disposal problem (Patra and Mukharjee 2017).

1.4.1 Quarry dust

As the aggregate requirement in the road and railway construction activities is on the rise, natural aggregate alone cannot meet the demand. Hence, Rock is crushed for the required gradation in quarries. During this process, the by-product of rubble crushers from this quarry known as quarry dust is formed. In the crushing process, about 20%-25% of dust is generated. In India, quarry dust of 200 million tonnes is produced every year. Quarry dust is a solid, inorganic, flaky, elongated, shapeless, rough, cohesion-less, grey coloured fractured dust of the granite stone crusher. This rough surface increases the friction angle and hence the shear strength (Mishra et al. 2019).

Nowadays, quarry dust is utilized in highway construction as a surface finish material. Also, quarry dust is used in the manufacture of lightweight concrete elements and hollow blocks. The use of quarry dust to improve soil properties draws vital attention due to its easy availability in many places.

1.4.2 Granulated Blast Furnace Slag (GBFS)

In the production of iron, iron ore is subjected to a high temperature of about 1600°C in the blast furnace. The by-product in this process generated is termed as slag. Depending on the type of cooling, slag can be classified into different categories. Molten slag cooled and solidified by water quenching through high-pressure jets generates GBFS (Amulya et al. 2016). Quenching prevents crystallization leading to the formation of granular aggregates. Granulated blast furnace slag is powered by crushing and then screened to produce Ground Granulated Blast Furnace Slag (GGBFS). GGBFS is mixed with cement for construction activities to increase the strength of the structure over time (Sharma and Sivapullaiah 2012). As slag contains lime (30% to 45%), it acts as a binding material when exposed to water. The pozzolanic reaction initiates when slag encounters water, leading to the formation of hydrates just like the hydration of cement. The National Slag Association has shown that the use of slag in building construction (Indian Minerals Yearbook 2015) does not affect the human environment (C. Sekhar and Nayak 2017).

With the increase in infrastructural development, the demand for iron and steel has also increased. Thus, slag is generated in a vast amount. Slag is stacked and piled which consumes a larger area. Hence the disposal is of great concern. The grinding of granulated blast furnace slag to obtain GGBFS is expensive. To reduce the burden of disposal and to mitigate the problem of marine clay, GBFS is used as a stabilizer in the proposed work. This is cost-effective, reduces the problem of disposal, and stabilizes the problematic marine clay (Mymrin et al. 2005).

1.5 Summary

Marine clay is fine grained high plastic clay. Marine clay poses low shear strength. It is highly compressible soil and spread along the coastal region. The geotechnical properties of marine clay advocates that the soil is weak and stabilization is required to support structures founded on them or for any other geotechnical applications. In line with improvement of gradation property of marine clay, granular additive incorporation can be done. Industrial by-product, such as quarry dust, GBFS can be used. Hydraulic binders can be added to improve the binding property of the soil. Marine clay with improved shear strength properties can be used in the geotechnical applications.

1.6 Scope and objectives of the investigation

Marine clay is a problematic soil, construction and maintenance over it is uneconomical with a high-risk factor. The high plasticity index, low safe bearing capacity, excessive compressibility and low permeability conclude the soil as unsuitable as a construction material for engineering projects. For the beneficial use of marine clay, soil stabilization is required. Quarry dust and GBFS are the granular by-products of the granite stone crusher and the iron industry respectively. Quarry dust with cement can also be used as a stabilizer to marine clay as quarry dust increases the friction angle and cement binds the constituent particles. Using GBFS and cement to marine clay may impart improved geotechnical property to marine clay due to improved gradation and binding property of composite. Incorporating these granular materials with and without cement may improve the engineering properties of the stabilized mix, which is still a novel process and has never attempted in soil stabilization in this region. There is a need for research in this regard.

The properties of Marine clay changes upon drying. In connection with the field applications namely weak soil replacement with stabilized soil in case of shallow foundation problem and manufacture of compressed stabilized earth blocks, air drying is adopted. The present problem statement is soil stabilization at shallow depth rather than in-situ ground improvement at a greater depth.

The main objectives of the study are:

1. To characterize Marine clay, Quarry dust and GBFS according to their geotechnical properties.
2. To examine the effect of the addition of quarry dust with and without cement on the Atterberg limits, compaction characteristics and shear strength properties of the stabilized mix.

3. To examine the effect of the addition of GBFS with and without cement on the Atterberg limits, compaction characteristics and shear strength properties of the stabilized mix.
4. To investigate the reaction products of the stabilized mix by SEM and XRD.
5. Application of the laboratory results for the typical foundation problem using PLAXIS-2D software.
6. To examine the suitability of stabilized mix in the manufacture of stabilized earth blocks.

The research program is split into four main phases:

1. The first Phase is to procure and characterize the test materials; Marine Clay and stabilizers (GBFS, Quarry dust, and additive). The characterization tests include grain size analysis, specific gravity, Atterberg limits, compaction, unconfined compressive strength, triaxial shear strength, oedometer 1D-consolidation, California Bearing Ratio (CBR), identification of oxide composition in soil and stabilizing agents from chemical tests.

2. The second phase includes the investigation on the replacement of marine clay (10%-50%) by granular stabilizers (GBFS/Quarry dust by an interval of 10% the dry mass of untreated soil) without and with cement (2%, 4%, 6%, 8% and 10% of the dry mass of soil) addition for the improved plasticity behaviour, shear strength, compressibility, and other geotechnical properties.

3. The third phase is to examine the reaction products of the stabilized mix for different curing time. The reaction products are identified using X-Ray Diffraction (XRD) peak analysis. For structural re-arrangement of stabilized mix Scanning Electron Microscope (SEM) test is carried out.

4. The fourth phase is the application of the lab results to execute the foundation problem in PLAXIS-2D software. The utilization of the stabilized mix in the production and testing of manufactured earth blocks.

1.7 Organization of the Thesis

The thesis work titled 'Geotechnical studies on Marine clay stabilized using Quarry Dust, Granulated blast furnace slag and cement and its Applications' is presented in nine chapters as follows:

Chapter 1: This chapter includes an introduction to marine clay and its problems. The need for stabilization of marine clay and the research objectives adopted for the thesis are also presented.

Chapter 2: A comprehensive literature survey for the present study is covered in this chapter.

Chapter 3: The third chapter deals with the discussion on the various materials, experimental tests and the methodology adopted.

Chapter 4: This chapter discusses the results obtained from the experiments performed on marine clay, quarry dust and marine clay stabilized with quarry dust and cement.

Chapter 5: This chapter discusses the results obtained from the experiments performed on marine clay stabilized using GBFS and cement.

Chapter 6: In chapter six, load-settlement behaviour of strip footings for untreated soil and stabilized soil were analyzed using PLAXIS 2Dsofware.

Chapter 7: This chapter discusses the detailed study on the incorporation of quarry dust, GBFS, and cement in the production of compressed stabilized earth blocks.

Chapter 8: This chapter deals with the effect of temperature on the properties of marine clay.

Chapter 9: This chapter presents the conclusion drawn from the research work along with the scope for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 General

The rapid advancement in every field of science and technology paved the way for industrial and infrastructural growth at an alarming rate. This advocated the utilization of every corner of useless sites that had been previously discarded by our ancestors due to their poor engineering performance (Basha et al. 2005). This created a knowledge transfer among various fields of technology and the emergence of innovative ideas regarding the usage of various additives to the soil, which would result in a strong and sound site. Conventional additives, such as lime and cement were added to improve the resistance of soils to external loads. But their negative impact on carbon emission and the greenhouse effect has pushed researchers to find the minimal usage of these additives or to find other alternatives. In selecting the alternatives, their sustainability and environmental impact should be assessed (Amulya et al. 2018). The best possible alternatives would be industrial by-products because their usage would in turn minimize its disposal issues, providing us with a double advantage. From the geotechnical point of consideration, the alternate material should fulfil the strength, serviceability, durability, and sustainability of the composite (Rashad et al. 2016). In this regard, a detailed literature survey is performed on the study soil, it's traditional way of stabilization and also on the research carried out on the utilization of industrial by-products in the stabilization process.

2.2 Marine clay

Marine clay is associated with low permeability, poor shear strength, high settlement, and instability. Moreover, the severe problem that makes the soil unsuitable to be used as foundation soil for construction is due to its high plasticity behaviour (Rao et al. 1990). Also, marine clay varies in its properties with the way of drying (Pandian et al. 1991; Rao et al. 1989). There was a serious issue in the Australian railway network of possible de-alignment of rail lines along with the excessive settlement due to the same resting on marine clay (Mirzababaei et al. 2018). Various kinds of un-computable stress are induced by excessive settlement, which may directly reflect as cracks in buildings and highways, de-alignment in railway lines, and damage to other engineering structures founded on marine clay (Yong 1984). Physically, marine clay is sluggish, very sticky when they are wet and the properties differ once it dries, which makes it difficult to compute its behaviour. The defects and the damages of infrastructure caused due to marine clay as foundation soil needs a high level of risk and budgets to maintain, repair and rehabilitate (Yi et al. 2015). Therefore, seeking solutions to marine clay induced problems have been studied by many researchers.

Table 2.1 Properties of marine clay in various parts of the world

Sl. No	Location	Particle size (%)				Consistency limits (%)		Plasticity index (%)	Reference
		Gravel	Sand	Silt	Clay	Liquid limit	Plastic limit		
1	India	0	13	23	64	85	46	39	Izabel and Sangeetha (2014)
2	China	0	3	44	54	62	59	27	Tongwei et al. (2014)
3	Singapore	0	0	46	54	85	46	39	Moses et al. (2003)
4	Nigeria	0	18	40	42	118	46	72	Otoko and Blessing (2014)

Marine clay is spread around the globe along the off-shore and coastal belt. Irrespective of the region/country, majority of the marine clay exhibits a high value of the liquid limit. This high value directly indicates the settleability of the soil (Jefferson and Rogers 1998). Also, the high liquid limit values leads to high value of its plasticity index, confirming the soil as problematic. Table 1 shows the comparative analysis of the properties of marine clay in various regions. From the table, the values indicate that irrespective of the region, the consistency limits and plasticity index of marine clay are always high. The fines (clay and silt) content are high which concludes the higher specific surface area of the soil. This also proves the large value of its consistency limits. Higher the specific surface area larger is the clay-water interaction and hence more amount of moisture is required to coat the particles. This leads to a higher liquid limit of marine clay (Mathew and Rao 1997). Sometimes the value of natural moisture content crosses the liquid limit. This makes the marine clay to become more sluggish and slurry.

The engineering characteristics of the marine clay of Singapore were studied by Arulrajah and Bo. Marine clay of the upper layer and lower layer exhibited a range of liquid limits of 80%- 95% and 65% -90% respectively. Plastic limit varied between 20%-28% and 20%- 30% for the same top and bottom layers. The clay size particle of Malaysian marine clay was found to be 40% (Mohammed Al-Bared and Marto 2017). Silt size particles were the next major particles, followed by sand-size particles. This trend was found to be similar for the majority of the marine clays in various regions of the world. The observation showed a least or no presence of gravel in the marine clays. Normally the specific gravity of marine clay varied with a range of 2.45 to 2.65 (Mathai et al. 2007). The maximum dry unit weight from the standard proctor test varied in the range of 13.2 kN/m^3 to 16.8 kN/m^3 and the corresponding optimum moisture content varied from 40% to 20% (Mohammed Al-Bared and Marto 2017). The value depends on the percentage of fines in the soil. The engineering properties of marine clay depend on its micro-structural arrangements and mineralogical compositions of the minerals within the soil (Tanaka et al. 2001). Marine clay has a high quantity of clay content and due to this, it exhibits the appearance of clay minerals that are responsible for the poor characteristics of marine clay (Mathai et al. 2007). The main minerals of marine clay are illite and a mixed layer of illite-smectite. Non-clay minerals, such as quartz and feldspar were confirmed by the XRD study, which varies from region to region. From the results obtained from Scanning Electron Microscopy (SEM), the arrangement of the minerals within the marine clay shows the fabric structure indicating that minerals exhibited a high void ratio that create an open network matrix (Venkateswarlu et al. 2014).

Mangalore marine clay and Cochin marine clay of western coastal regions of India are of medium sensitivity. Mangalore clay contains an appreciable quantity of kaolinite (52%), followed by smectite (32%) and illite (16%). Mangalore marine clay has less or no expansive clay minerals (Rao et al. 1990). Marto et al. (2015) conducted the Unconfined Compression Strength (UCS) test on dredged marine clay obtained from

Johor and a strength of 23kPa was observed. Chong and Kassim (2015) had found the UCS value of 261 kPa for the marine clay collected from the Pontian region. Similarly, un-drained shear strength of 30 kPa to 50 kPa was observed for tiller marine clay, which reveals the soil is of medium-stiff consistency. Also, the value was found to increase with depth (Chong and Kassim 2015). In the eastern part of Melbourne, the existence of soft marine clay showed a range of shear strength varying from 5 kPa to 30 kPa. Also, further studies revealed that regardless of the magnitude of the applied loads, a huge settlement occurred. The total and differential settlements of 500 mm to 700 mm were expected for shallow foundation-supported structures built on it (Phetchuay et al. 2016).

The increase in temperature due to climatic, microbial, or man-made activities makes corresponding changes in the stress field, strain field and seepage field in soils. The distortion of the respective equilibrium will cause damage to the engineering structures built on or surrounded by soil (Chen et al. 2016; Mitchell 2009). Many researchers have studied the influence of temperature on the physical, hydraulic, mechanical, and index properties of clays. Results showed the alteration in geotechnical properties of clay due to the thermal variation is significant (De Bruyn and Thimus 1996; Sunil and Deepa 2016; Villar and Lloret 2004). The effect of high temperature on the index properties and compaction characteristics on two clays were studied. Maximum dry density has increased and the liquid limit has decreased till a particular temperature and beyond this temperature, marginal changes were observed. Soil acts as non-plastic from the same specific temperature (Tan et al. 2004). Clays were subjected to different temperatures (25 to 150 °C) and examined for their various geotechnical properties. The test results showed the decrease in the free swell index, increase in maximum dry density and increase in California Bearing Ratio (CBR) till 100 °C treated soil and the trend reverses for further temperature increment. This was due to the development of micro-cracks at 150 °C (Gadzama et al. 2017). There was a considerable change in the shear strength and compressibility characteristics upon drying the clay at different temperatures. Also, the plasticity index, specific gravity, and percentage fines decrease with drying the soil at a higher temperature (110 °C). This is due to the agglomeration of clay particles, which leads to the formation of bulk particles (Pandian et al. 1991). The consequences of seasonal variation, elevation in the groundwater table, or any other source of moisture movement, the rewetting of the air-dried and oven-dried samples by distilled water for a period of 4 months was studied (Pandian 1993). The observation concluded that no significant changes in liquid limit values due to rewetting of dried soil, indicating that the plasticity characteristics undergo irreversible changes upon drying (Pandian et al. 1993). The knowledge of the drying temperature affects the geotechnical property of clays which attempts in evaluating the laboratory pre-treatment and correlating with the field simulation (Sahib and Robinson 2020).

At present, the increasing traffic, building infrastructure, large construction of facilities and emerge of metropolitan areas especially along the coastal belt are of solicitude. This is due to the encounter of soft marine clay in the coastal region. Marine clay is showcased by its poor geotechnical characters such as high liquid limit, high plasticity index with poor shear strength and high compressibility. Hence, engineering structures built on this soft clay threats the human life and assets.

2.2.1 Marine clay stabilization

Traditional stabilizers including lime, cement, fly ash, and other binding materials were attempted to stabilize marine clay. Cement is often used as an additive to improve the strength and stiffness of soft clayey soils (Uddin et al. 1997). Cement is the most commonly used stabilizing agent in various ground engineering applications. Lime stabilization involves the alteration of soil structure flocculation, carbonation, and pozzolanic reaction. In cement stabilization, the strength gain is due to the pozzolanic reaction which is irrespective of soil type (Baldovino et al. 2018; Lin et al. 2007; Yong and Ouhadi 2007). The Portland cement (PC) stabilization for clay is through the cement hydration reaction and formation of Calcium Silicate Hydrate (CSH), Calcium Aluminum Hydrate (CAH), and Calcium Alumino Silicate Hydrate (CASH) (Nayak and Sarvade 2012).

The cement addition to dredged soil for its bulk utilization in the engineering projects as a construction material was attempted. This resulted in the improvement in index properties upon the addition of cement in varying percentages (4%, 8%, 12%, and 16%) of the dry mass of soil. Compaction characteristics and shear strength parameters of stabilized marine clay samples were studied. The test results revealed that with increasing cement content and curing age, the unconfined compressive strength increases significantly. 12% cement content was concluded as an optimum dosage required to achieve the prescribed strength.

The binding compounds formed due to hydration reactions fill the void spaces and enhances the strength (Jan and Mir 2018). The strength behaviour of cement stabilized dredged marine clay was studied. 20% cement addition to dredged soil resulted in an increase in strength making it suitable for engineering application (Chiu et al. 2009). However, cement-treated soils are more brittle than untreated soil and fail at a lower strain. Similarly, a case of 6% cement addition to dredged marine sediments has met the requirements as a construction material for foundation and base layer in highway construction (Jan and Mir 2018). The cement consumption to stabilize marine clay depends on the gradation character of the soil. The higher the fines more is the cement consumed for the treatment (Venkatarama Reddy and Latha 2014).

Higher cement content addition of 10% to 25% was attempted for the reduction in the compressibility of the clay. About 12 to 19 times increment in the coefficient of consolidation was observed with the increase in cement content of 20%. For 15% of cement-treated clays, the reduction of the compression index reached an optimum lower value indicating, the lower settlement of soil upon external pressure (Lorenzo and Bergado 2004). From the results of the consolidation test conducted on soft soil with cement/lime addition with an increment of 2% additive by the dry weight of soil, an increase in pre-consolidation stress of around 50.8 kN/m^2 and 52.5 kN/m^2 was observed with each increment in lime and cement addition respectively. However, beyond optimum additive addition, the overall rate of pre-consolidation pressure decrease was observed for higher cement stabilized mix (Ouhadi et al. 2014). Lime (3% and 6%) was added to marine sediments, with the addition, improvement in the consolidation properties was observed through the increase in pre-consolidation pressure and coefficient of consolidation. Also, the compression index and swelling index decreased proving the stabilized mix to be resistant against settlement (Wang et al. 2013).

The maximum dry density decreases from 1.4 8g/cc to 1.34 g/cc with a 10% lime addition to Pontian marine clay. The UCS value has increased by 49% compared to basic soil. The shear strength parameters of lime stabilized soil have increased the internal friction from 20.2° to 30.5° and cohesion intercept from 31 kPa to 109 kPa. The compression index and permeability had decreased by 48% and 67% respectively when the stabilized mix was cured for 56 days (Chong and Kassim 2015). Injection of lime to marine clay in the form of a lime column was studied. The observation from the studies indicated that the formation of several bonding compounds of soil-lime treatment had reduced the compressibility of marine clay by about 33% to 50% of the value of the basic soil. The compressibility index decreased from 0.85 to 0.36, showing better performance with lime addition (Mohammed Al-Bared and Marto 2017). Microstructural analysis through XRD peaks on hydraulic binder stabilized marine clay indicates that with the increasing percentage of additive, a reduction in the intensity of XRD peaks of clay minerals happens. This is due to the solubility of clay fraction at elevated *pH* (Ouhadi et al. 2014).

The influence of granular additives on fine-grained marine clay was studied. In this context, the Recycled Blended Ceramic Tiles (RBT), a waste material generated from the ceramic tile factories were added to marine clay. With the addition of the RBT, the plasticity property and compaction properties have improved with the decrease in the compressibility characteristics of soil (Al-Bared et al. 2018b). Maximum dry density (MDD) and optimum moisture content (OMC) of soft untreated soil were 1.59 Mg/m^3 and 22%, respectively. With the addition of 40% RBT with 0.3mm size, the OMC decreased to 15% while MDD increased to 1.77 Mg/m^3. Besides, samples treated with 40% RBT of 1.18 mm size resulted in a major decrease of OMC to 13.5% while MDD increased effectively to 1.82 Mg/m^3. From the study, it was concluded that the larger the size of RBT, the more effective is the compaction characteristics of the treated marine clay (Al-Bared et al. 2018).

A series of UCS tests were conducted to determine the efficacy of Poly Vinyl Alcohol (PVA) and 1,2,3,4% Butane Tetra Carboxylic Acid (BTCA) along with the fiber reinforcement with short polypropylene fibers for enhancing the compressive strength of clay. A comprehension test series were performed to determine the optimum binder content for improving the mechanical behaviour of relatively soft and stiff clays. The test also included different initial void ratios (0.64, 1.64) and moisture content (16.8%, 48%). The results depicted that the combined effects of PVA, BTCA and fiber reinforcement have improved the strength and ductility of the clay significantly. However, the optimum content of binders and fiber content was sensitive to the initial void ratio and moisture content of the soil (Mirzababaei et al. 2018).

2.3 Soil stabilization using quarry dust

Aggregate crushers and quarries are the basic requisites that feed the construction industries and highway development projects. The primary by-product obtained from rubble crushers is quarry dust. About 20% -25% of quarry dust is generated in the stone crushing operations. Nearly 200 million tons of quarry dust is produced every year in India alone. Earlier quarry dust was considered as waste material but, in the present days, it is used in road construction activities. These are fine-grained aggregates that are coarser than the clay and silt. Quarry dust is physically attributed to its inert nature and texture of the particles. The particles are angular and flaky, which imparts a higher frictional angle. A high frictional angle of about 44° was observed for quarry dust, which in turn gives good shear strength (Soosan et al. 2005). Due to its better mechanical properties, quarry dust is used in many geotechnical applications, such as sub-base material in highways, embankment construction, backfill material, etc. Quarry dust is used as a substitute for natural aggregate (sand).

Quarry dust is added in different proportions to three tropical soils in the context of utilizing the composite as a highway construction material (Kumar et al. 2016). Results from this study showed the problem associated with clayey soil in highway construction and that incorporation would solve the problem considerably and the approach proved to be economical. The soaked and un-soaked CBR of soil showed a considerable difference in the value. But with the addition of quarry dust to the soil, the value was too close to each other. Geotechnical properties of problematic clay had improved substantially with the quarry dust addition. The liquid limit and plasticity index had reduced and the compaction characteristics have improved. The increase in maximum dry density and decrease in optimum moisture content with a steeper compaction curve is attributed to the improvement in the grain size distribution curve showing a better-graded composite. With the increasing percentage of quarry dust addition, the CBR values steadily increased for all three tropical soils. The improved CBR is due to its improved shearing resistance; as the frictional angle of soil-quarry dust mix increases. The research concluded that for much better results a 150% of quarry dust needs to be added to the soil (Jose et al. 1988).

The sound and durable stabilized sub-grades are the ones that can retain the stability and integrity over the years even after exposure to detrimental environmental effects and loading. This is accomplished with the alteration of soil properties due to the long term physio-chemical changes. Hence durability studies are also important along with the initial characterization of the stabilized mix. Usage of quarry dust and cement kiln dust into high plastic soil in view of achieving a durable sub-grade material was attempted. The integrity and loss of shear strength upon immersion in water of specimens cast with black cotton soil was assessed. The samples were prepared at a constant amount of 10% quarry fines with various proportions of CKD (0%, 4%, 8%, 12%, and 16%) compacted to their respective optimum moisture contents and maximum dry density. The initial characterization showed an increase in the swell index and an increase in CBR. The 0-4% CKD samples along with quarry dust failed due to their low CBR value and inadequate strength upon immersion in water. While the 8-16%CKD addition along with 10% quarry dust meets all the requirements as a sub-grade material in pavement structures. The overall durability of CKD along with quarry dust addition to soil has proven the best soil stabilization through the improvement of both the physical-chemical property of clays (Amadi 2014).

The economics study was carried in the utilization of granular materials into the soil in obtaining the minimum CBR criteria of 8% under soaked conditions. The study concludes that 30% of quarry dust or 20% of 10 mm size coarse aggregates with study soil individually can meet the criteria. Also, the same criteria could be achieved by the combination of soil with 10% quarry dust + 10% coarse aggregates (Mishra et al. 2019).

Quarry dust addition to the poor soil has improved the shrinkage limit, increased the maximum dry density, friction angle and decreased the liquid limit, plasticity index, optimum moisture content and cohesion of the soil (Sabat 2012). Consistency limits test, proctor test and shear strength test were performed on gravelly soil with the addition of stone dust in varying percentages (20%–30%) and from the results of these test it was noticed that with the addition, 3%–7% increase in MDD value and 16%–52% increase in CBR value of the stabilized mix was recorded. With the addition of 25–35% of the stone dust to gravelly soil, the mix was now able to meet the criteria as sub-base material according to the Ministry of Road Transport and Highways (MoRTH) (Satyanarayana et al. 2013). The impact of the addition of 20% quarry dust on the geotechnical properties of an expansive soil was assessed and it was found that the MDD and CBR value of the expansive soils increased by 5% and 35% respectively. Similar improvement in consistency limits and optimum moisture content was found with the decreased value of the properties of the treated soil for sub-grade application (Dixit and Patil 2017). The influence of the addition of stone dust on compaction characteristics of clayey soil was studied. Stone dust was used in various proportions (10%, 20%, 30%, 40% and 50%) by weight of dry clayey soil. Results predict that the 30% stone dust addition to clayey soil concluded as an optimum mix and can be used as a construction material (Chetia et al. 2018). A detailed experimental study was carried out with coarse aggregates of different proportions from 5% to 30% mixed with weak soil for the construction of the pavement layer in view of enhancing the CBR value. The CBR valve increased from 20.99% to 115.83% of the soil. In regions where coarse aggregates are available, this type of soil treatment works to be economic (Kesharwani 2016).

Quarry dust is a sandy inert material left as by-product during crushing of rock. Quarry dust exhibits high shear strength due to its high frictional angle, which is beneficial for many geotechnical applications. It has good permeability and can be used in stabilizing many soils.

2.4 Soil stabilization using granulated blast furnace slag

Granulated blast furnace slag (GBFS) is generated from the ferrous industry as primary waste material. The controlled mass of limestone, iron ore and coke is fed into the blast furnace operating at 1500 °C and thus iron is produced leaving behind the by-product slag. Slag of low specific gravity compared to that of iron, floats on the top of molten iron. Later, slag is skimmed off from the furnace. Molten slag is composed of silicates and aluminates from iron ore and limestone in various oxide forms, respectively (Dhoble and Ahmed 2018). This molten slag, when cooled with high-pressure water jets turns the slag into glassy granular aggregates (Nidzam and Kinuthia 2010). For the cement production, the same is crushed, ground to a fine powder and sieved which is termed as Ground Granulated Blast Furnace Slag (GGBFS) (Yi et al. 2010). In this iron production, for every ton of crude iron, about 300–540 kg of slag will be produced. Ten million tonnes and more amount of slag are accumulated in India every year. The National Slag Association (NSA) recommends the incorporation of slag as construction materials in various civil activities, as they induce no harm to the environment or threat to human life. Also, the utilization of slag in engineering projects solves the problem of its disposal (Metals 2015). CaO present in the slag acts as a binding agent to bind the constituent particles. The particle size or the specific surface area of the slag directly affects the reaction products which are responsible for the strength gain. Finer the slag, more is its reactivity (Kavak and Tüylüce 2012). However, the fine grinding process makes the same costlier. The reaction products are similar to the cement hydration products. But the strength gain is a function of time. As the slag reactivity is slow and hence, activators like lime or cement is required (C. Sekhar et al. 2017).

The stabilization efficiency of alkali-activated GGBFS compared with the stabilization using Portland cement (PC) for the marine clay was carried out. With any age of curing, the optimum combination of alkali-activated GGBFS gave at least twice the strength compared with the Portland cement-stabilized marine clay (Yi et al.

2010). Lime-GGBS-stabilized marine clay had exhibited an enhanced unconfined compressive strength (UCS) by 170% than that of PC-stabilized marine clay which is due to the formation of ettringite, CSH, CASH, and CAH. The products are formed by the hydration of lime, GGBFS, and silica (Nidzam and Kinuthia 2010; Yi et al. 2015).

The acquisition of GGBFS to the soil attempts in the densification of soil by increasing the specific gravity of the mix. The addition has decreased the plasticity index, improved the cohesion and friction angle of soil. This has resulted in soil suitable for pavement or embankment construction. Research carried with the incorporation of GGBFS alone, indicated that composite of clay with GGBFS gave increased strength due to the hydration reaction of GGBFS, while the void space filled resulted in the crystalline growth (Poh et al. 2006).

Work carried with the addition of fine and coarse blast furnace slag resulted in a solid matrix. This was due to the formation of Calcium Silicate Hydrate (CSH) upon curing. As slag content increases in the mixture, the maximum unit weight of the soil increases due to the formation of hydration products (Guda 2016). The hydration reaction took place between the soluble silica and calcium hydroxide [$Ca(OH)_2$] in the presence of water. Silica components of clay and calcium obtained from slag or fly ash undergo pozzolanic reaction and responsible for the production of Calcium Silicate Hydrate (CSH) and this CSH gel occupies the pore spaces (Pathak et al. 2014). From the experimental data, it was found that the UCS and CBR value increased by 30–40% and 100% for slag stabilized soil respectively. The detrimental effect of seasonal and climatic changes in tropical regions on the soil as a highway construction material was studied. In that context, the soil treated with alkali solution with either fly ash or GGBS was tested for its performance to sustain 12 cycles of wetting and drying and the results showed a weight loss that was within the limit of 14%. Hence the alkali-activated soil stabilized using GGBFS can be laid as a base course layer for both high and low volume roads (Amulya et al. 2018).

The effect of basic oxygen furnace slag and granulated blast furnace slag for controlling dispersivity of soil at an early age of curing was analyzed. The dispersion of soil by water causing erosion is due to the ion exchange phenomenon. The addition of both slags has decreased the erosion potential of soil. The reason for the better resistance is due to the increased electrical conductivity (EC) caused by additives. This higher EC value resulted in a more flocculated structure and producing bulk particles that have a lower affinity to erosion. The fact is that the flocculated particles showed lower mechanical capacity as the short-time reactions are incapable of producing cementitious compounds. However, the enhancement in strength with due course of time upon curing the composite samples was observed with the hydration reactions (Goodarzi and Salimi 2015).

MgO was used as an activator for GGBS; this has accelerated the rate of strength development of the stabilized clay; however, the negative effect was observed through excessive MgO with high reactivity causing strength loss (Yi et al. 2016). Hence, MgO of low content with high reactivity is suggested for the stabilization of soft clay (Yi et al. 2016). The usage of the blast furnace slag has reduced the swell potential, swelling pressure, and free swell index of the soil. Moreover, the addition of a one percent activator (lime) has made significant improvements on the same properties of the soil. Also, the compressibility characteristics and its values decreased, indicating that slag along with activator has an immense effect in the stabilization of soil rather than slag alone in the stabilization treatment (Sharma and Sivapullaiah 2017). The soaked CBR of cement GBFS-stabilized clay and cement GGBFS stabilized clay was found to increase in both systems. As the cement percentage increases, the soaked CBR continuously increased; thus, the thickness of the cushion mix reduces. From the research, 2% of cement-activated GGBFS can be sufficient as cushioning material to reduce the heave of the expansive soil. Thus slag with an activator in minimal dosage improves the performance of highway pavements (Guda 2016).

The introduction of granular material to poor soil had altered the compaction characteristics and also increased the shear strength of poor soil. The UCS has enhanced because of improvement in the gradation characteristics of the mixture (Nayak and Sarvade 2012; Soosan et al. 2005). For concrete, GBFS is used as an alternate source for the conservation of natural fine aggregate. This incorporation has shown improvements in the tensile and compressive capacity of the concrete mix (Patra and Mukharjee 2017). The increased addition of GBFS to lithomargic clay showed an improvement in the gradation character. Besides, there was a drop in liquid limit and plasticity index with higher GBFS addition; the same had also decreased the compressibility of the stabilized mix (C. Sekhar et al. 2017). The improvement in the strength was noticed with the addition of GBFS to lateritic soil. The combination of 20% GBFS and 80% lateritic soil yielded maximum UCS strength upon curing when compared to the rest of the other combinations and hence this proportion was termed as optimum GBFS-laterite combinations.

A similar kind of stabilization was carried on lithomargic clay and the laboratory results manifested the optimum combination as 25% GBFS and 75% lithomargic clay as an optimum mix. The optimum slag requirement to the soil as a stabilizer relies on the particle size and clay content of the untreated soil (C. Sekhar and Nayak 2018). Finer the soil more is the slag required to yield maximum UCS strength. The UCS strength increased from 406 kPa to 938 kPa and from 232 kPa to 712 kPa for the optimum slag-lateritic and optimum slag-soil respectively. The addition of granular slag to the soil can improve the friction angle and marginal improvement in the cohesion of the composite. For further more enhancement cohesion, cement/lime addition was recommended. Scanning electron microscope (SEM) and X-Ray diffraction (XRD) analysis were carried out to examine the reaction products of GBFS to the soil. Improvement in strength was due to the formation of CSH, CASH, CSHH, and CAOH gel. These cementitious compounds reduce the void space in the mix and also coat and bind the particle resulting in a rigid structure (C. Sekhar and Nayak 2017). Replacing the weak soil with the stabilized soil–granular slag mix with and without the addition of cement can be used for supporting the shallow foundations. Stabilized soil–granular slag mix with and without the addition of cement can also be used in the construction of embankments (C. Sekhar and Nayak 2017).

The addition of either cement or lime induces the alkalinity in the system, i.e., the pH elevates. This elevation in pH ($pH \geq 12$) facilitates the slow dissolution of binding agents in the medium and interaction with clay minerals leads to the formation of aluminate and silicate anions. Ca^{2+} link the silicate and aluminate of clay and forms calcium-alumina-silicate hydrates gel, this gel coats the discrete soil particles together. This reaction is now as pozzolanic reaction that produces various amorphous phases (gels) which are very slow at ambient or room temperature. Adding more lime/cement fills the pores of the sample and reduces its void space and permeability. The permeability of the treated sample decreases continuously as the hydraulic binder-clay reaction propagates. Changes in the fabric-stricture, texture, and pore accessibility enhance the tortuosity leading to the reduction in permeability (Al-mukhtar et al. 2012).

Knowledge of the physical, chemical, morphological and mineralogical properties of slag is important to assess their mechanical and cementitious properties. This plays a vital role in their optimum requirement to soil. Although slag requires an activator as the hydration reaction rate is very slow. Unless an activator is used, a further hydration reaction is inhibited. Ordinary Portland Cement (OPC), lime, gypsum, and many alkali solutions can be used as activators. Higher alkali concentrations and dosage leads to faster hydration. In the aqueous media despite using the chemical activator, it is difficult to solubilise silica when pH is less than 11.5. Hence pH plays a key role in the hydration process and also in CSH formation (Song et al. 2000).

Granulated blast furnace slag (GBFS) is granular by-product generated from the ferrous industry as primary waste material. Slag mainly consist of calcium, magnesium, manganese and aluminum silicates in different combinations. This makes GBFS moderately active leading to the formation of binding products upon pozzolanic reaction with soil. Hence GBFS can be used in stabilizing soil through its physical and chemical properties.

2.5 Settlement analysis using PLAXIS 2D software

Haghbin studied the bearing capacity of strip footing resting on soil comprising of two-layered soil. From the research work, it was concluded that the ultimate bearing capacity of the layered system depends on the on soil properties (friction angle, cohesion and modulus of elasticity), compacted soil width below the foundation, layer thickness, footing width and depth (Haghbin 2016).

The strip footing was analyzed using PLAXIS 2D, modelled as 15-node triangular elements 12 interior stress points. The findings from the work was that the ultimate bearing capacity of layered soils is largely influenced by the geotechnical properties and thickness of the top soil layer and the foundation width. The capacity depends on the thickness of stronger top layer till a unique critical depth beyond which the influence of bottom layer is least (Mosallanezhad and Moayedi 2017).

The load settlement response of strip footing was analyzed through experimental and numerical investigations. The numerical simulation was performed using PLAXIS 2D software. Granular fill was introduced beneath the footing in the view to minimize the settlement and to improve the load-bearing capacity of the foundation. Compared to natural clay, granular-fill is stronger and stiffer, the replacement of clay with the granular-fill resulted in a redistribution of the applied stress to a wider area and achieving an improved bearing capacity of soil. This approach has also reduced the footing settlement. The numerical and experimental outputs were in good agreement to each other (Ornek et al. 2012).

2.6 Compressed stabilized earth blocks

It's been centuries or even more since human civilization had started building structures using earth materials. One such building material that utilizes soil/earth is the bricks. As the need overcomes the availability of the bricks and the over exploitation of the natural resource for brick production exceeds the limit, there comes up an urgent need for an alternative for brick. One such alternative is the stabilized earth block (Walker and Stace 1997). With the advancement in technology, Compressed Stabilized Earth Blocks (CSEBs) made a tremendous advancement because of their sustainability, durability, and eco-friendly nature when compared with conventional fired bricks. Besides, CSEBs has also reduced the housing cost and made it accessible even for lower-income sections and proved to be an innovative construction material that is highly economic (Riza et al. 2010). The primary advantage of CSEBs is that it can be locally cast in the construction site with semi-skilled or un-skilled labour and doesn't require any specialized equipment. CSEBs can also be used as better thermal and acoustic insulator. CSEBs can be of different sizes based on consumer requirements. Moreover, blocks consume lesser energy compared with the fired bricks or concrete masonry blocks (Walker 2004). Despite these advantages, the CSEB's acceptance percentage is low in the housing sector due to its low performance regarding tensile strength, durability, impact, and abrasion resistance, when compared with traditional fired bricks. These blocks are prepared using wet soil with suitable admixtures and compacted using a manually operated press. Usually, cement or lime is used as additives to bind the soil constituents (Walker 1995).

Many factors decide the performance of the block. The cement percentage plays a vital role in evaluating the strength of the block. As cement/lime is used as additives, curing time also quantifies the strength of the block. A rough estimate of cement proportion was given by Nagaraj. Accordingly, clayey soil requires about 12% to 15% of cement by volume, silty soil requires 8% to 12% cement and sandy soil requires 5% to 9%. Also, beyond 15% of cement consumption was termed as uneconomical (Nagaraj et al. 2014). The percentage of cement required was based on the grain size of the soil. Finer the soil more is the cement required and cost consuming. According to previous literature, 16% of cement is the amount restricted for block manufacture.

An industrial-scale experiment concluded that the harbour sediments can be utilized in brick manufacture as harbour sediments cause no harm/impact on the environment. This utilization of marine clay has solved the problem of its disposal (Cappuyns et al. 2015; Hamer and Karius 2002). The utilization of steel slag in the brunt brick was investigated and the report mentions that the compressive strength and shrinkage dropped with the addition of steel slag. Further calcium and aluminum silicate increased with the higher slag addition (Shih et al. 2004). Though various factors are influencing the strength, durability, and quality of CSEBs, the admixture dosage and the density attainment of the block are the noteworthy parameters. Furthermore, for the selected compaction effort (density) and admixture dosage, soil gradation plays a vital role in the block properties. The soil in its natural form may not meet the possible highest resistance to loads and adverse climatic impact. The soil needs to be reconstituted to attain appropriate gradation. Past research evidenced that high clay content cracked the block while high sand content was unable to sustain sufficient green strength resulting in block crumbling. Hence obtaining the optimum grading limits of soil is a major challenge in the manufacture of CSEBs to yield the highest strength and better durability characteristics of the block (González-López et al. 2018). Olivier and Ali in their investigation suggested that, the soil gradation for achieving maximum strength of CSEBs through the soil of 70 % sand and 20 % clay content(Olivier and Mesbah 1987). Reddy and Jagadish from their experimental data have concluded that coarse-grained soil composed of 70 ± 5 % sand content and 15 % clay content yields maximum strength for CSSB. Reddy and Walker recommend the optimum clay content as 10–12 % for sound CSEBs (Venkatarama Reddy 2009).

Walker and Stace observed noticeable poor performance in the strength and durability characteristics with high clay content. The findings also recommend that for soils with 15% and 30% clay content, the amount of cement required is about 5% and 10% to meet the required criteria (Walker and Stace 1997). Another way of obtaining the optimum clay content is by reconstituting the soil and testing the sample for its unconfined compression test cast at its respective maximum dry density and optimum moisture content. The sample yielding maximum compressive strength is recommended as the best-graded sample (C. Sekhar and Nayak 2018).

In the view of generating a good and durable soil block the clay, silt, and sand proportions were varied. The conclusion from the work is that minimum of 13% of clay content produces durable blocks, while an increase in the clay content beyond this percentage gave reduced strength of the block when cement alone is used as an additive. Whereas, lime in combination with cement showed a better resistance for blocks prepared with higher clay content. At the optimum clay content, the void ratio was observed to attain its least value and this is also the reason to achieve a higher strength of blocks prepared at optimum clay content. To achieve higher strength, it was advised to reconstitute the fine grained soil with sand to bring soil to optimum clay content (Nagaraj and Shreyasvi 2017). The durability test was performed on various combinations of clay content samples. The sample with optimum clay content showed a lower mass loss and hence concluded as durable (Venkatarama Reddy and Latha 2014).

The process of mixing admixtures and soil on account of improving the strength, volume stability, permeability, and durability of the mixture is known as soil stabilization. Additives supplementation to the improved graded soil is an important step involved in the manufacture of CSEBs. Additives, such as cement, lime, alkali-activated binders, synthetic compounds, etc can be used as stabilizers. Amongst these, Portland cement and lime are used as the most popular stabilizers in the manufacture of blocks. Past researchers have attempted the use of cement and documented the performance of CSEBs. Cement stabilized blocks was found more durable with better strength compared to blocks prepared with lime as the stabilizer. The amount and type of additives rely on the characteristics of the soil and expected strength achievement of compressed stabilized earth block (CSEB) (Harikumar et al. 2016; Palanisamy and Kumar 2018; Tripura and Singh 2008).

A higher dosage of binder (cement) can stabilize more efficiently and hence lead to a gain in dry and wet compressive strength but turns out to be uneconomical. The higher the cement content more is the binder coating leading to non-separable and water-insoluble bonds of constituent particles in the mixture. The soil of optimum sand-clay fractions shows better strength with the introduction of cement as clay occupies the void space between the sand particles and cement bind these constituent particles. The CSH gel formed upon hydration of cement also occupies the void space leading to the highest possible degree of packing. This matrix is impermeable to moisture movement and resistant to erosion and climatic change. The improved performance of the cement stabilized blocks is through the formation of C_2S and C_3S in the mixture. With the higher dosage of cement, more quantities of binding compounds are produced leading to non-separable particles of the mixture. If fine particles (clay) in the soil are more, the quantity of cement required to coat the particles increases (Kumar et al. 2017). Upon immersing the blocks into the water, the development of pore pressure leads to the formation of cracks and failure initiates. Hence wet compressive strength plays the role in deciding the performance of the block. Sand blending is required for the clay-rich soils to reduce the clay fraction and to reach the optimum clay content for the manufacture of CSEBs. This process of modifying the gradation property with sand is however, uneconomical as sand being a natural resource gets depleted fast through this process and conservation of the same will be of concern (Patra and Mukharjee 2017).

2.7 Summary

The review of literature is concentrated initially on the study of marine clay, its performance and associated problems. Many researchers have contributed significantly in the area of stabilization of marine clay and their applications in the engineering field.

The influence of various admixtures like cement, lime, fly ash, synthetic compounds, etc. on the physical, chemical and mechanical behaviour of marine clay was evaluated. Marine clay is a fine-grained soil with poor gradation properties. The granular material (tile waste, rubber waste, sand, etc) incorporation to the fine grained marine clay has improved the geotechnical properties of marine clay. Recently, the utilization of granular industrial by-products in every field is highly focussed. In this regard, Quarry dust and Granulated blast furnace slag were incorporated in the stabilization of fine-grained soils for achieving better gradation and improved properties. Quarry dust is composed of inorganic, rough-textured angular and inert particles. GBFS is a moderately active, rough-textured granular material. The utilization of these industrial by-products can be economical and solves its disposal problem. Need-based detailed research was not performed in the stabilization of marine clay using quarry dust, GBFS, and cement. Hence this motivated in evaluating the performance of stabilized mix for the geotechnical engineering applications.

CHAPTER 3

MATERIALS AND METHODOLOGY

3.1 Introduction

The characterization and examination of soil properties are very important for a geotechnical engineer. In any construction project, the index, compaction, strength, and settlement characteristics govern the major decision in the selection of soil as a construction material. To characterize the untreated soil and stabilized soil, various laboratory tests were conducted. The untreated soil and the additives used to stabilize the weak soil in various combinations are depicted in the present chapter. All the test procedures are performed as per the Bureau of Indian Standards.

3.2 Materials

Marine clay is a soft consistent soil that is widely available in the coastal regions of the world. Marine clay was procured from Chitrapura, Mangalore, Dakshina Kannada, Karnataka, India located at 12.95° N, 74.80° E. By removing the top soil, marine clay was taken at a depth of 2 m from the ground level and transported to the laboratory (Figure 3.1). For the establishment of physical and geotechnical properties, the soil was subjected to various geotechnical investigations. Initially, tests for analyzing the geotechnical properties and chemical properties like silica content, *pH*, tests, etc. were performed.

Then the treatment of the fine-grained marine clay with comparatively granular additive quarry dust and granulated blast furnace slag was carried out separately. Quarry dust (Figure 3.2) was procured from Pakshikere, Mangalore taluk of Dakshina Kannada district of Karnataka, India. Granulated Blast Furnace Slag (GBFS) (Figure 3.3) was procured from Kirloskar Ferrous Industries Limited, Koppal, Karnataka, India. Before the treatment of the soil (marine clay) with quarry dust and granulated blast furnace slag, the initial characterization of the additive was carried out.

The tests include those for evaluating the oxides of calcium, silica, alumina, *pH*, etc. The mechanical tests were also conducted which includes the gradation and strength tests.

Fig. 3.1 Marine clay

Fig. 3.2 Quarry dust

Fig. 3.3 Granulated blast furnace slag

Cement (Ultra Tech 53 grade Ordinary Portland Cement) was added to marine clay in various percentages (2%, 4%, 6%, 8% and 10%) of the dry weight of soil. Later, Marine clay was replaced by quarry dust in various percentages (10%, 20%, 30%, 40% and 50%) and analyzed for its geotechnical performance. The optimum quarry dust percentage replacing the soil was established through unconfined compression test results. For the optimum quarry dust replacing the soil, cement was added in proportions of 2%, 4%, 6%, 8%, and 10% of the dry weight of the optimum quarry dust-soil mixture and analyzed for its strength and serviceability. A similar procedure was carried out with granulated blast furnace slag instead of quarry dust. Experiments were performed to analyze the strength and the index properties of the soil treated with the optimum granular mix. For conduction of strength, consolidation and durability test, specimens were cast for their corresponding maximum dry unit weight and optimum moisture content. These samples were then cured for 28 days and later tested. After curing, samples were saturated and tested for Unconsolidated Undrained (UU) triaxial test and consolidation test.

3.3 Laboratory tests

The following tests were performed to evaluate the technical parameters of the untreated and treated soil.

- Gradation analysis: sieve analysis and hydrometer analysis
- Atterberg's limits
- Specific gravity test
- Standard proctor test
- Unconfined compression strength test
- California bearing ratio test
- Unconsolidated undrained triaxial compression test
- Permeability test
- Durability test
- Chemical tests

3.4 Standard laboratory test procedure

All the laboratory tests performed to evaluate the geotechnical properties of untreated and treated soils were according to IS 2720 and SP 36. Both index and strength properties were analyzed. The procedure adopted to analyze the technical properties of the untreated and treated soil is briefly discussed in the subsequent sections.

3.4.1 Grain size analysis

Soil and granular additives (quarry dust/slag) individually were washed thoroughly by distilled water using a 75-micron sieve until the clear water is observed. The retained mass on the sieve was analyzed for sieve analysis and the collected slurry when washed was examined for sedimentation analysis. Particulate materials (soil/granular additives) were sieved using a set of sieves and the retained weights of the material on the various sieves were noted. These values can be used to estimate the distribution of coarser fraction in the sample. For analysis of the finer fractions (less than 75 microns) hydrometer test was performed. Around fifty grams of the soil passing 75 microns sieve were used in the test. The soil, dispersion agent and distilled water were thoroughly mixed using the mixer. The hydrometer measures the specific gravity of the soil-water suspension at the mid-section of the bulb, which directly depends on the diameter of solids present at that particular instant. The Grain size analysis test performed was in accordance with IS 2720 (Part 4)-1985 (Reaffirmed 2006).

3.4.2 Atterberg's limit tests

The consistency limits of the soil decide the amount of clay and silt in the soil mass. The liquid limit indicates the compressibility character of the soil, whereas the plastic limit is an indicative of the shear strength of the soil. Particle fractions finer than 425 microns are used for the analysis. The liquid limit was determined using Casagrande's standard mechanical liquid limit instrument. Liquid limit is the water content at which the groove cut in a soil pat by a standard grooving tool will flow together and make a contact for a width of 13 mm for 25 blows. From the plot of water content and the number of blows in a semi-log graph, the liquid limit of the untreated and stabilized soil mix was determined.

The plastic limit is the water content corresponding to the soil state, when the soil is threaded into a 3 mm diameter roll, just begins to crumble. Tests were performed to determine the consistency limits of soil samples are as per IS 2720(Part 5)–1985 (Reaffirmed 2006).

3.4.3 Specific gravity test

Specific gravity is the ratio of weights of a given volume of soil solids to an equal volume of water at a standard temperature of 27 °C. Specific gravity is indicative of the minerals present in the soil. During the testing, care must be given so that the entrapped air is expelled out from the sample. The test procedure is as per IS 2720 (Part 3)-1980 (Reaffirmed 2002).

3.4.4 Standard compaction test

The standard compaction test which is also known as the light compaction test was performed to evaluate the degree of possible achievement of packing of the soil or stabilized matrix. The Maximum Dry Unit Weight (MDU) with its corresponding Optimum Moisture Content (OMC) can be assessed through the compaction test which is a basic requirement in the construction field. The instrument and test procedures adopted are in accordance with IS 2720 (Part 7)-1980 (Reaffirmed 2011).

3.4.5 Unconfined compressive strength test

The Unconfined Compressive Strength (UCS) is the capacity at which a cylindrical soil specimen without confinement, fails in compression. The maximum resistance to compression is termed as the unconfined compressive strength of the soil. Unconfined compressive strength tests were conducted on samples that are compacted to their respective maximum dry unit weights and optimum moisture contents as obtained from the standard compaction test. The unconfined compressive strength is the maximum load resisted per unit cross-sectional area, or the load per unit area corresponding to 20% of axial strain, whichever is attained first during the test. The test conducted was according to IS 2720 (Part 10)-1991 (Reaffirmed 2006).

3.4.6 California bearing ratio test

The California Bearing Ratio (CBR) is the load required to impregnate the soil mass with a standard circular piston at the strain rate of 1.25 mm/minute to standard load. The CBR test performed is to determine the suitability of the material as subgrade, in the sub-base and base for a flexible pavement system. The load required by the standard plunger for penetration of 2.5 mm and 5 mm were noted. The ratio of penetration load to the standard load at the respective penetration level of 2.5 mm or 5 mm was determined. To replicate the worst conditions and rainfall in the field, the samples were soaked for 4 days in a water bath before testing. The test procedure followed was according to IS 2720 (Part 16)-1987 (Reaffirmed 2002).

3.4.7 Triaxial compression test–UU test

The Unconsolidated Undrained (UU) triaxial compression test was performed to analyze the shear strength parameters (cohesion and friction) of the soil samples. The specimen was subjected to three-dimensional compressive stresses in mutually perpendicular directions (Figure 3.4). The sample was saturated to simulate the groundwater table or the worst conditions in the field. The maximum deviatoric stress corresponds to the maximum resistance of the sample before failure. This test is called a quick test as the test is comparatively faster than other triaxial tests. Samples cured at 7 and 28 days were used to evaluate the shear strength parameters of the stabilized mix. The tests follow the procedure prescribed in IS 2720 (Part 11)-1993 (Reaffirmed 2002).

Fig. 3.4 Triaxial apparatus

3.4.8 Permeability test

Permeability is a measure of the ease of moisture movement through the void spaces in the soil mass. Tests were performed to determine the coefficient of permeability of the samples. The test procedure is according to IS 2720 (Part 17)-1986 (Reaffirmed 2002).

3.4.9 Durability test

Durability is the resistibility of treated soil matrix to retain its integrity and stability over a long term of exposure to climatic change or any other weathering forces. The measure of the durability is quantified by two methods, which are discussed below:

- Resistance to loss of strength

  Samples cured for 28 days were immersed in water for 7 days and tested for their unconfined compressive strength. This soaked compressive strength is compared with unsoaked compressive strength in their respective ratio. This ratio (R_i) denotes the resistance to loss of strength upon immersion in water which is a measure of the durability of the stabilized mix (BS 1924 (Part 2) (1990)).

- Alternate wetting and drying

  Samples cured for 28 days were subjected to a series of alternate drying and wetting cycles. Each cycle involves 5 hours of complete soaking the samples in water and subsequent drying the soaked samples at a temperature of 72 °C for 42 hours in a thermostatically controlled oven. The oven-dried specimens are then removed and scratched using a wire brush. The scratching process involves an application of 1.5 kg force on all six sides with two strokes per side. The scratched block is weighed and the mass loss is noted, which completes one cycle of wetting and drying. This weight loss is a measure of the resistibility of stabilized soil for severe climatic exposure and hence a measure of durability (ASTM D559 (2003)).

3.4.10 *pH* test

The acidity and alkalinity of the soil sample are quantitatively measured by its hydrogen ion concentration which is denoted as *pH*. An electrically connected electrode assembly can directly measure the suspension of soil-distilled water relatively with a saturated potassium chloride solution. The test is conducted according to IS 2720 (Part 26)-1987 (Reaffirmed 2002).

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

The different chemical compounds in study materials were determined as per the procedure prescribed in IS 1727- 1967 (Reaffirmed 2004).

3.5 Experimental program on compressed stabilized earth block

For the preparation of the earth block, the MARDINI block making machine was employed. A manual toggle lever-based ram generating a pressure of 2-3 MPa was attached to compress the mixture to a block of size 230mm x108mm x75mm. The calculated amount of optimum granular materials-soil and cement were dry mixed and spread on a big tray. For the block to eject successively as one unit, optimum block water content was determined based on the initial trials. To obtain this, dry mixed soil-additives mixture was sprinkled with water and thoroughly mixed. The limiting water content sufficient to prepare an intact ball made of this mix without sticking on hand is the water content selected for each mixture. The wet mixture thus prepared was transferred to the press and compressed. The ejected soil block was labelled for its identification and cured for 28 days.

3.5.1 Compressive strength

The 28 days cured stabilized block prepared as mentioned in the previous section was tested for its dry and wet compressive strength. The possible worst condition is its loss of strength during moisture encounter and hence its wet compression strength is critical. The cured blocks were immersed in water for 48 hours. Later the block was removed from the water bath, its surface was wiped and its mass and dimensions were noted. The sample was sandwiched between two 10mm thick iron plates and placed in a universal testing machine before the loading is introduced. The sample is loaded at a rate of 2 $N/mm^2/min$. The detailed procedure is outlined in IS 3495 (Part 1)-1992 (Reaffirmed 2002). Six samples of each combination were tested.

3.5.2 Water absorption and rate of water absorption

The level of compactness of the block can be assessed through the water absorption test, as water occupies the void spaces in the matrix. Lower the water absorption value higher is the density attained by the block as the value indicates the least voids. Water absorption tests were performed as per IS: 3495 (Part 1)-1992 (Reaffirmed 2002). Cured blocks were oven-dried at a temperature maintained at 60 °C. The dry mass of the blocks was accurately recorded and immersed in a water bath for 24 hours. Later, the surface of the blocks was wiped by a damp cloth and weighed again. Water absorption can be calculated based on the weight gain of the block. The rate of water absorption is calculated by conducting a similar procedure of recording the mass of the block at the end of different time intervals at 1, 2, 4, 6, 8, 10, 15, 20 minutes, 0.5, 1, 4, 6, 24 and 48 hours.

3.5.3 Initial water absorption

The capillary suction of the block is obtained by the initial water absorption test as initial water absorption takes account of the moisture absorption from the mortar during the construction time. This test was carried out according to ASTM C 67-02c. The dimensions of the block coming in contact with water were noted. The initial dry weight of the specimens was recorded. Two supports were placed inside the tray to support the block and water is filled inside such that a constant level of 3.18 ± 0.25 mm of water height is maintained above the supports when the block is placed. To achieve this water level, a waxed reference block (to ensure that no moisture will be absorbed by the block) was used. The waxed block was placed and water is filled inside the tray till a level of water of 3.18 ± 0.25 mm is achieved. Later the waxed block is removed and the reference mark was marked at the surface water level on the edge of the tray. For the determination of Initial water absorption water should be filled up to that mark. The study specimen was placed on the support and the stop clock was started simultaneously. At the end of a minute, the block was taken out from the tray and weighed again after the surface was wiped using a damp cloth. The initial rate of water absorption is calculated and reported to the nearest 0.1 g/min/cm^2.

3.5.4 Durability studies

Alternate wetting and drying method was employed to evaluate the durability of the blocks (IS 1725-2013). The test involves weight loss monitoring for 12 cycles of alternate wetting and drying. The cured blocks were initially dried at a temperature of 60°C by a thermostatically controlled oven till it attains a constant weight. The initial mass of the specimen was noted and immersed in water for 6 hours. Later the block was kept in an oven maintained at a temperature of 70 °C for 42 hours. The oven-dried specimens are then removed and scratched using a wire brush. The scratching process involves an application of 1.5 kg force on all six sides with two strokes per side. The scratched block is weighed and the mass loss is noted which completes one cycle of wetting and drying. 12 such similar cycles were performed and the specimen was finally dried at 60 °C until it attained a constant weight. The mass loss percentage is evaluated. The procedure adopted is in accordance Indian Standards (IS 1725-2013).

3.6 Summary

For the evaluation of index, shear strength and settlement properties, various tests are discussed. The performance of the treated soil can be justified and their usage in various geotechnical applications can be advocated by performing these experimental investigation and durability tests.

CHAPTER 4

STABILIZATION OF MARINE CLAY USING QUARRY DUST AND CEMENT

4.1. General

In the present investigation, detailed laboratory tests were carried out on the untreated marine clay, quarry dust, marine clay stabilized using quarry dust, cement treated marine clay and marine clay stabilized using quarry dust and cement. To analyze the performance of stabilized mix, several tests were conducted on both un-stabilized and stabilized marine clay. All the test results for quarry dust and cement stabilized marine clay are tabulated and characterized systematically in this chapter.

4.2. Properties of marine clay

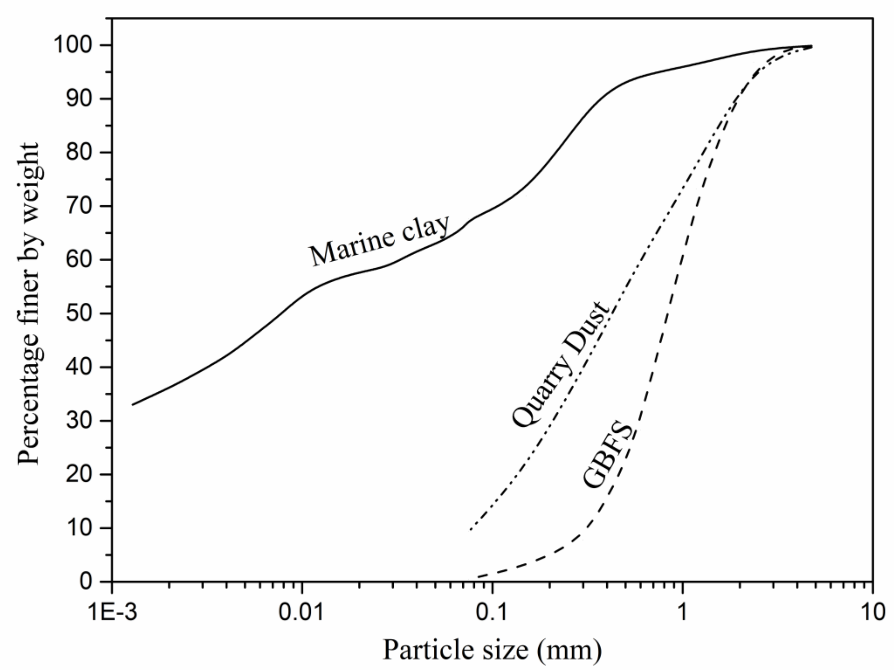


Fig. 4.1 Gradation curve of marine clay and granular additives

Marine clay is soft consistent soil. Marine clay was air-dried before testing. This soil is problematic due to its poor shear strength and compressibility properties. The geotechnical properties of untreated marine clay are summarized in Table 4.1. The grain size distribution curve of marine clay is shown in Figure 4.1. From the figure, it is noticed that the amount of sand, silt and clay size particles are more or less the same. The high liquid limit (91%) indicates that the soil is highly compressible. The plasticity index is 58%, making the soil fall under the CH (clay of high plasticity) category as per Indian Standard (IS) classification. The optimum moisture content (OMC) is 27% and the maximum dry unit weight (MDU) is 13.6 kN/m^3. The high value of liquid limit and fines percentage (69%) indicates that the soil possessing a more specific surface area compared to other soils (Mohammed Al-Bared and Marto 2017)

Table 4.1 Properties of marine clay

Sl. No.	Properties	Particulars
1	Particle size distribution	
	Sand size (%)	31
	Silt size (%)	33
	Clay size (%)	36
2	Specific Gravity	2.6
3	Atterberg's limits	
	Liquid limit (%)	91
	Plastic limit (%)	32
	Shrinkage limit (%)	14
	Plasticity index (%)	58
	IS Classification	CH
4	Compaction characteristics	
	Maximum dry unit weight (kN/m^3)	13.6
	Optimum moisture content (%)	27
5	Unconfined compressive strength (kPa)	98
6	Shear strength parameters	
	Cohesion c_{uu} (kPa)	19
	Angle of internal friction ϕ_{uu} (degrees)	13
7	*pH*	7.2

4.3 Stabilization of marine clay using cement

Cement is a traditional stabilizer used in the stabilization of various types of soil. In cement stabilization, the strength gain is due to the hydration and pozzolanic reaction. Cement was added to marine clay in various percentages from 2% to 10%, with an increment of 2% by the dry weight of soil.

4.3.1 Effect of cement addition on index properties of marine clay

The variation of index properties is shown in Table 4.2.The liquid limit decreases from 91% to 76% with the addition of cement (10%) to the soil. The plasticity index decreases by 36%. Maximum dry unit weight increases by 8% upon cement addition (10%) to marine clay. As cement is heavier compared to soil, MDU increases. marginal changes in OMC is observed. The index properties are analyzed before the initiation of hydration reaction, as the tests were conducted immediately after the addition of cement (Nayak and Sarvade 2012).

Table 4.2 Index properties of marine clay with cement addition

Sl. No.	Properties	Marine Clay	Percentage addition of cement to soil				
			2%	4%	6%	8%	10%
1	Liquid limit w_L (%)	91	91	90	86	83	76
2	Plastic limit w_P (%)	33	33	33	36	38	39
3	Plasticity Index $I_{P(\%)}$	58	58	57	50	45	37
5	MDU (kN/m^3)	13.6	13.6	13.7	13.7	14.1	14.7
6	OMC (%)	27	27	26.5	26.2	25.4	25

4.3.2 Effect of cement addition on shear strength properties of marine clay

Table 4.3 UCS of marine clay with cement addition and curing age

Percentage Cement addition to Marine clay	UCS (kPa) with the duration of Curing		
	0 day	7 day	28 days
0	98	98	98
2	98	182	249
4	107	296	379
6	121	417	501
8	148	487	582
10	157	541	697

The UCS value of marine clay increases with the addition of cement and curing period (Chew et al. 2004) as shown in Table 4.3 and Figure 4.2. The strength increases by 600% with the addition of 10% cement after 28 days of curing.

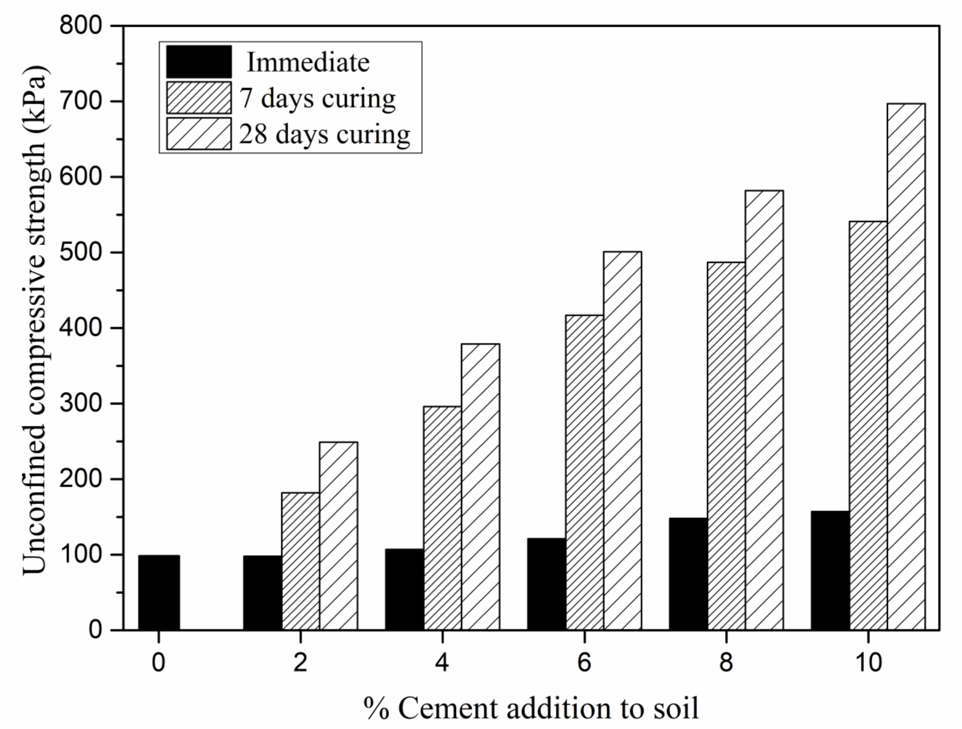


Fig. 4.2 Variation in UCS for different proportions of cement added to marine clay upon curing

Table 4.4 UCS and shear strength parameters of marine clay stabilized with cement

Curing age	Strength properties obtained from triaxial UU test	Marine Clay	Percentage cement added to soil				
			2%	4%	6%	8%	10%
0 days	Cohesion c_{UU} (kPa)	19	19	19	23	24	24
	Angle of internal friction ϕ_{UU} (degrees)	14	14	14	14	15	15.5
7 days	Cohesion c_{UU} (kPa)	19	21	28	44	58	76
	Angle of internal friction ϕ_{UU}(degrees)	14	15	16	16.5	19	20
28 days	Cohesion c_{UU} (kPa)	19	26	40	58	82	108
	Angle of internal friction ϕ_{UU} (degrees)	14	16.5	18	19	21	21.5

The variations in shear strength parameters of cement stabilized marine clay upon curing are presented in Table 4.4. The shear strength parameters increase with the higher dosage of cement to marine clay. The shear strength parameters show minimal changes with cement addition when tested on the day of casting. This is because the hydration reaction has not been initiated. Upon curing, the cohesion tremendously increased by 4.7 times with the addition of 10% cement and cured for 28 days (Figure 4.3).

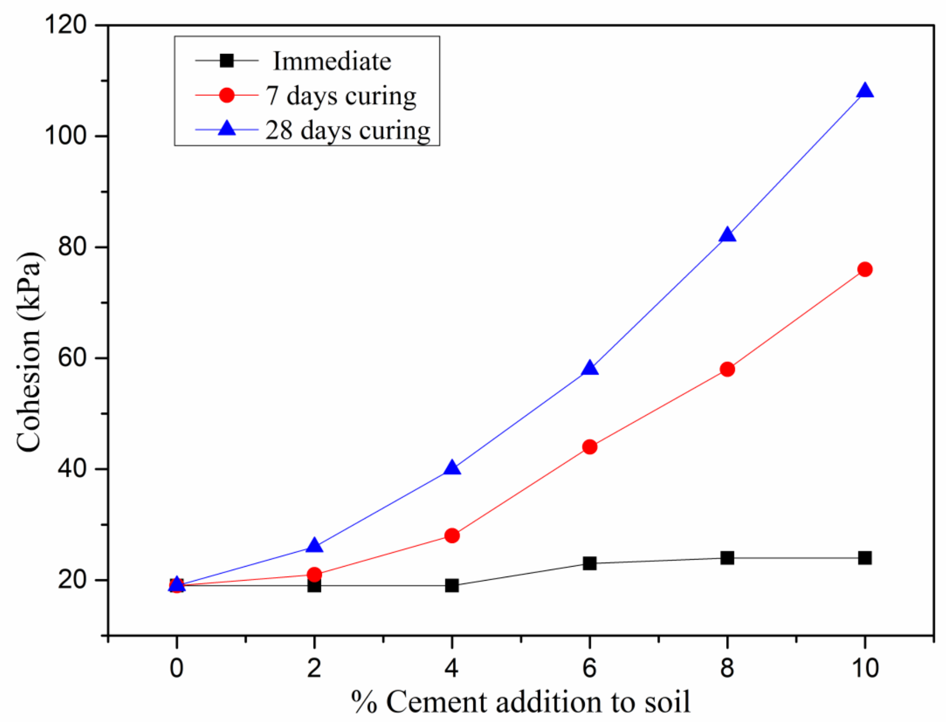


Fig. 4.3 Variation in cohesion for different proportions of cement added to marine clay upon curing

4.4 Stabilization of marine clay using quarry dust and cement

Marine clay was replaced by quarry dust in different proportions from 10% to 50% and investigated for its various geotechnical properties. From the laboratory test results, the optimum dosage of quarry dust replacing the soil is established. For the optimum quarry dust-soil mix, cement is added in various proportions (2%, 4%, 6%, 8% and 10%). The results obtained are analyzed and discussed in the present chapter.

4.4.1 Properties of quarry dust

The laboratory tests were conducted on quarry dust and the results obtained are tabulated in Table 4.5.

Table 4.5 Properties of quarry dust

Sl. No.	Properties	Particulars
1	Specific gravity	2.76
	Grain size distribution	
	Coarse sand size (%)	7
2	Medium sand size (%)	45
	Fine sand size (%)	39
	Clay and silt	9
	Strength parameters	
3	Cohesion (kPa)	4
	Angle of internal friction ϕ (degrees)	45
4	*pH*	7.8
5	Calcium oxide (%)	3.25
6	SiO_2 (%)	71.3
7	Fe_2O_3 (%)	4
8	Al_2O_3 (%)	17.5
9	MgO (%)	2.1
10	Loss on ignition (%)	0.28

The grain size distribution curve of quarry dust is shown in Figure 4.1. It can be noticed that quarry dust has predominantly medium and fine sand-sized particles. The maximum and minimum dry densities of GBFS are 17.6 kN/m^3 and 13.8 kN/m^3.For its 95% of relative density, the direct shear test results showed the frictional angle of 45° and cohesion intercept of 4 kN/m^2. The *pH* value of quarry dust is 7.8 and the calcium oxide (CaO) content in quarry dust is 3.25%, hence quarry dust can be concluded as inert material. Quarry dust is heavier (G=2.76) compared to marine clay (G=2.6).

4.4.2 Effect of quarry dust on index properties of marine clay

The variation of consistency limits and compaction characteristics of marine clay due to the replacement of marine clay by quarry dust are shown in Table 4.6. Specific gravity increases with quarry dust addition because quarry dust (G=2.76) is heavier than marine clay (G=2.6). With the addition of quarry dust to marine clay, the liquid limit decreases (Amadi 2014b). The reduction is due to the addition of granular quarry dust to the finer soil. For 35% quarry dust replacing the soil, the liquid limit and plasticity index reduces by 33% and 41% compared with untreated soil respectively. With the addition of heavier quarry dust (G=2.76) to soil (G=2.6), maximum dry unit weight increases and the optimum moisture content decrease (Soosan et al. 2005). About 23% increase in MDU and 33% decrease in OMC is observed for a mix of 35% quarry dust replacing the soil. The cumulative surface required to coat the particles decreases with the increased dosage of quarry dust to soil and hence the optimum moisture content decreases. The increased MDU and decreases OMC leads to better performance of the mixture and hence the ground is improved (Chetia et al. 2018).

Table 4.6 Index properties of marine clay stabilized using quarry dust

Sl. No.	Properties	Marine Clay	Percentage of quarry dust replacing soil					
			10%	20%	30%	35%	40%	50%
1	Liquid limit w_L (%)	91	76	69	62	61	58	55
2	Plastic limit w_P (%)	33	30	29	27	27	26	26
3	Plasticity Index I_P(%)	58	46	40	35	34	32	29
4	Specific gravity (G)	2.6	2.6	2.62	2.65	2.65	2.66	2.68
5	MDU (kN/m^3)	13.6	14.5	15.5	16.3	16.7	16.9	17.2
6	OMC (%)	27	25.2	22.5	19.3	18	17.5	16.5

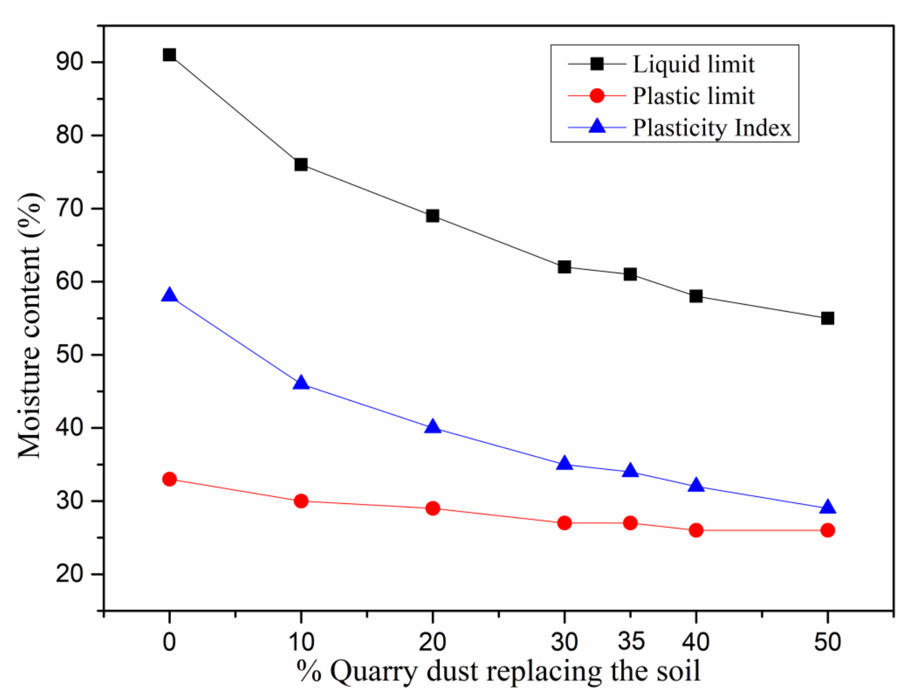


Fig. 4.4 Variation in Atterberg's limits for different proportions of quarry dust replacing the marine clay

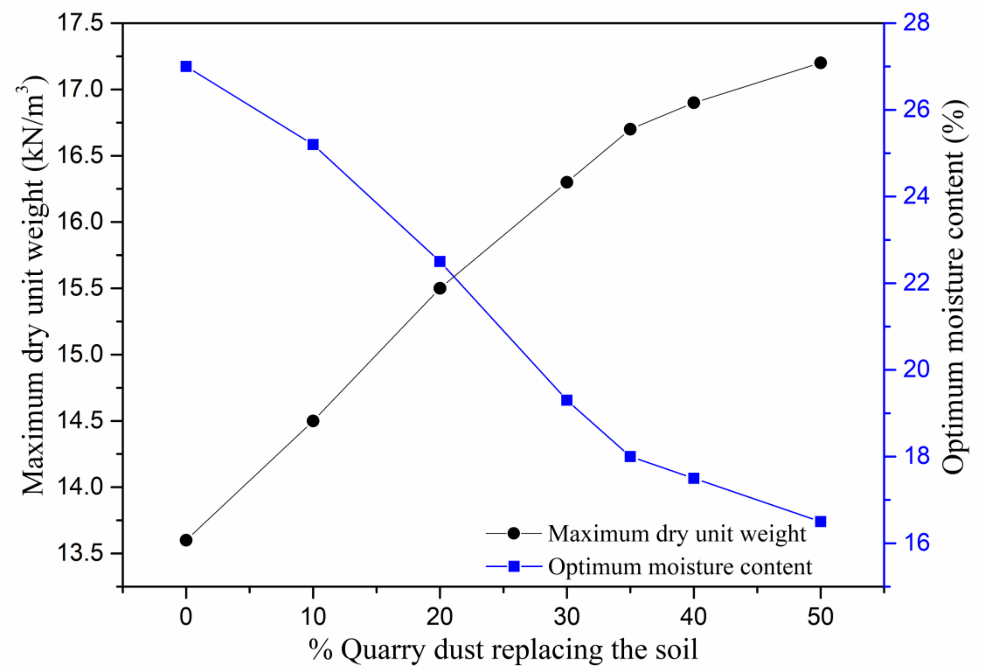


Fig. 4.5 Variation in MDU and OMC for different proportions of quarry dust replacing the marine clay

4.4.3 Effect of quarry dust on UCS of marine clay

The UCS of untreated soil and soil-quarry dust mix for various combinations is shown in Figure 4.6. The unconfined compressive strength increases till 35% quarry dust replacing the soil. With further-more quarry dust addition, the UCS decreases due to the lack of confinement and decrease in the cohesion of the mixture. The strength has enhanced due to the better packing, higher density attainment with improved gradation characteristics of the soil-quarry dust mix (Table 4.7). Hence 35% quarry dust replacing the soil is considered as optimum quarry dust-soil mix as it exhibits the maximum strength (Abraham 2006).

Table 4.7 UCS with different percentage of quarry dust replacing marine clay

	Percentage quarry dust replacing the soil						
	10%	20%	30%	35%	40%	45%	50%
UCS (kPa)	126	160	242	281	275	250	230

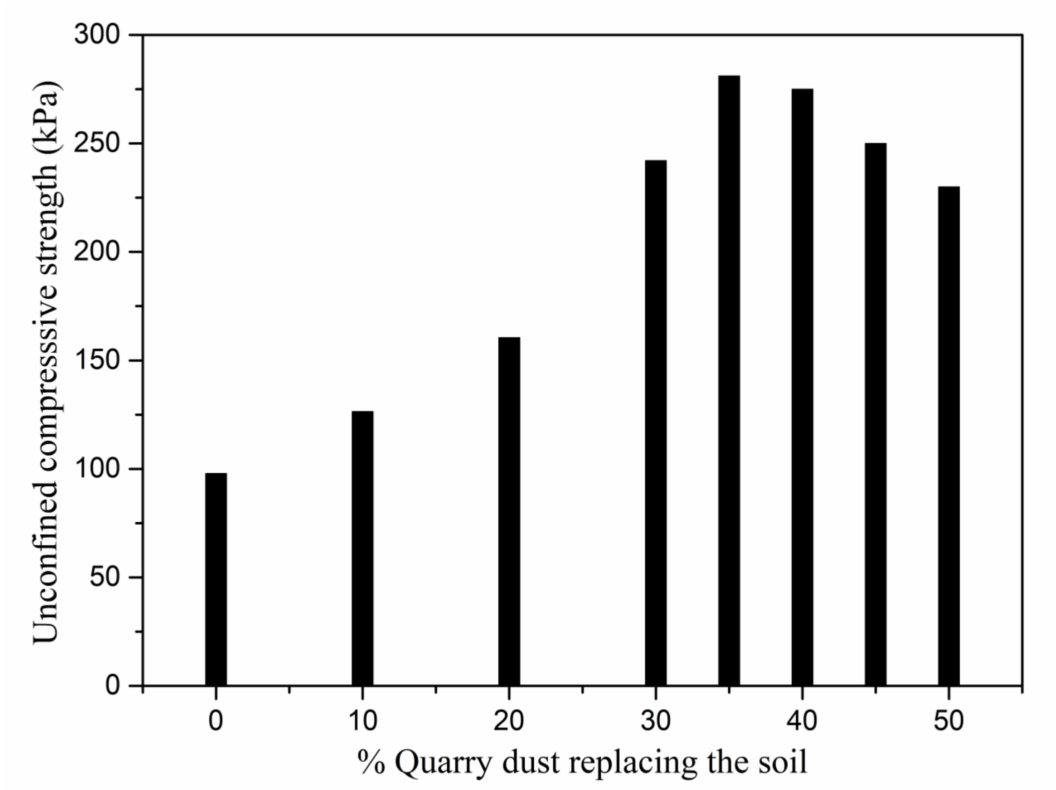


Fig. 4.6 Variation of UCS of marine clay and quarry dust blend

4.4.4 Effect of quarry dust on shear strength parameters of marine clay

To estimate the shear strength parameters, the unconsolidated undrained triaxial tests were conducted on saturated quarry dust-soil specimens. The test results of the various combinations are shown in Table 4.8 and variation is plotted in Figure 4.7. Quarry dust has a rough texture and angular particles. This physical property imparts better interlocking and thus the friction angle increases with the addition of quarry dust to the soil. With the increased replacement of marine clay by inert quarry dust, the cohesion of the mix decreases which is justified through the triaxial results (Nayak and Sarvade 2012). To increase the cohesion of the mix which is now a better-graded mix, cement is introduced.

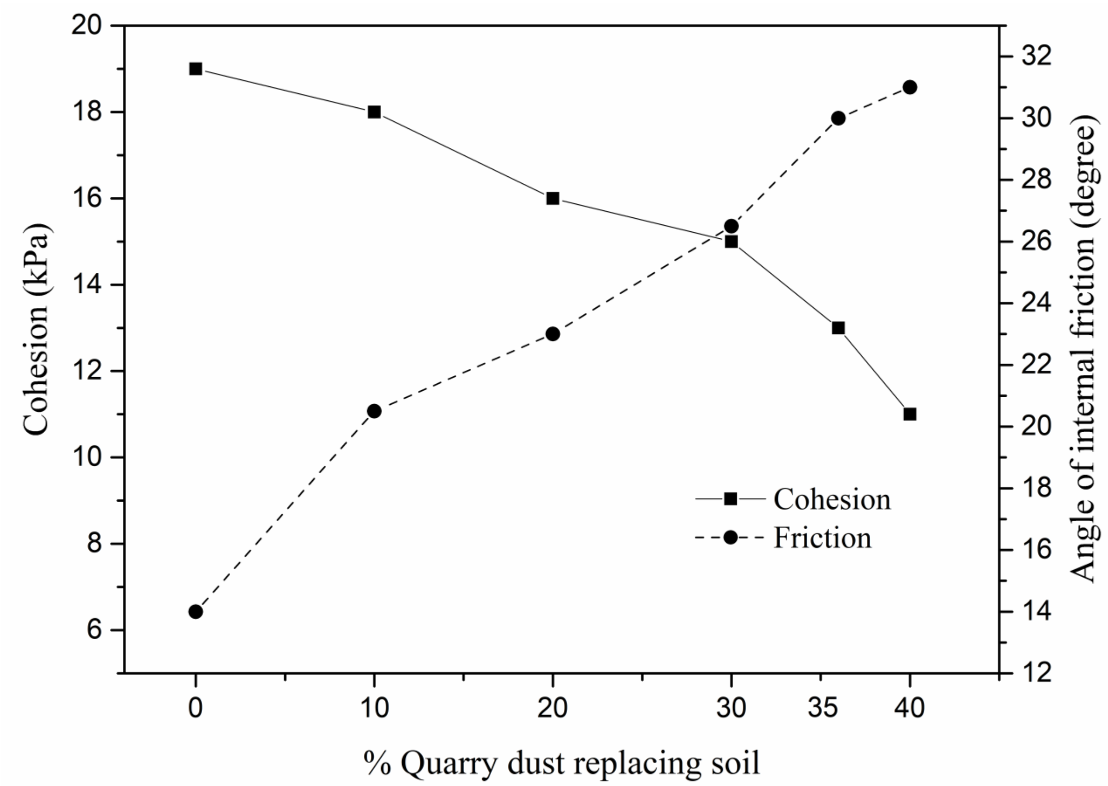


Fig. 4.7 Variation in frictional angle and cohesion for different proportions of quarry dust replacing the marine clay

4.4.5 The effect of cement addition to Optimum quarry dust-marine clay mixture on the geotechnical properties of marine clay

For the optimum quarry dust-soil mix (65% soil+35% quarry dust) cement is added in proportions of 2%, 4%, 6%, 8% and 10% of the dry weight of optimum quarry dust-soil mix. The various combinations were examined for their strength properties. The combined effect of cement and quarry dust on the soil is significant (Nayak and Sarvade 2012). This tremendous improvement is due to the decreased amount of fine fractions and the addition of hydraulic binder. The binder (cement) efficiency is more for the improved graded mix. Hence the quarry dust improves the angle of internal friction and cement improves the cohesion of the mix (Nayak and Sarvade 2012). Also, the improvements in failure envelope and stress-strain behaviour are shown in Figs. 4.8 and 4.9.

Table 4.8 UCS and shear strength parameters of marine clay stabilized with optimum quarry dust and cement

Curing age	Strength properties	Marine Clay	65% Marine clay + 35% quarry dust + varying % of cement addition					
			0%	2%	4%	6%	8%	10%
0 days	UCS (kPa)	98	281	283	290	311	329	336
	Cohesion c_{UU} (kPa)	19	15.5	16	17	19	21	25
	Angle of internal friction ϕ_{UU} (degrees)	14	25	27	28	29	30.5	32
7 days	UCS (kPa)	98	281	444	565	719	971	1258
	Cohesion c_{UU} (kPa)	19	15.5	41	61	95	120	142
	Angle of internal friction ϕ_{UU} (degrees)	14	25	29	32	35.5	41	43
28 days	UCS (kPa)	98	281	564	928	1102	1350	1640
	Cohesion c_{UU} (kPa)	19	15.5	49	102	142	170	196
	Angle of internal friction ϕ_{UU} (degrees)	14	25	30	33	37	41	44.5

California Bearing Ratio test on Marine clay and treated marine clay specimens cured for 28 days and soaked for 96 hours was conducted. From the test results (Table 4.9), it is observed that CBR value steadily increases with the increase in addition of quarry dust. For the optimum quarry dust-soil mix, CBR increased by 3.61 times compared to CBR of marine clay specimen prepared at its MDU and OMC. Further, when cement was added to the optimum quarry dust-soil mix, CBR value enhanced to a higher value.

Marine clay and marine clay treated by quarry dust samples failed in both of the durability tests. These samples crumbled upon immersion to water. Cement added to optimum quarry dust-soil samples performed well.

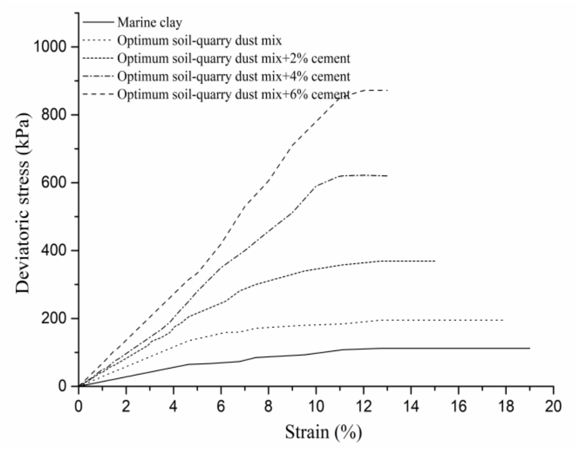


Fig. 4.8 Stress strain graph for different proportions of cement to Opt. Quarry dust-soil mix

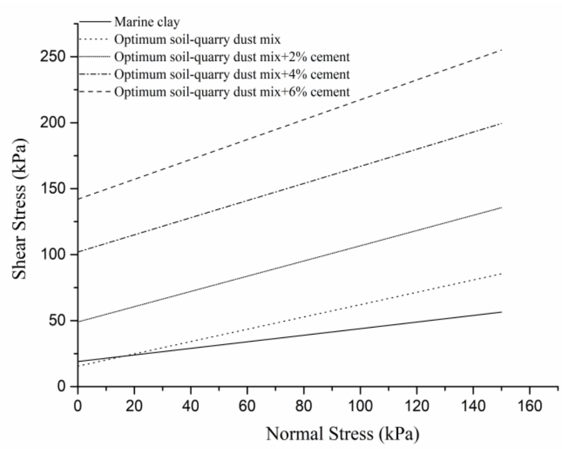


Fig. 4.9 Failure envelope for different proportions of cement to Opt. Quarry dust-soil mix

Table 4.9 CBR value of marine clay stabilized with quarry dust and cement

Property	Marine clay	Percentage quarry dust replacing the soil						Percentage cement added to optimum quarry dust-soil mix		
		10%	20%	30%	35%	40%	50%	2%	4%	6%
CBR (%)	2.6	3.6	7	10.2	12	15	17.3	22	41	63

Table 4.10 Durability performance of marine clay stabilized with optimum quarry dust and cement

Sl.No	Durability properties		65% Marine clay + 35% quarry dust + varying % of cement addition				
			2%	4%	6%	8%	10%
1	R_i(%)		79.1	83.6	89.3	95.7	96.4
2	Loss of mass on alternate wetting (W) and drying (D) (%)	W	6.1	4.9	4.2	3.7	2.5
		D	11.6	9.3	7.5	6.1	5.1

The minimum R_i value that IRC-89 recommends is 0.8 for the stabilized mix to be used as a sub-base material in pavement construction. Accordingly, 4% cement added to optimum quarry dust-soil mix meets the criteria. Cement added to optimum quarry dust-soil mix passed the durability test, as the upper limit for the mass loss is 14% as per IRC 89. Higher the cement content lower is the mass loss and strength loss on soaking (Table 4.10). This is because of the better bonding between the constituent particles of the mix and hence the better performance (Shahu et al. 2014).

4.4.6 X-Ray Diffraction analysis

For the mineralogical characterization of soils, a well established approach is the X-ray diffraction (XRD) analysis. Based on the unique crystalline structure, the corresponding minerals are identified. As XRD depicts the mineralogical alteration due to the stabilization processes, the analysis is vital. The mineralogical changes that occurred due to the introduction of cement and admixtures to the soil originate new crystalline minerals (compounds) in the mix can also be analyzed. Figure 4.10 shows the XRD graph for powdered untreated marine clay. The figure shows the presence of clay minerals, such as kaolinite and montmorillonite. Quartz, a non-clay mineral is also identified.

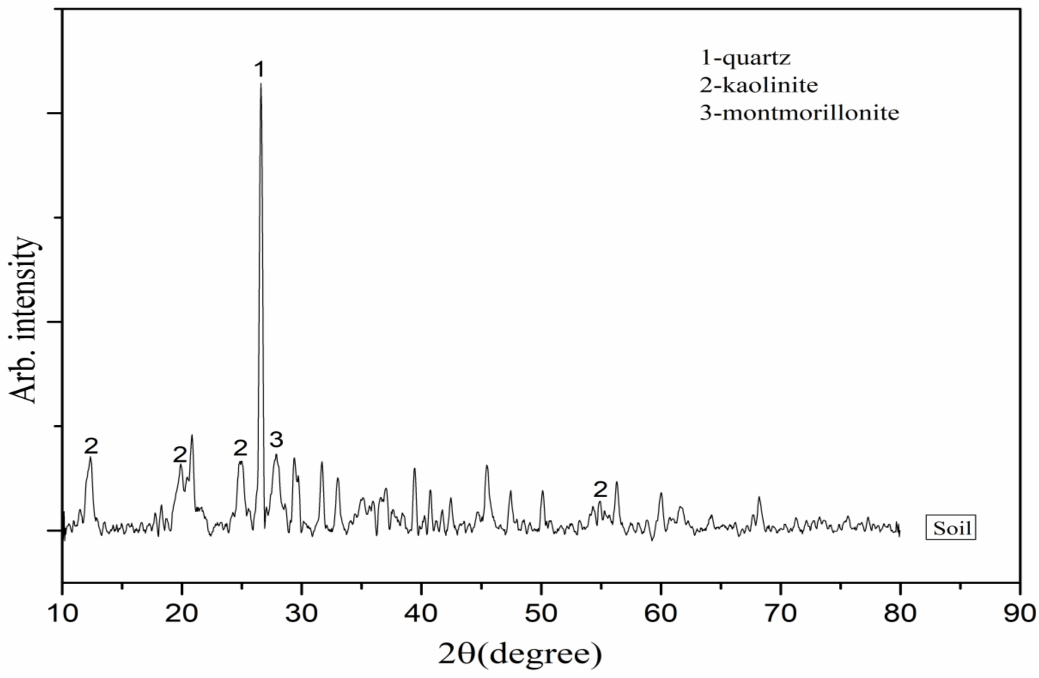


Fig. 4.10 XRD diffractogram of untreated marine clay

Figure 4.11 shows the XRD graphs of quarry dust replacing the marine clay in various percentages (30%, 35% and 40%), which were analyzed in its powder form. The three diffractograms in the figure are almost similar. This is because the additive (quarry dust) generates a new peak corresponds to Anorthite which is an in-organic rock mineral. Comparing the diffractograms of untreated soil and quarry dust treated soil, new peaks corresponding to the mineral of quarry dust is witnessed. As quarry dust is inert, no chemical reaction is observed in the treatment (Nayak and Sarvade 2012).

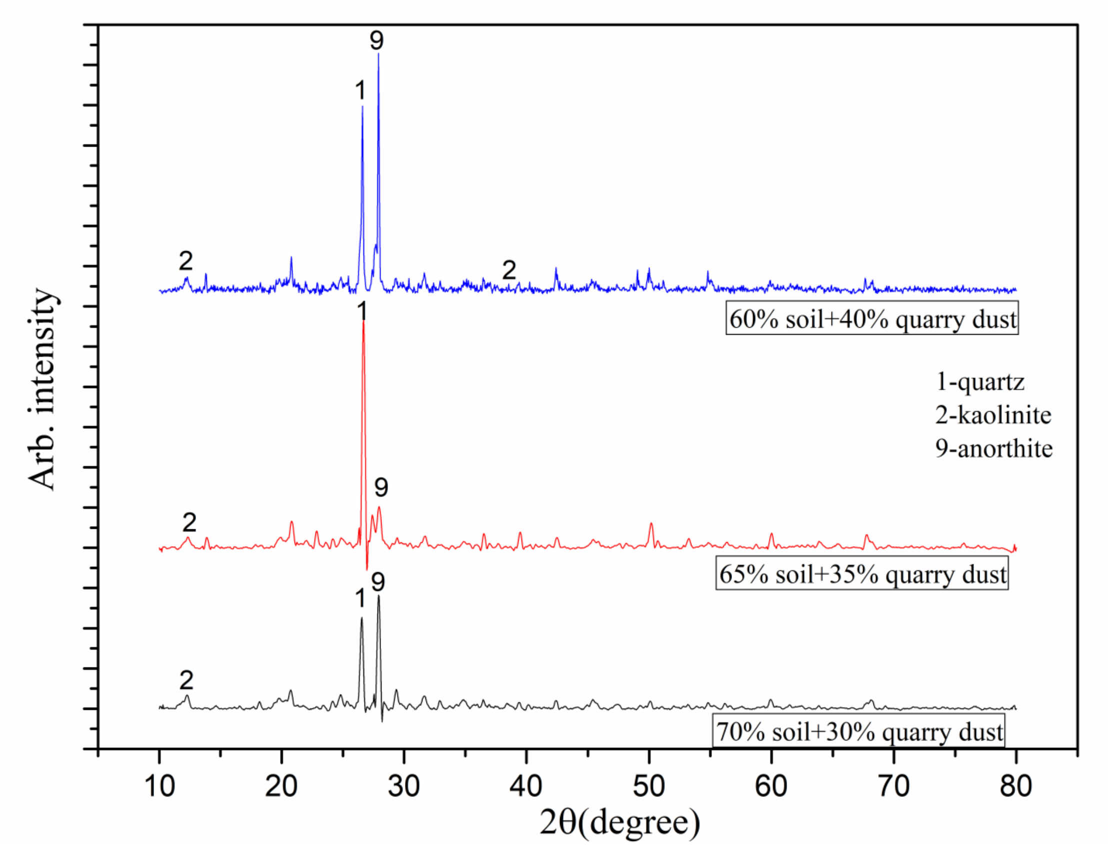


Fig. 4.11 XRD diffractograms of quarry dust replacing the marine clay

Figure 4.12 is the XRD graph of cement added to the optimum quarry dust-marine clay mix which is cured for 28 days. Peaks corresponds to new cementing compounds are generated due to the cement treatment involving the hydration reaction. Hence the new peaks are produced. The analysis of the peaks thus generated corresponds to the formation of reaction products, such as, CSH, CAH, CASH and other cementing compounds in their mineral form. Thus XRD graphs justify the improved shear strength characteristics through the chemical stabilization process.

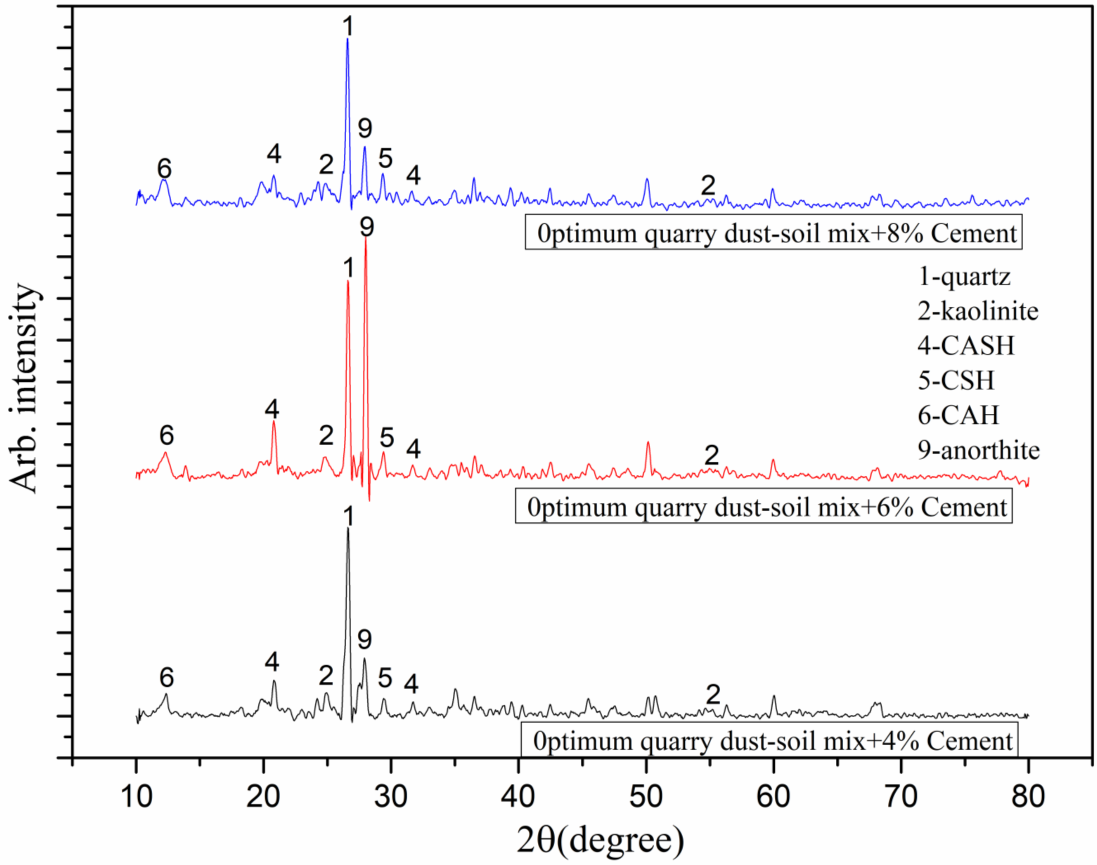


Fig. 4.12 XRD diffractograms of cement addition to optimum quarry dust-soil mix

4.4.7 Scanning electron microscope analysis

To analyze the microstructure of soil particles, Scanning Electron Microscope (SEM) is employed. The study utilizes the emission of secondary electrons from the surface of interest (soil) and appears as a three-dimensional image. Through the SEM emitted electrons, the surface topography, shape, size and texture of the soil and stabilized soil mix can be worked out. The sample was dried and coated with a thin layer of gold for the surface conduction and then kept in the SEM instrument. The SEM micrographs of marine clay stabilized with granular additives and cement were captured at a magnification capacity of 2000 and a scale bar of 10 μm. Through the micrographs obtained from SEM, the new compounds were studied.

Figure 4.13 shows the image of untreated marine clay. The figure clearly shows the clay platelets with more interparticle void space (Joseph et al. 2018). The clay platelets are dispersed in the soil mass. These platelets when comes in contact with water creates an electrical double layer. The double-layer causes pore water pressure, which is responsible for the lower MDU attainment in the compaction process (Mathai et al. 2007). With the granular additive introduction (quarry dust) to marine clay, the pore pressure decreases and makes the mass permeable. This leads to the attainment of comparably better MDU achievement (Soosan et al. 2005).

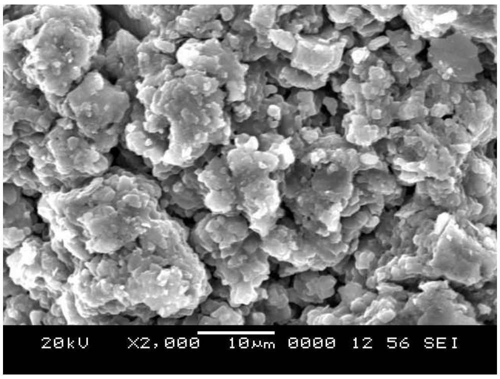


Fig. 4.13 SEM of marine clay

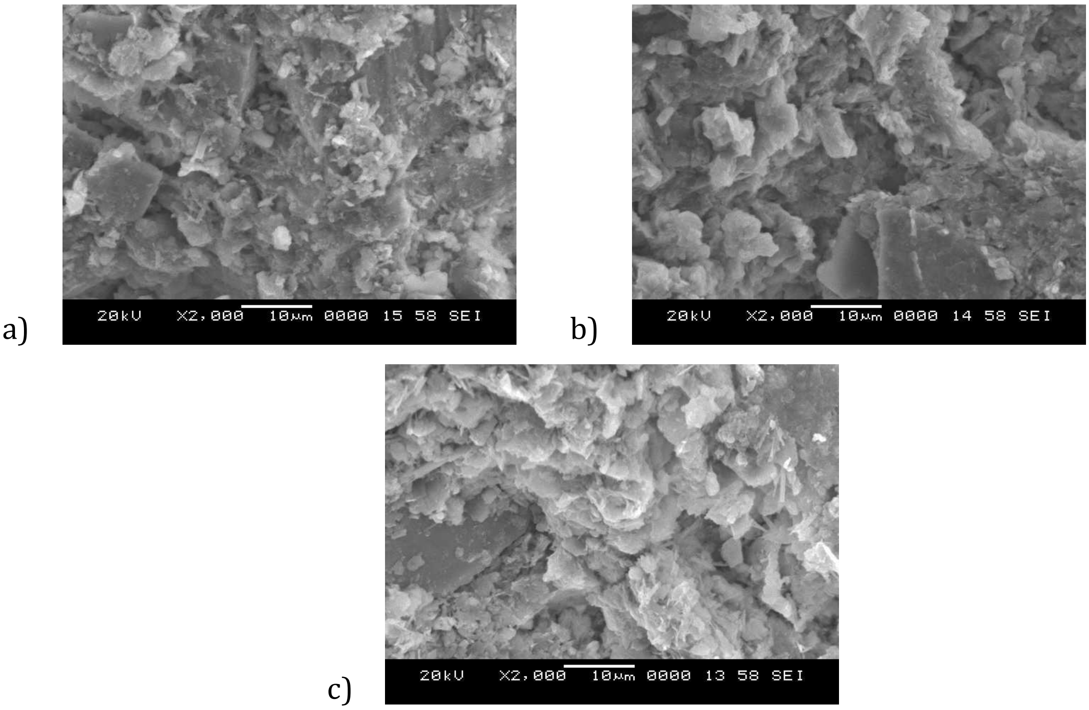
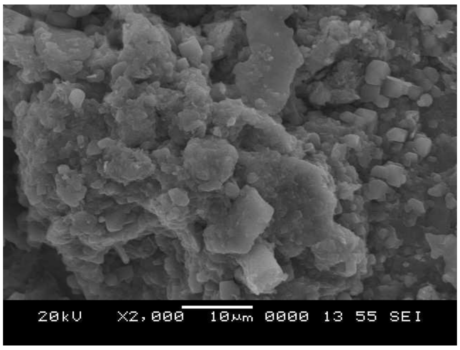
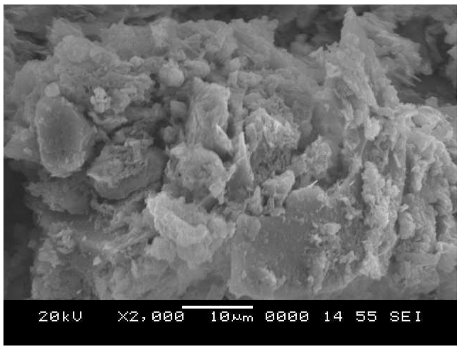


Fig. 4.14 SEM of (a) 30% (b) 35% (c) 40% quarry dust replacing the marine clay

Figure 4.14 represents the SEM micrographs of marine clay stabilized using quarry dust. From the micrographs, it can be clearly seen that the clay particles have adhered to the bulk particles (quarry dust). This leads to a reduction in pore pressure and an easy way of draining water. Thus higher MDU achievement is observed. Better compaction characteristics result in higher mechanical strength and higher friction angle of the mix. The clay particles are attached to quarry dust, but these particles have not bonded to quarry dust.

a)


b)


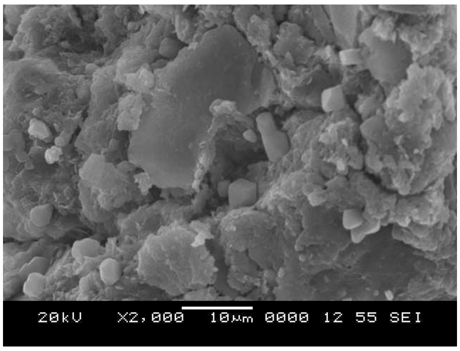
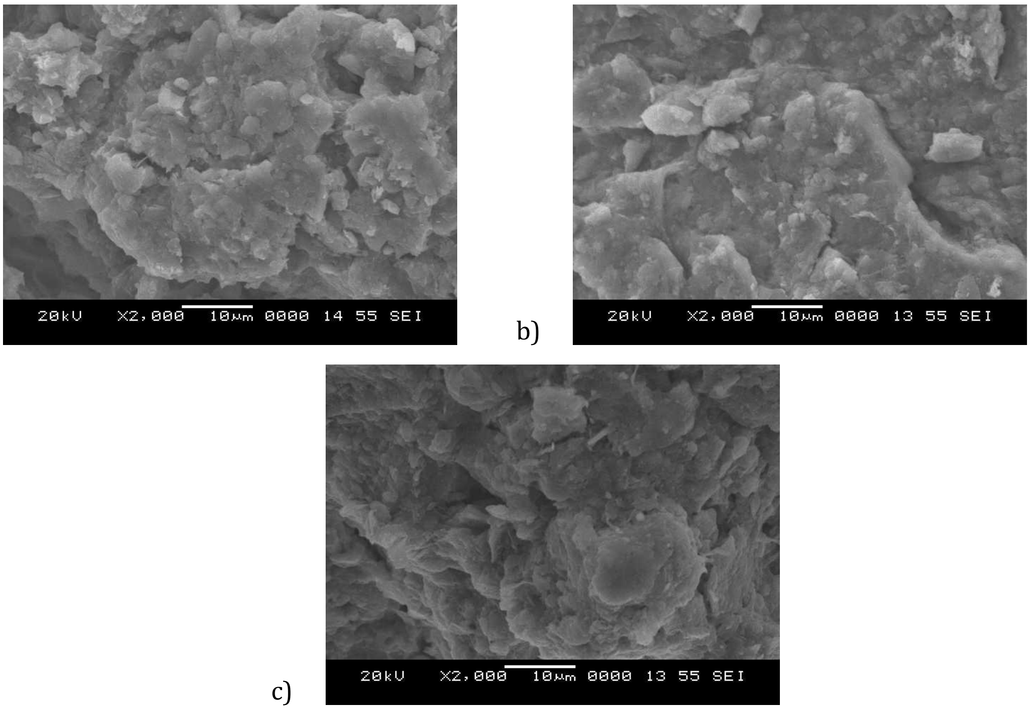
c)


Fig. 4.15 SEM of (a) 4% (b) 6% (c) 8% cement addition to optimum quarry dust replacing the marine clay cured for 7 days

Cement added to optimum quarry dust stabilized soil mix can improve the shear strength of the mix. The improvement is primarily through cohesion enhancement. Cement coats and binds the constituent particles of the mix and increases the cohesion of the mixture. Figure 4.15 shows the SEM images of cement addition to optimum quarry dust replacing the marine clay and cured for 7 days. The figure indicates the close packing of the constituent matrix and evolving as a giant particle.

The effectiveness of cement addition can be achieved through curing. Hence SEM image of 28 days cured sample shows a high rigid composite leading to a more dense state. Comparing Figure 4.16 with Figure 4.13, a clear understanding of marine clay stabilization using quarry dust and cement can be observed. The discrete plate structure of clay particles (marine clay) has turned into a rigid matrix (Nayak and Sarvade 2012).



a) b) c)

Fig. 4.16 SEM of (a) 4% (b) 6% (c) 8% cement addition to optimum quarry dust replacing the marine clay cured for 28 days

4.5 Summary

The addition of quarry dust to fine grained marine clay has altered the plasticity index, dry density and shear strength parameters in a positive way. The improvement of frictional angle is observed with addition of quarry dust to marine clay. Further improvement in cohesion and durability property is evidenced upon addition of cement to optimum quarry dust treated marine clay. The improvement was also witnessed through the SEM and XRD studies.

CHAPTER 5

STABILIZATION OF MARINE CLAY USING GRANULATED BLAST FURNACE SLAG AND CEMENT

5.1. General

Marine clay was replaced by granular slag in different proportions from 10% to 50% and investigated for its various geotechnical properties. From the laboratory results, the optimum dosage of slag replacing the soil is established. For the optimum slag-soil mix, cement is added in various proportions (2%-10%). The results obtained are analyzed and discussed in the present chapter.

5.2 Properties of Granulated Blast Furnace Slag (GBFS)

The laboratory tests were conducted on GBFS and the results obtained are tabulated in Table 5.1

Table 5.1 Properties of GBFS

Sl. No.	Properties	Particulars
1	Specific gravity	2.29
	Grain size distribution	
	Coarse sand size (%)	6
2	Medium sand size (%)	76
	Fine sand size (%)	17
	Clay and silt (%)	1
	Strength parameters	
3	Cohesion (kPa)	2
	Angle of internal friction ϕ (degrees)	43
4	pH	9.5
5	Calcium oxide (%)	41
6	SiO_2 (%)	35.3
7	Fe_2O_3 (%)	0.17
8	Al_2O_3 (%)	13.5
9	MgO (%)	7.2
10	Loss on ignition (%)	0.66

The grain size distribution curve of GBFS is shown in Figure 4.1. It can be noticed that GBFS has predominantly medium sand-sized particles. The maximum and minimum dry densities of GBFS are 5.86 kN/m^3 and 8.52 kN/m^3.The direct shear test results showed the frictional angle of 43° and cohesion intercept of 2 kN/m^{2}for its 95% relative density. The pH value of GBFS is 9.5, proving to be alkaline. The calcium oxide (CaO) content in GBFS is 41%; hence GBFS can be concluded as a moderately active material. GBFS are lighter (G=2.29) compared to marine clay (G=2.6).

5.3 Effect of GBFS on geotechnical properties of marine clay

5.3.1 Effect of GBFS on index properties of marine clay

The summary of the index properties of the soil–slag mixture is shown in Table 5.2.

Table 5.2 Index properties of marine clay stabilized using GBFS

Sl. No.	Properties	Marine Clay	Percentage of GBFS replacing soil				
			10%	20%	30%	40%	50%
1	Liquid limit w_L (%)	91	79	72	67	63	61
2	Plastic limit w_P (%)	33	31	30	28	27	27
3	Plasticity Index I_P(%)	58	48	42	39	36	34
4	Specific gravity (G)	2.6	2.56	2.54	2.5	2.47	2.43
5	MDU (kN/m^3)	13.6	14.4	14.7	14.9	15	14.9
6	OMC (%)	27	24.6	24	22.6	22.2	21.8

Results depict that with the higher addition of GBFS to marine clay, the specific gravity of the mixture decreases. The decrease is primarily due to lighter slag with a specific gravity of 2.29 added to the comparatively heavier soil with a specific gravity of 2.6. Increased percentage of GBFS to the soil, reduces the fines percentage of mixture. Hence, the specific surface area of the mix decreases, leading, to the drop in the values of liquid limit and plasticity index (Figure 5.1). With 40% and 50% GBFS replacing the marine clay, plasticity index reduced by 38% and 41% respectively, compared with untreated marine clay indicating an improved geotechnical property of the mix (C. Sekhar et al. 2017).

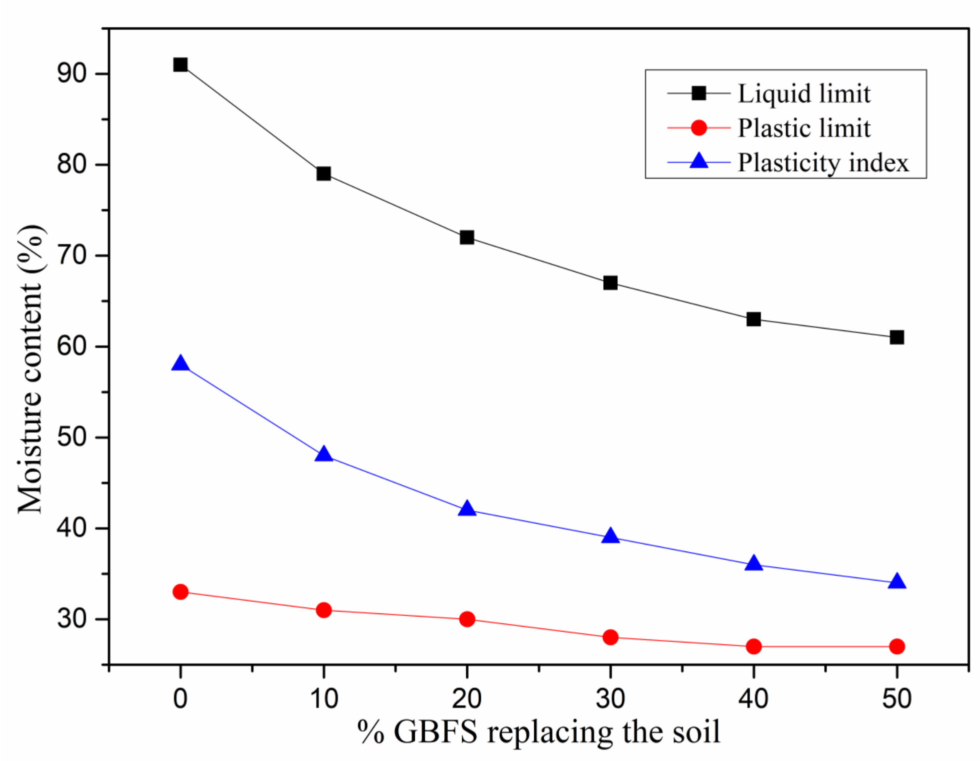


Fig. 5.1 Variation in Atterberg's limits for different proportions of GBFS replacing the marine clay

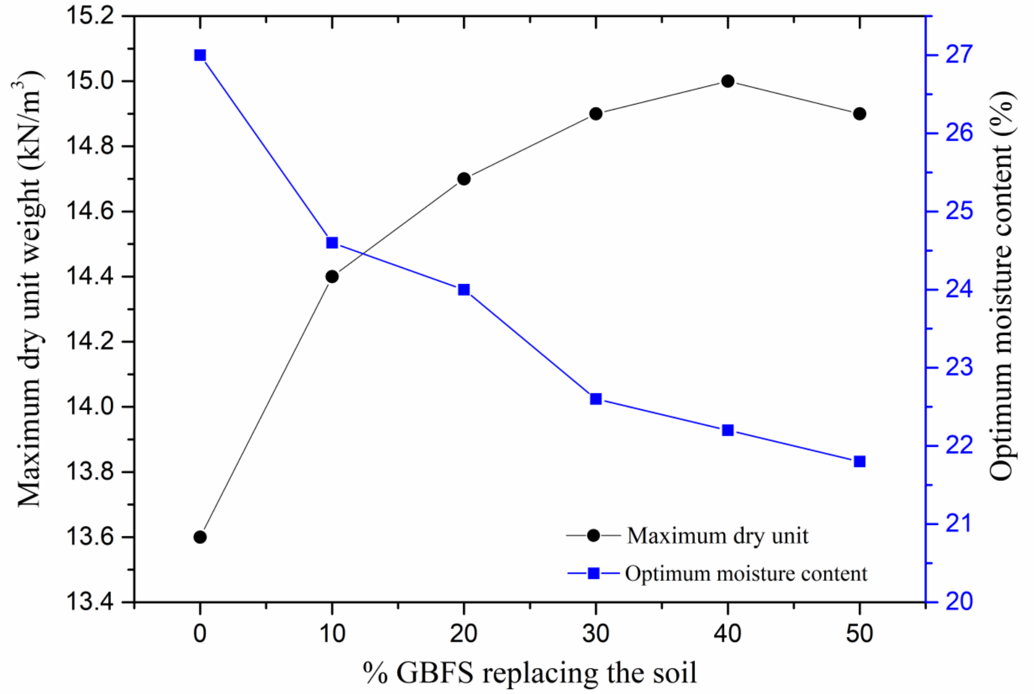


Fig. 5.2 Variation in MDU and OMC for different proportions of GBFS replacing the marine clay

5.3.2 Effect of GBFS on compaction characteristics of marine clay

It is observed from Table 5.2that till 40% replacement, the MDU has increased despite the addition of lighter slag. The fine and plate structure of clay dominated in marine clay attracts the water and holds in forms of the diffused double layer. In the compaction process, the part of energy imparted by the blows is absorbed by water in the voids resulting in lesser density attainment of marine clay. With the addition of granular slag (non-plastic), the energy shared by solid particles increases; by this the soil–slag mixture gets well compacted, thus achieving more density. Further, for 50% GBFS replacing the soil, MDU decreases (Figure 5.2) because of the dominance of lighter slag to the soil. There is a decrease in OMC with GBFS addition because GBFS being non-plastic and sand size particle, water required to coat the cumulative surfaces in the mixture becomes less.

5.3.3 Effect of GBFS on UCS of marine clay

Figure 5.3 displays the unconfined compression strength (UCS) of untreated marine clay and different soil–slag mixtures cast at their OMC-MDU condition and cured for 0, 7 and 28 days. It is noticed that the UCS of mixtures increases marginally till 30% GBFS replacing the soil when tested immediately after sampling (Table 5.3). For further replacement, the strength decreases due to the dominance of coarser non-plastic slag, causing a lack of cohesion and confinement when tested immediately after casting. The increased strength is because of the rough texture and annular particles of granular slag particles imparting good friction between the slag and soil particles. With 7 days of curing, the UCS enhanced significantly till 40% replacement. This is because of the hydration reaction between the free lime in slag and the soil in the presence of water. There is the formation of CSH with the higher age of curing; thus, strength increases for all mixtures with curing (Amulya et al. 2018). The 40% GBFS replacing the soil gave maximum strength compared to all other blends upon curing, and for 28 days of curing, the increase in strength was five times compared to untreated soil. Thus, 40% GBFS replacing the soil is concluded as the optimized GBFS-soil mix.

Table 5.3 Variation of UCS with different percentage of GBFS replacing the soil

% GBFS Replacing Marine clay	UCS (kPa) with duration of curing		
	0 day	7 day	28 day
0	98	98	98
10	117	219	282
20	125	267	470
30	147	335	514
40	138	377	585
50	132	316	518

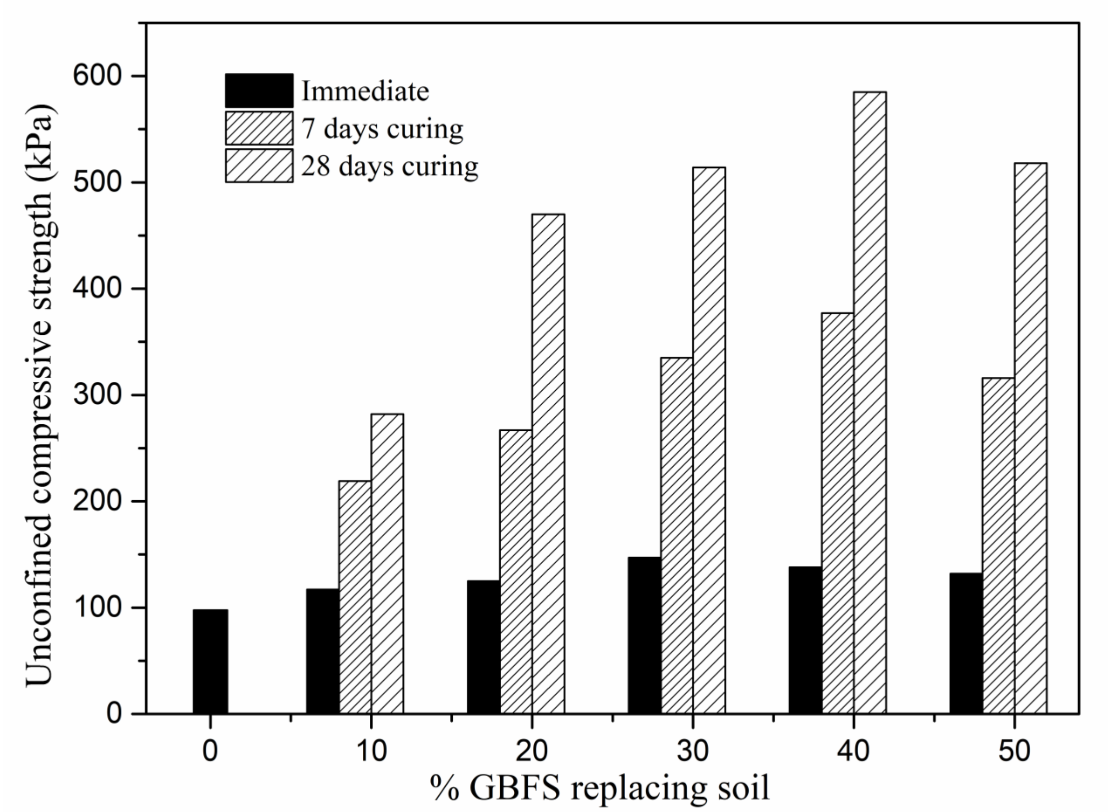


Fig. 5.3 Variation in UCS of marine clay and GBFS blend with curing age

5.3.4 Effect of GBFS on shear strength parameters of marine clay

The unconsolidated undrained (UU) triaxial compression test under fully saturation condition was carried on soil and soil–slag mixtures cast for their MDU and OMC which was cured for 0 days, 7 days and 28 days. It can be noted that the test performed immediately after casting showed a decline in cohesion upon supplementing the non-plastic granular slag to the soil.

But the frictional angle increases upon the addition of GBFS and also with curing (Fig. 5.4). This is due to its rough texture and better interlocking. The cohesion intercept enhanced upon curing (7 days and 28 days), which is shown in Figure 4. The cohesion is maximum for 40% GBFS replacing the marine clay. The cohesion increases from 19 kPa (untreated soil) to 48 kPa for 7 days and 66 kPa for 28 days cured specimens (Fig.5.5). This is because of the pozzolanic reaction between the free lime in slag and soluble silica of the soil. CSH gel is formed by this hydration (C. Sekhar and Nayak 2017). With further increase in replacement of soil by GBFS (50%), the cohesion decreased due to the unavailability of reactive silica for the reaction in the mix. Results are shown in Table 5.4

Table 5.4 Shear strength parameters of marine clay stabilized using GBFS

Curing age	Strength properties obtained from triaxial UU test	Marine Clay	Percentage GBFS replacing soil				
			10%	20%	30%	40%	50%
0 days	Cohesion c_{UU} (kPa)	19	18	17	15.5	14.5	13
	Angle of internal friction ϕ_{UU} (degrees)	14	18	18.5	19	23	24.5
7 days	Cohesion c_{UU} (kPa)	19	22	29	37	48	36
	Angle of internal friction ϕ_{UU}(degrees)	14	16.5	23.5	36	25	26
28 days	Cohesion c_{UU} (kPa)	19	32	41	55	66	46
	Angle of internal friction ϕ_{UU} (degrees)	14	17	19.5	22	25	27

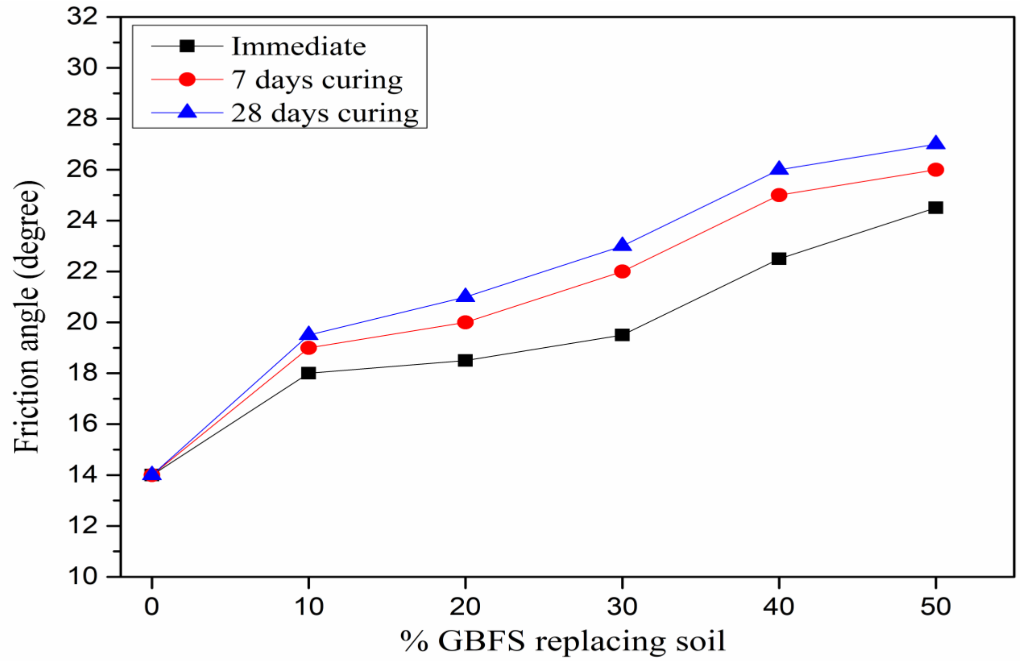


Fig. 5.4 Variation in frictional angle for different proportions of GBFS replacing the marine clay

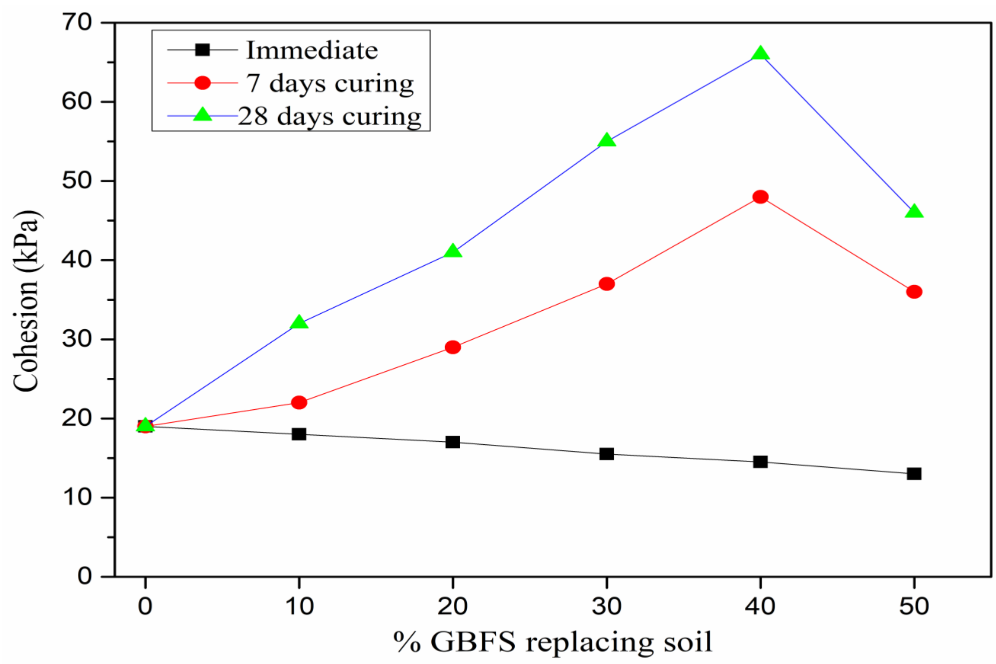


Fig. 5.5 Variation in cohesion for different proportions of GBFS replacing the marine clay

5.3.5 The effect of cement addition to optimum GBFS-marine clay mixture on the geotechnical properties of marine clay

To the optimum GBFS replacing the marine clay (40%), cement is added in various percentages (2%, 4%, 6%, 8%, and 10%) of its dry weight. The addition of cement activates the pozzolanic reaction. The UCS increases from 585 kPa (optimum mix) to 802 kPa for a 28-day cured sample with the addition of only 2% cement to the optimum GBFS replacing the marine clay. i.e., 37% increase in strength compared with UCS of the optimum mix. Similarly, 68%, 104%, 160% and 213%increase in unconfined compressive strength is observed with the addition of 4%, 6%, 8% and 10% cement to optimum mix (Table 5.6). The individual sum of 7-day cured UCS of optimum slag-soil (60% marine clay + 40% GBFS) and soil with various percentage of cement (2%, 4%, 6%, 8%, and 10%) is less than the 7-day cured UCS for the particular combination of cement percentage added to the optimum mix. This is due to the enhanced reactivity of GBFS with soil in the presence of an activator (cement). The combined effect (soil–slag–cement) is more than the sum of individual effect (soil–slag and soil–cement) for 28-day cured specimens. A similar effect is reflected in the shear strength parameters. Cohesion increased tremendously with the increased addition of cement to optimum GBFS-soil mix. For addition of 2%, 4%,6%, 8% and 10% cement to optimum mix, there is 39%, 88%,145%, 180% and 233% increase in cohesion, and frictional angle enhances by 20%, 36%, 60%, 84% and 108% for the respective addition of cement when compared with optimum GBFS-soil mix for 28 days of curing (Table 5.6). This improvement is due to the cracking up of coating and dissolution of gel surrounding the particle. This further increases reactivity of lime in the system and thus the formation of CASH products. The reactivity of slag also depends on the *pH* of the system. With the addition of cement, the *pH* elevates, facilitating/activating the hydration reaction. Due to this, shear strength parameters have improved (C. Sekhar and Nayak 2018).

Table 5.5 UCS and shear strength parameters of marine clay stabilized with optimum GBFS and cement

Curing age	Strength properties	Marine Clay	60% Marine clay + 40% Granulated blast furnace slag + varying % of cement addition					
			0%	2%	4%	6%	8%	10%
0 days	UCS (kPa)	98	138	139	152	169	183	190
	Cohesion c_{UU} (kPa)	19	14.5	14.5	16	18	20	22.5
	Angle of internal friction ϕ_{UU} (degrees)	14	23	23	24	24	26	27.5
7 days	UCS (kPa)	98	377	497	643	912	1194	1452
	Cohesion c_{UU} (kPa)	19	48	64	106	134	160	195
	Angle of internal friction ϕ_{UU} (degrees)	14	25	30	34	39	45	53
28 days	UCS (kPa)	98	585	802	983	1198	1519	1829
	Cohesion c_{UU} (kPa)	19	66	92	124	162	185	220
	Angle of internal friction ϕ_{UU} (degrees)	14	25	30	34	40	46	52

California Bearing Ratio test on Marine clay and GBFS treated marine clay (with/without cement) specimens cured for 28 days and soaked for 96 hours was performed. From the test results (Table 5.6), it can be noted that CBR value increased with the increase in addition of GBFS. GBFS addition performed better than quarry dust addition to marine clay (Table 4.9). The optimum GBFS-soil mix showed an increased CBR of 6.42 times compared to the marine clay specimen prepared at its MDU and OMC. Also, the value tremendously increased upon the addition of cement to the optimum GBFS-soil mix.

Both marine clay and marine clay treated by GBFS samples failed in durability tests. These samples crumbled upon immersion due to insufficient bonding between the particles. When cement is added to the optimum GBFS-soil mix, the samples performed well. 2% cement addition showed a R_i value of 0.88 and thus meet the criteria (IRC-89). R_i index of 10% cement addition to the optimum mix is 1.04. This increase in strength upon soaking is due to the improved pozzolanic reaction upon immersion in water. Also, the improvements in failure envelope and stress-strain behaviour are shown in Figs. 5.6 and 5.7.

From the alternate wetting and drying test results, it can be inferred that cement added optimum GBFS-soil mix is durable. The maximum mass loss observed was 5.6% which is less than the upper limit (14%) as per IRC-89. Higher the cement content lower is the mass loss and strength loss on soaking (Table 5.7). This is because of the better bonding between the constituent particles of the mix (Yadu and Tripathi 2013).

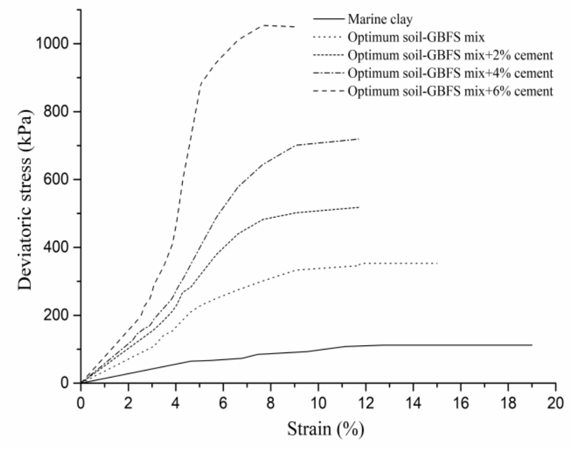


Fig. 5.6 Stress strain graph for different proportions of cement to Opt. GBFS-soil mix

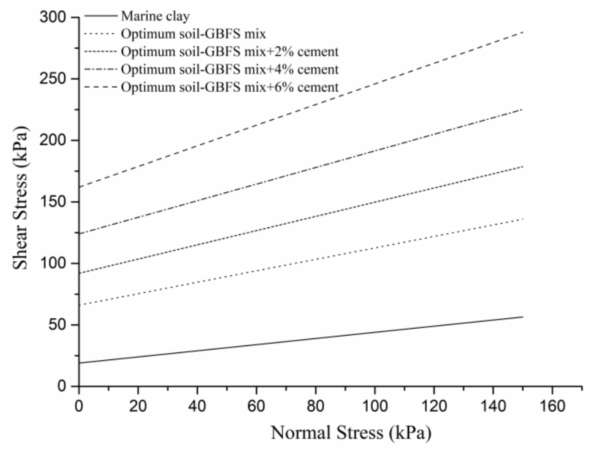


Fig. 5.7 Failure envelope for different proportions of cement to Opt. GBFS-soil mix

Table 5.6 CBR value of marine clay stabilized with GBFS and cement

Property	Marine clay	Percentage GBFS replacing the soil					Percentage cement added to optimum GBFS-soil mix		
		10%	20%	30%	40%	50%	2%	4%	6%
CBR (%)	2.6	4.3	9.6	14	19.3	22.1	42	67	89

Table 5.7 Durability performance of marine clay stabilized with opt. GBFS & cement

Sl. No	Durability properties		60% Marine clay + 40% GBFS + varying % of cement addition				
			2%	4%	6%	8%	10%
1	R_i(%)		88	95.9	97.3	96.7	104
2	Loss of mass on alternate wetting (W) and drying (D) (%)	W	4.1	3.6	1.7	1.5	1.4
		D	5.6	4.5	3.1	2.3	2.1

5.3.6 X-Ray diffraction analysis

The mineralogical changes that occurred due to the introduction of cement and slag to soil originate new crystalline compounds in the mix. The data obtained from the diffractometer are analyzed and presented in Figs. 5.8 and 5.9. With the introduction of slag to the soil, new peaks are found when compared with diffractogram of untreated soil. The peak indicates the occurrence of new compounds due to the initiation of the pozzolanic reaction. The reaction generates new minerals that are responsible for strength improvements. The change in the peak and relative intensity proves the formation of minerals originated by the reaction between the soil and the free lime in slag.

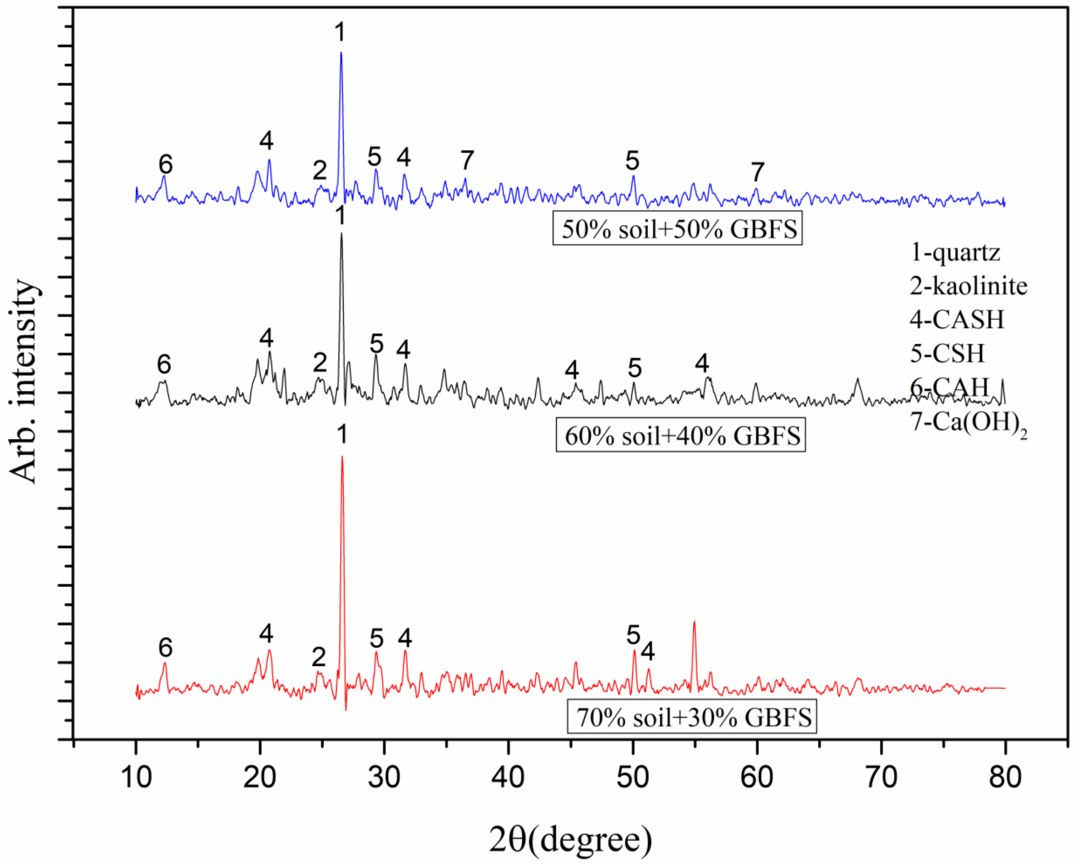


Fig. 5.8 XRD diffractograms of GBFS replacing the marine clay

Figure 5.8 shows the XRD graphs of GBFS replacing the marine clay in various percentages (30%, 40% and 50%) which are cured for 28 days and analyzed in their powder form. From the graph, it is clear that the calcium oxide in the GBFS has initiated the hydration reaction in the presence of water with silica of soil. The reaction product is similar to the product obtained due to cement hydration. The development of peaks in the graph emphasizes the corresponding formation of cementitious products. They are CSH, CASH, CAH and other cementitious products upon curing.

The improvement in geotechnical properties such as UCS and shear strength parameters with the incorporation of GBFS to marine clay is due to the formation of the above reaction products. By comparing Figure 5.8 with Figure 4.10, it is also clear that the relative peaks are better for optimum GBFS than the rest of the mix. This is also the reason for the drop of UCS for 50% GBFS replacement compared to the optimum mix. Further, the improvement in the peak formation was observed due to the addition of cement to the optimum GBFS and soil, which is exhibited in Figure5.9. Cement as a binding agent also acted as an activator to GBFS-soil hydration reaction (C Sekhar and Nayak 2017). The peaks representing the reaction products, such as CSH, CASH, and CAH have relatively improved upon increased addition of cement to optimum GBFS-soil mix.

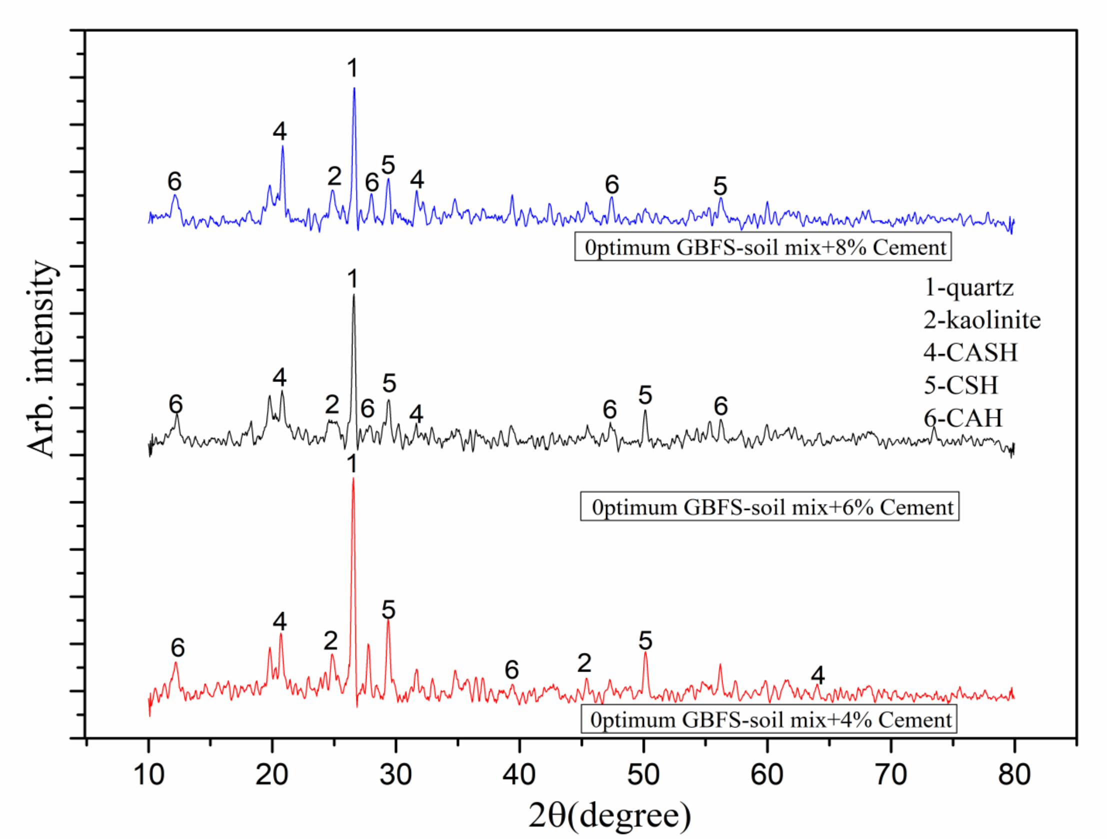


Fig. 5.9 XRD diffractograms of cement addition to the optimum slag-soil mixture

5.3.7 Scanning electron microscopic analysis

Through the micrographs obtained from SEM, the new compounds are studied. From Figure 4.13, it is noticed that the interparticle voids are more with the dominance of dispersed structure of untreated soil. Figure 5.10 indicates the annular particles of GBFS having a glassy and rough texture. This physical property resulted in the improvement of the angle of internal friction of GBFS-marine clay mix.

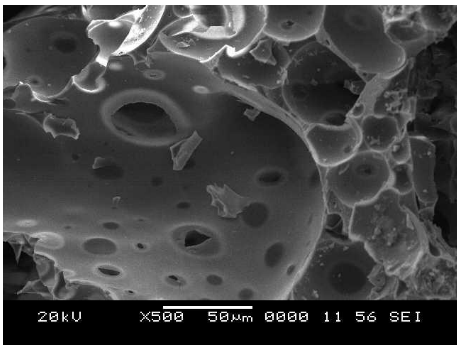
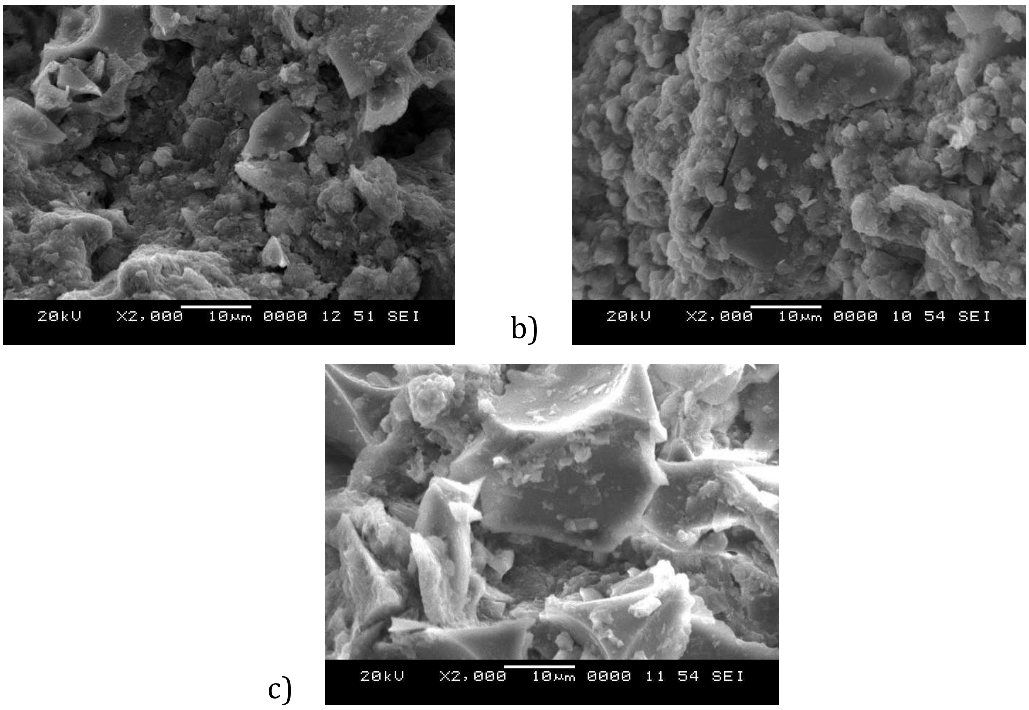


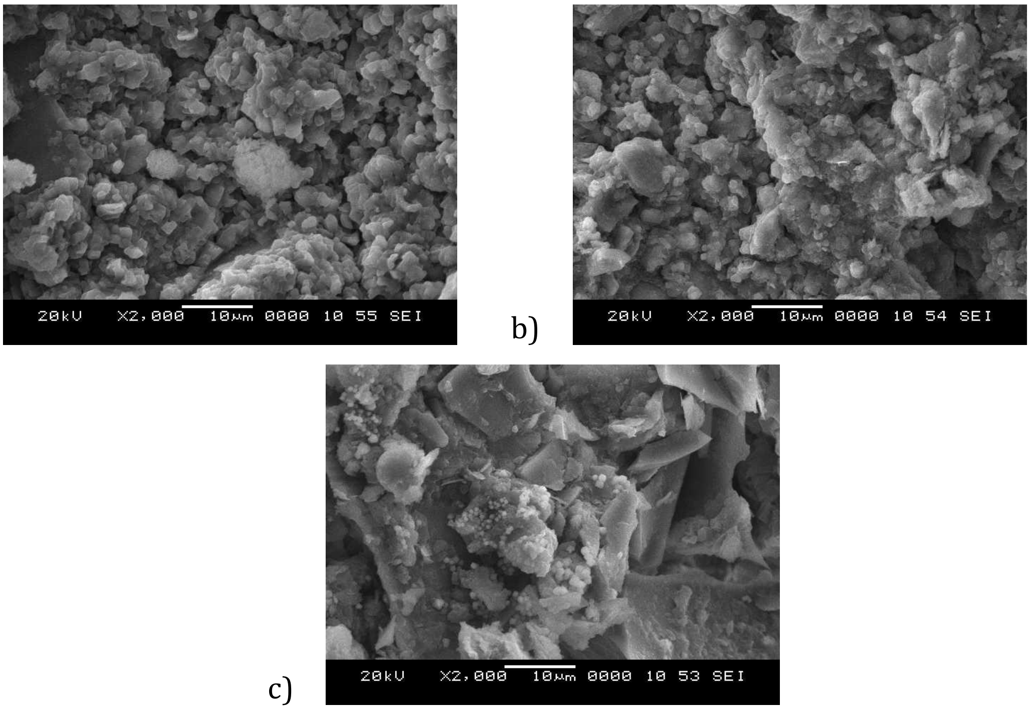
Fig. 5.10. SEM of granulated blast furnace slag.

Figure 5.11a–c shows the SEM images of 30%, 40% and 50% replacement of soil by GBFS cured for 7 days, respectively. In these figures, the soil particles turned to a flocculent structure from the dispersed structure in its untreated state. With the higher percentage replacement (i.e., 50%), the dominance of GBFS can be seen; as the mixture contains the flaky granular and annular particles (Figure 5.11 c). Figure 5.12(a–c) shows the dense matrix upon curing for 28 days for the 30%, 40% and 50% replacement mixtures. This dense matrix is due to the coating of hydration reaction products. The agglomeration of particles with significant bonding is due to the occurrence of hydration of free lime in GBFS which justifies the enhanced cohesion of soil–slag mix (Figure 5.5). The patchy blurred mass adhered in the interface and interparticle space is the cementitious gels (CASH), which are responsible for the binding of particles. The formation of cementitious products results in the decrease of voids/air spaces. This phenomenon occurs due to the coating and binding of individual particles of composite with cementitious gels resulting in the decrease in migration of ions into the pore space, making a rigid structure of soil slag composite.



a) b) c)

Fig. 5.11. SEM of (a) 30% (b) 40% (c) 50% GBFS replacing the marine clay cured for 7 days



a) b) c)

Fig. 5.12. SEM of (a) 30% (b) 40% (c) 50% GBFS replacing the marine clay cured for 28 days

Further, with the addition of cement to GBFS, the dense structure turns out to more rigid with fewer pores and more packing. For 6% cement addition (Figure5.13b) and8% cement addition (Figure 5.13c) to the optimum mix, SEM images show the formation of a needle-like structure called ettringite, which is responsible for strength attainment at the initial stage (Basha et al. 2005). This ettringite dissolves in the medium upon curing. Figure 5.14 (a–c) shows the SEM micrographs of 4%, 6% and 8% cement addition to optimum soil–slag mix cured for 28 days. From the images, the formation of the cementitious gel is noticed. This gel is responsible for the development and coating of dense mass binding the composite particles and occupying the void space leading to well-compacted dense rigid intactparticles. This is the reason for the tremendous increase in cohesion, thus the strength of the stabilized mix upon curing (Guda 2016).

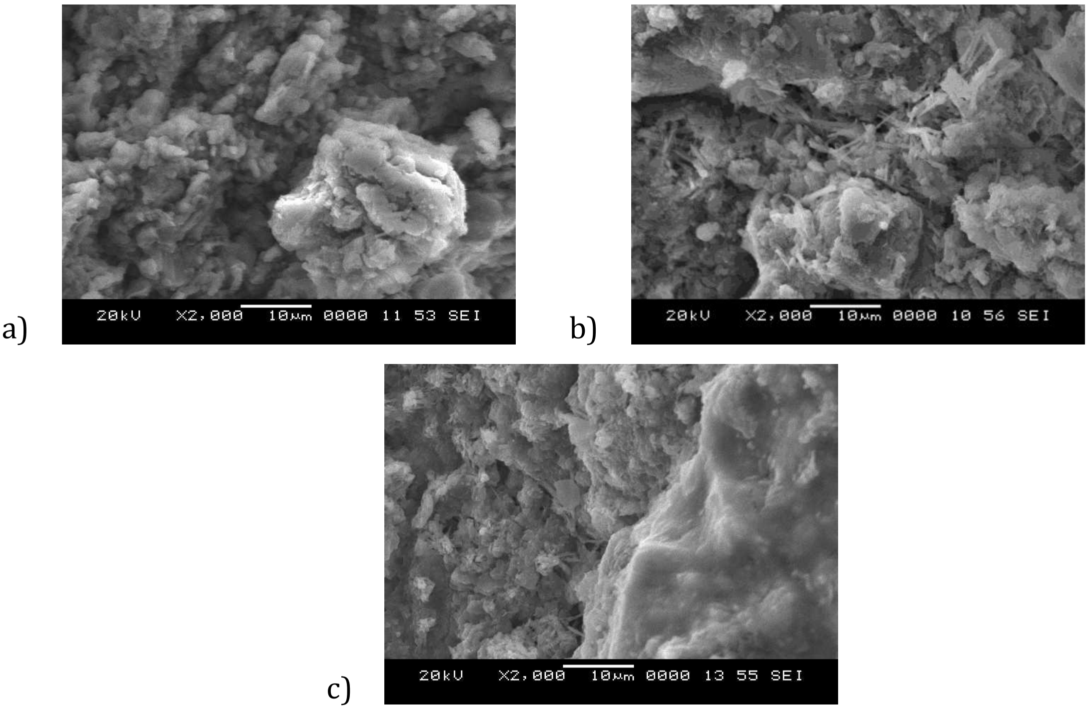


Fig. 5.13. SEM of (a) 4% (b) 6% (c) 8% cement addition to optimum GBFS replacing the marine clay cured for 7 days

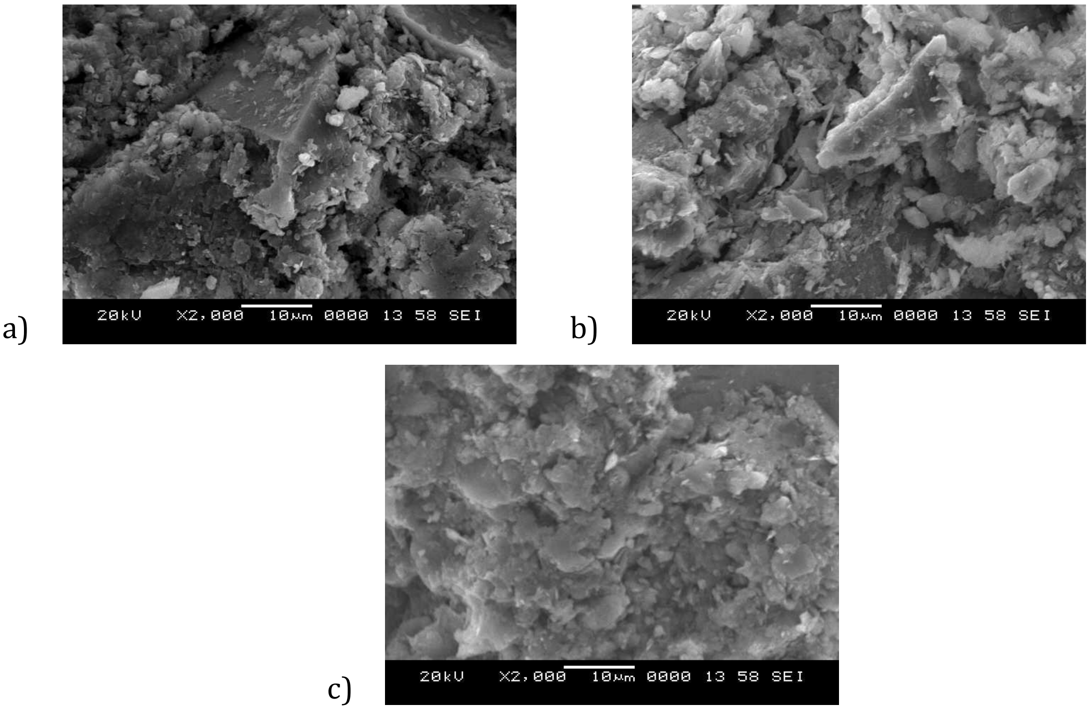


Fig. 5.14. SEM of (a) 4% (b) 6% (c) 8% cement addition to optimum GBFS replacing the marine clay cured for 28 days

5.4 Summary

The addition of GBFS to marine clay has improved the plasticity behaviour, dry density and shear strength parameters. Upon curing, the unconfined compressive strength of GBFS treated marine clay has increased. This shows that incorporation of GBFS to soil involves pozzolanic reaction. The improvement of frictional angle and cohesion is observed with addition of GBFS to marine clay. Further tremendous increment in cohesion and durability property was evidenced upon addition of cement to optimum GBFS treated marine clay. The improvement was also witnessed through the SEM and XRD studies.

CHAPTER 6

ANALYSIS OF STRIP FOOTING RESTING ON STABILIZED MARINE CLAY USING PLAXIS 2D

6.1 General

PLAXIS is a software-based on finite element approach that covers the analysis of almost all the applications of geotechnical engineering. PLAXIS simulates using a user-friendly interface with the concepts of finite element computations. PLAXIS is programmed for the study of stresses and deformations of 2-Dimensional and 3-Dimensional soil structures.

PLAXIS 2D is a tool developed to analyze stresses, deformation, stability and flow of groundwater in the geotechnical application using a two-dimensional finite element approach. The creation of the model involves a simple input of laboratory results followed by the generation of complex finite element models. The run of the program thus generates an enhanced output providing a presentation of detailed computational results.

Analysis of geotechnical problem involves an advanced constitutive model, and the simulation may require a time-dependent, non-linear, anisotropic complex behavior of rocks/soils. Rigorous procedures may be adopted in dealing with the advanced problems of hydrostatic/non-hydrostatic pore pressure, as these come under a multiphase material modeling. PLAXIS is well equipped with options to deal with a wide range of complex geotechnical structures (Mosallanezhad and Moayedi 2017).

6.2 Overview of the analysis

Laboratory test results obtained from the geotechnical tests performed on marine clay and marine clay stabilized mixes are introduced to run a typical strip footing problem using PLAXIS 2D. The analysis results in obtaining load settlement computation of strip footing resting on marine clay and marine clay stabilized mixes.

The results obtained from the PLAXIS program on load settlement analysis of strip footing is discussed in this chapter.

The concept behind the finite element analysis is that, a complex body or a structure is discretised into smaller elements. Nodes connect these elements. The stress and the displacement at each node are obtained by solving the boundary conditions. The real situation can be simulated in PLAXIS 2D by modeling the program by either an axisymmetric model or a plane strain model. To carry out this modeling, the user has to first create a geometry comprising of points and lines on an X-Y plane in PLAXIS 2D. Further, the model should be assigned with the material property and respective boundary conditions. This can be achieved through soil and structure options in PLAXIS 2D. The calculation stage can proceed once the user defines the geometry model. Mesh generation is initiated and the construction stage is defined. Construction stage involves the excavation of the soil cluster by activating/deactivating the cluster. After the excavation phase, simulation of the construction phase is followed. Later, a series of sequential finite element calculations will be performed by the software. This calculation depends on the complexity of the model. Displacements and the stresses are the primary output quantities of finite element analysis. Structural forces are also calculated if the model involves structural elements (C. Sekhar 2017).

To estimate the load settlement response of strip footing of various footing widths, PLAXIS 2D can be used. A series of load settlement behavior were worked out on the different 2-Dimensional model and presented in the subsequent sections. The analysis involves the load settlement response of strip footing resting on marine clay, quarry dust stabilized marine clay with or without cement, and granulated blast furnace slag treated marine clay with or without cement. The above combinations were run to note the settlement of the strip footing for various load intensities and for different widths of strip footing using PLAXIS 2D software package.

Table 6.1 Properties of marine clay, quarry dust stabilized marine clay and quarry dust + cement stabilized marine clay

Properties	Unit weight (kN/m^3)	Moisture Content (%)	Modulus of Elasticity E (MPa)	Cohesion, c (kN/m^2)	Angle of internal friction $\phi°$	Coefficient of permeability k (m/day)	Poisson's ratio μ
Marine Clay	10.67	65	0.97	16	13	0.00096	0.39
Opt. Quarry dust-Marine clay	16.7	18	2.9	15.5	25	0.031968	0.36
Opt. Quarry dust-Marine clay + 2% Cement	17	18	4.13	49	30	0.0008	0.33
Opt. Quarry dust-Marine clay + 4% Cement	17.3	18	4.82	102	33	0.00004814	0.32
Opt. Quarry dust-Marine clay + 6% Cement	17.5	18	6.33	142	37	0.0000181	0.31

Table 6.2 Properties of marine clay, granulated blast furnace slag stabilized marine clay and granulated blast furnace slag + cement stabilized marine clay

Properties	Unit weight (kN/m^3)	Moisture Content (%)	Modulus of Elasticity E (MPa)	Cohesion, c (kN/m^2)	Angle of internal friction $\phi°$	Coefficient of permeability k (m/day)	Poisson's ratio μ
Marine Clay	10.67	65	0.97	16	13	0.00096	0.39
Opt. GBFS-Marine clay	15	22.2	3.1	66	25	0.00204	0.35
Opt. GBFS-Marine clay + 2% Cement	15	22.2	4.72	92	30	0.000542	0.31
Opt. GBFS-Marine clay + 4% Cement	15.4	22.3	5.37	124	34	0.0000304	0.3
Opt. GBFS-Marine clay + 6% Cement	15.7	22.2	7.03	162	40	0.0000165	0.3

Properties of strip footing:

Unit weight (γ) : 25 kN/m^3

Modulus of Elasticity (E) : 22 x 10^{6}kPa

Poisson's ratio (μ) : 0.15

Details of Strip footing:

Depth of strip footing placed : 1.2 m below GL

Width of strip footing : 1 m, 1.5 m and 2 m

Thickness of concrete strip footing : 0.6 m

The properties of strip footing and its details are mentioned above.

The input properties of Marine clay and the stabilized Marine clay combination (Optimum QD + various percentages of cement and Optimum GBFS + various percentages of cement) are presented in Table 6.1 and Table 6.2.

6.2.1 Steps involved for the analyses of strip footing resting on marine clay and stabilized marine clay using PLAXIS 2D

Step 1: In the start window of PLAXIS 2D, the project title is designated and details of the project is given. The geometry of the model should to be defined. For the present analysis, the geometry of -4B depth and 8B width was fixed (where B is the width of the strip footing). The geometry is fixed such that no considerable change in settlement of the strip footing is observed beyond this size.

Step 2: For defining the soil properties, a borehole is created in the model, and soil properties are assigned. The properties include unit weight of soil, coefficient of permeability, modulus of elasticity, Poisson's ratio and shear strength parameters (c and ϕ). A borehole is placed in the model to define the soil properties like unit weight, permeability, modulus of elasticity and shear strength parameters (c and ϕ).

Step 3: Concrete strip footing of thickness 0.6 m is placed at a depth of -1.2 m. Concrete properties are assigned to the strip footing..

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

Step 5: 2D medium size meshes are generated.

Step 6 (Calculation Phase): Soil, concrete properties and applied load intensity were activated as defined below:

> Phase 1: In Excavation stage, by default, above the footing level (depth: -1.2m) natural soil layer will be activated. Hence this natural soil above the footing level is deactivated by turning it off.
>
> Phase 2: The concrete strip footing is then activated. A load intensity of 10 kPa is applied by activating the load intensity.

Step 7: Calculations are initiated.

Step 8: The maximum settlement value is shown in the figure displayed in the Output tab. The footing settlement can be obtained either from the figure or from the settlement contour or from the table in the output tab.

Step 9: Step six is repeated with increasing the applied load intensity (50, 100, 150, 200, 250, 300, 350, 400 kPa) and the corresponding settlement is noted.

6.2.2 Cases analyzed using PLAXIS 2D

In the present analysis, three different cases have been run using PLAXIS 2D software. Each case is executed by varying the width of the foundation for each admixture dosage.

Case 1 corresponds to the strip footing resting on natural soil. Case 1 is analysed for various width of the footing (1m, 1.5m and 2m) and for different load intensity (10k Pa, 50 kPa, 100 kPa, 150 kPa, 200 kPa, 300 kPa, 350 kPa and 400 kPa).

Case 2 corresponds to strip footing resting on a stabilized zone of 1B depth and 2B width below the foundation ('B' is the width of the footing).

Similarly, case 3 corresponds to strip footing resting on a stabilized zone of 2B depth and 3B width below the foundation. In all the above cases, the footing widths (1m, 1.5m and 2m) and loading intensities (10, 50, 100…..400kPa) are varied and analyzed for the load intensity- settlement behaviour.

Case 1: Footing resting on marine clay (Figure 6.1)

Case 2: Footing resting on stabilized soil zone of depth 1B and width 2B (Figure 6.2)

Case 3: Footing resting on stabilized soil zone of depth 2B and width 3B (Figure 6.3)

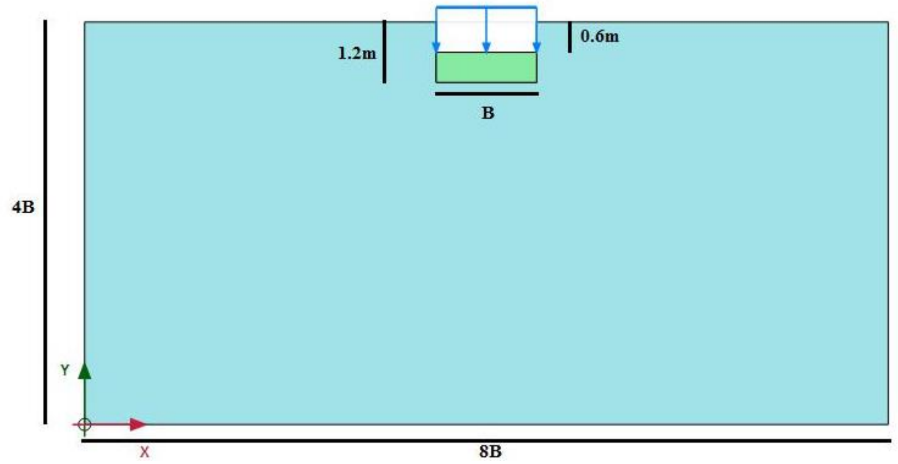


Fig. 6.1 Strip footing resting on natural marine clay (Case I)

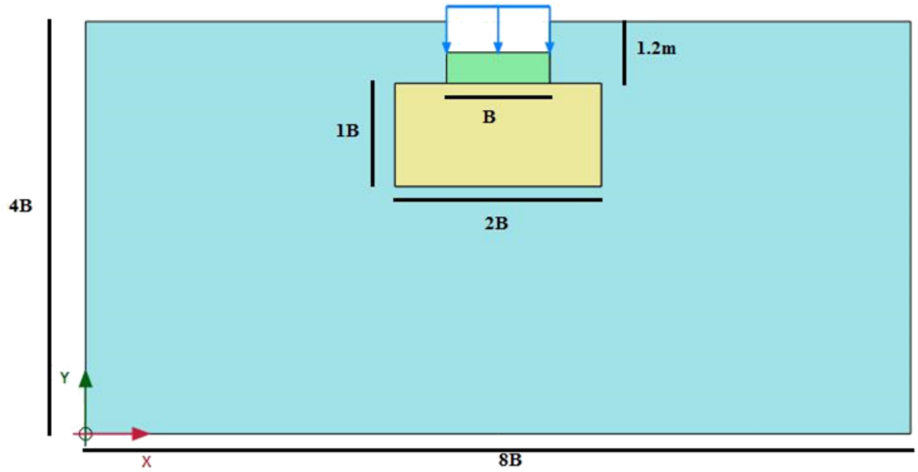


Fig. 6.2 Strip footing resting on stabilized zone of depth 1B and width 2B (Case II)

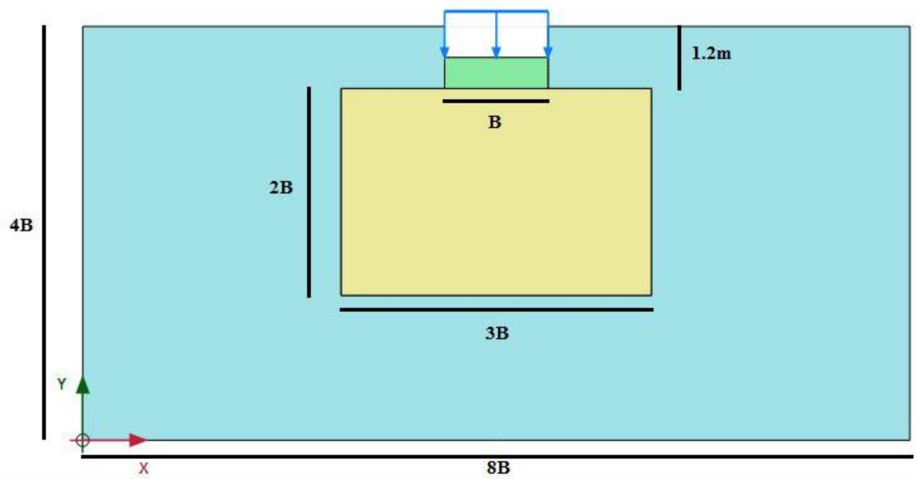


Fig. 6.3 Strip footing resting on stabilized zone of depth 2B and width 3B (Case III)

6.3 Settlement of strip footing

6.3.1 Settlement of strip footing resting on marine clay (Case I)

The results of the analysis using PLAXIS 2D for the strip footing of width 1m, 1.5m and 2m resting on natural soil (marine clay) are presented in Table 6.3 and Figure 6.4. The settlement increases with the increase in applied pressure on the footing (Figure 6.4). Further, with the increase in footing width, the settlement increases. For example, the settlement corresponds to load intensity of 100 kPa is 59.7 mm and for the same applied pressure, the settlement increases to 156 mm on increasing the width of the strip footing from 1m to 2m. The corresponding increase in the settlement is 161%. Similarly, for the same strip foundation width, say B=1.5m, the settlement increases from 44.47 mm to 103.6 mm upon increasing the load intensity from 50 kPA to 100 kPa.

Table 6.3 Settlement of strip footing of different widths resting on untreated soil

Load intensity (kPa)	Settlement (mm)		
	1m wide strip footing	1.5m wide strip footing	2m wide strip footing
10	7.14	11.43	15.27
50	26.13	44.47	62.3
100	59.7	103.6	156
150	188.8	334.7	523.7

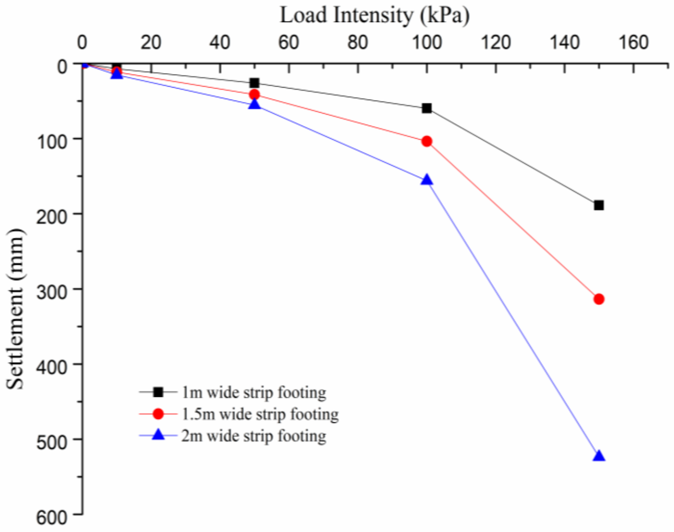


Fig. 6.4 Load intensity-settlement curve of strip footing resting on untreated soil for various width of foundation

6.3.2 Settlement of strip footing resting on stabilized soil zone of depth 1B and width 2B (Case 2)

The stabilized zone of soil corresponds to optimum quarry dust-soil, optimum quarry dust-soil+2% cement, optimum quarry dust-soil+4% cement and optimum quarry dust-soil+6% cement mixtures in case of quarry dust stabilized blends. Similarly for GBFS, optimum GBFS-soil, optimum GBFS-soil+2% cement, optimum GBFS-soil+4% cement and optimum GBFS-soil+6% cement mixtures were considered as stabilized soil mixtures in the analysis. The results from the analysis are tabulated in Table 6.4 (B=1m), Table 6.5 (B=1.5m) and Table 6.6 (B=2m) and the corresponding graphical presentation is shown in Figure 6.5, Figure 6.6 and Figure 6.7 for quarry dust treated stabilized zone. Similarly, for slag treated stabilized zone, the results are tabulated in Table 6.7 (B=1m), Table 6.8 (B=1.5m) and Table 6.9 (B=2m) and the corresponding graphical presentation is made in Figure 6.8, Figure 6.9 and Figure 6.10.

The Tables and figures conclude that the increase in load intensities and footing width will increase the settlement of the strip footing. Upon stabilization using quarry dust/GBFS with or without cement, the settlement decreases. For an applied load intensity of 100 kPa and footing width of 1.5m, the settlement of footing resting on untreated marine clay is 103.6 mm, which reduces to 98.7 mm and 60.92 mm upon soil treated by optimum quarry dust and optimum GBFS, respectively. Percentage reduction in the settlement is 5% and 41% for soil replaced by optimum quarry dust and optimum GBFS. Further the settlement reduces with the dosage of cement to the optimum granular-soil mix. Similar trends can be seen for all applied load intensities and for all blends. From the analysis, it is clear that GBFS treated stabilized zone have performed better compared to quarry dust treated stabilized zone by exhibiting lower settlement values. The reason behind this is the improved shear strength parameters and higher young’s modulus of GBFS treated marine clay compared to quarry dust treated marine clay.

Table 6.4 Settlement of strip footing of width B=1m resting on stabilized soil zone of depth 1B and width 2B considering various soil-quarry dust blends

Load intensity (kPa)	Settlement (mm)				
	Untreated Marine clay	Optimum QD-soil	Optimum QD-soil + 2% cement	Optimum QD-soil + 4% cement	Optimum QD-soil + 6% cement
10	7.14	7.05	6.408	6.149	5.757
50	26.13	20.47	18.41	17.63	16.43
100	59.7	51.06	34.59	33.08	30.08
150	188.8	157.4	53.49	51.49	48.26
200	--	--	73.01	68.78	64.8
250	--	--	124	92.9	86.8
300	--	--	297.7	128.4	123.3
350	--	--	--	213.5	201.6

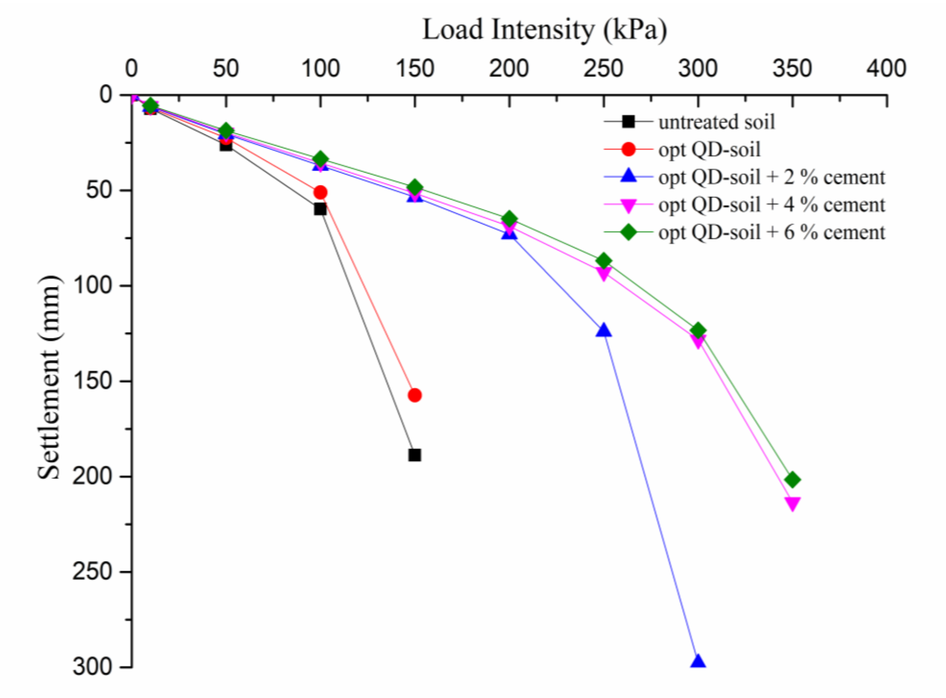


Fig. 6.5 Load intensity-settlement of B=1m wide strip footing resting on stabilized soil of depth 1B and width 2B for various soil-QD-cement mixtures

Table 6.5 Settlement of strip footing of width B=1.5m resting on stabilized soil zone of depth 1B and width 2B considering various soil-quarry dust blends

Load intensity (kPa)	Settlement (mm)				
	Untreated Marine clay	Optimum QD-soil	Optimum QD-soil + 2% cement	Optimum QD-soil + 4% cement	Optimum QD-soil + 6% cement
10	11.43	10.93	9.417	8.329	7.948
50	44.47	28.7	24.16	23.83	22.74
100	103.6	98.71	65.59	63.19	59.43
150	334.7	296.2	94.27	90.7	85.36
200	--	--	133.6	127	119.2
250	--	--	241.1	184.1	173.9
300	--	--	--	260.7	248.3
350	--	--	--	420.1	393.7

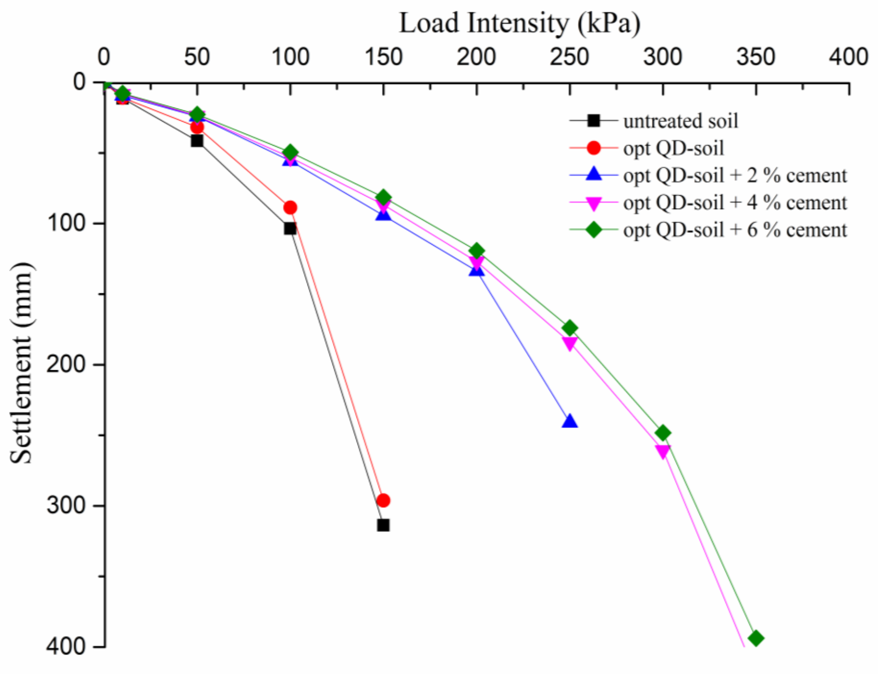


Fig. 6.6 Load intensity-settlement of B=1.5m wide strip footing resting on stabilized soil of depth 1B and width 2B for various soil-QD-cement mixtures

Table 6.6 Settlement of strip footing of width B=2m resting on stabilized soil zone of depth 1B and width 2B considering various soil-quarry dust blends

Load intensity (kPa)	Settlement (mm)				
	Untreated Marine clay	Optimum QD-soil	Optimum QD-soil + 2% cement	Optimum QD-soil + 4% cement	Optimum QD-soil + 6% cement
10	15.27	13.86	12.93	10.51	9.92
50	62.3	43.27	34.87	31.92	28.96
100	156	144.4	94.56	91.11	85.7
150	523.7	468	135.2	129.6	122.1
200	--	--	197.3	185.3	173.6
250	--	--	372	271.9	256.9
300	--	--	--	379.4	361.2
350	--	--	--	539	515.4

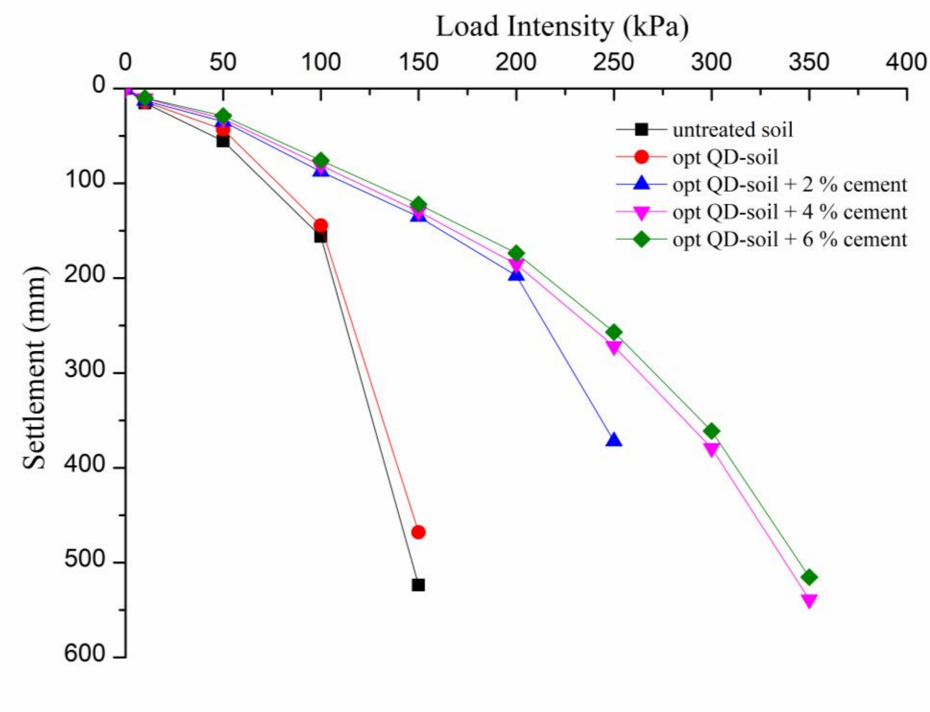


Fig. 6.7 Load intensity-settlement of B=2m wide strip footing resting on stabilized soil of depth 1B and width 2B for various soil-QD-cement mixtures

Table 6.7 Settlement of strip footing of width B=1m resting on stabilized soil zone of depth 1B and width 2B considering various soil-GBFS blends

Load intensity (kPa)	Settlement (mm)				
	Untreated Marine clay	Optimum GBFS-soil	Optimum GBFS-soil + 2% cement	Optimum GBFS-soil + 4% cement	Optimum GBFS-soil + 6% cement
10	7.14	6.093	5.545	5.442	5.148
50	26.13	19.19	17.05	16.54	15.48
100	59.7	36.77	32.62	31.54	29.31
150	188.8	56.27	51.07	49.59	46.52
200	--	79.58	68.18	66.64	62.47
250	--	110	92.48	88.52	84.23
300	--	169.5	127.6	123.7	119.4
350	--	--	207.1	198.1	188.9

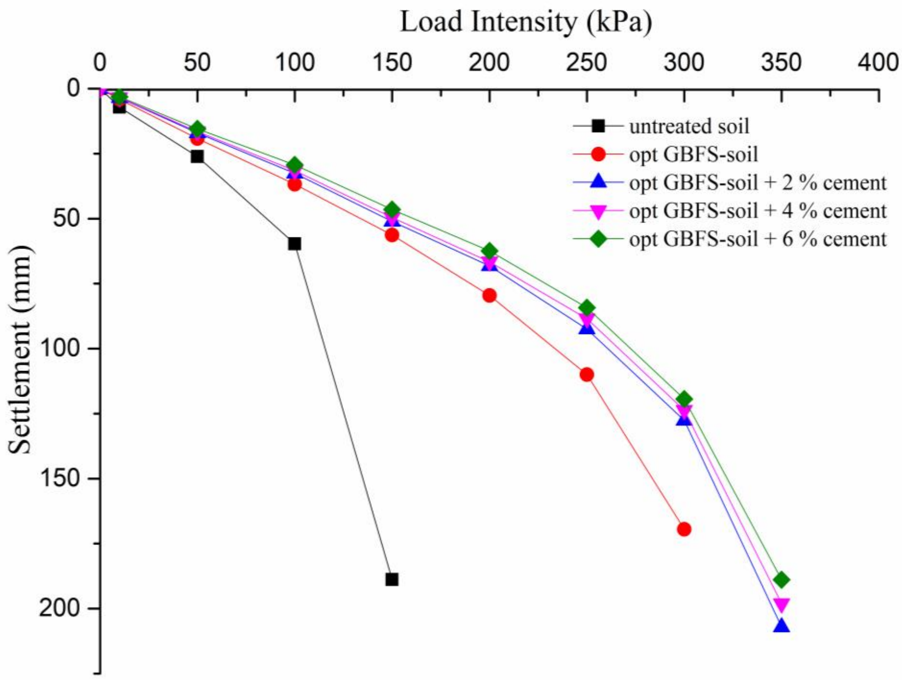


Fig. 6.8 Load intensity-settlement of B=1m wide strip footing resting on stabilized soil of depth 1B and width 2B for various soil-GBFS-cement mixtures

Table 6.8 Settlement of strip footing of width B=1.5m resting on stabilized soil zone of depth 1B and width 2B considering various soil-GBFS blends

Load intensity (kPa)	Settlement (mm)				
	Untreated Marine clay	Optimum GBFS-soil	Optimum GBFS-soil + 2% cement	Optimum GBFS-soil + 4% cement	Optimum GBFS-soil + 6% cement
10	11.43	9.69	7.87	6.95	6.56
50	44.47	24.11	20.47	19.87	18.51
100	103.6	60.92	57.90	56.49	53.39
150	334.7	94.2	85.48	83.31	78.53
200	--	134	117.2	113.9	107
250	--	190.8	169.1	164.9	156.1
300	--	--	239.6	234.3	223.5
350	--	--	357.1	347.6	334.42

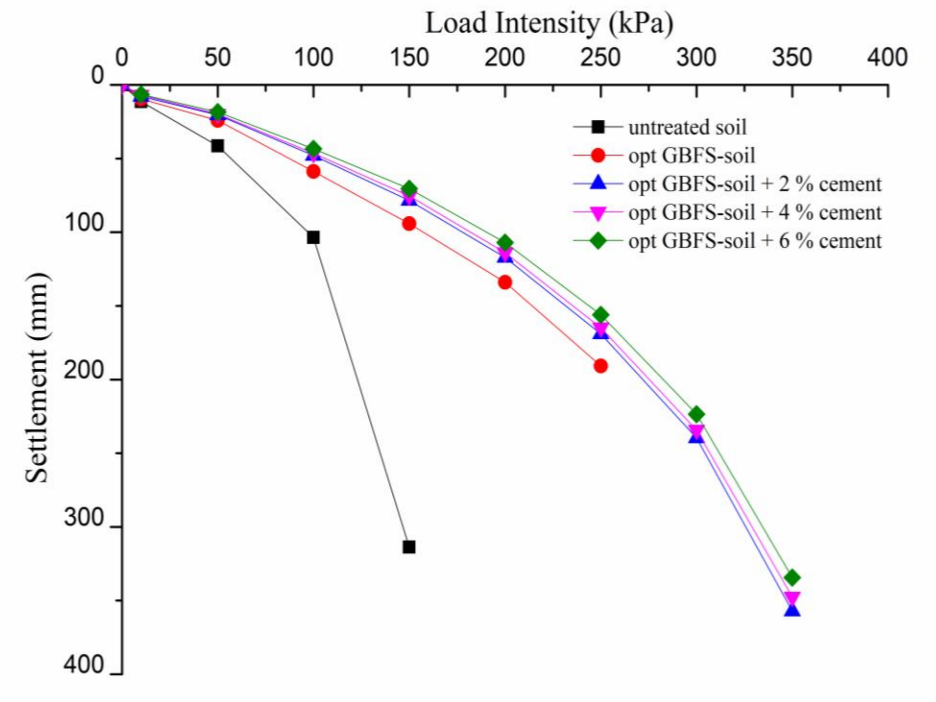


Fig. 6.9 Load intensity-settlement of B=1.5m wide strip footing resting on stabilized soil of depth 1B and width 2B for various soil-GBFS-cement mixtures

Table 6.9 Settlement of strip footing of width B=2m resting on stabilized soil zone of depth 1B and width 2B considering various soil-GBFS blends

Load intensity (kPa)	Settlement (mm)				
	Untreated Marine clay	Optimum GBFS-soil	Optimum GBFS-soil + 2% cement	Optimum GBFS-soil + 4% cement	Optimum GBFS-soil + 6% cement
10	15.27	12.04	10.79	10.7	9.99
50	62.3	34.62	27.06	25.72	24.33
100	156	96.9	87.99	85.84	81.08
150	523.7	142.9	126.6	123.4	116.6
200	--	205.2	179.6	174.3	164.4
250	--	295.3	263.6	257.4	244.3
300	--	487.8	369.9	361.6	345.9
350	--	--	521	507.6	489.1

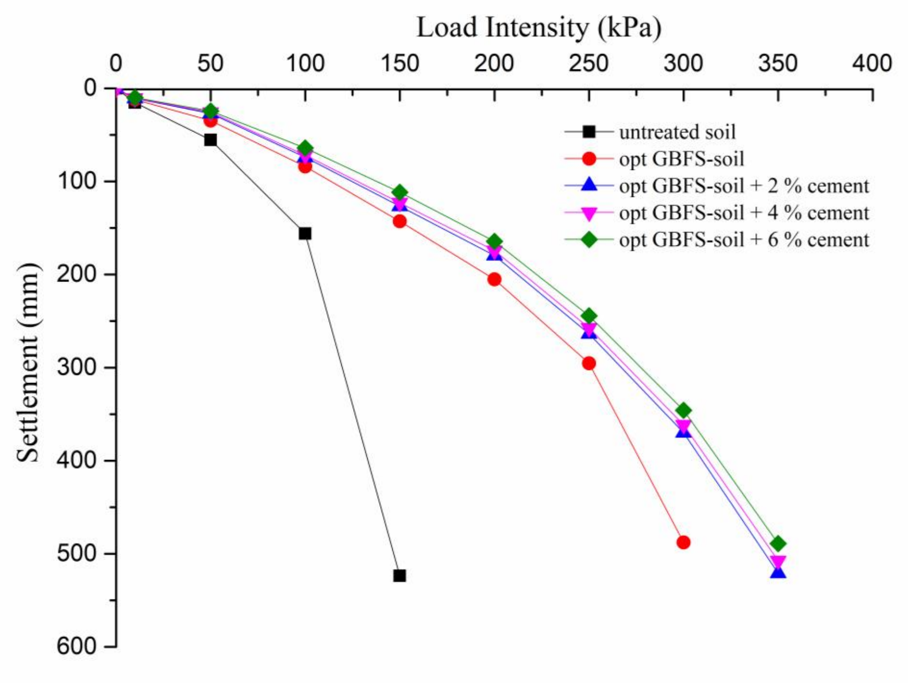


Fig. 6.10 Load intensity-settlement of B=2m wide strip footing resting on stabilized soil of depth 1B and width 2B for various soil-GBFS-cement mixtures

6.3.3 Settlement of strip footing resting on stabilized soil zone of depth 2B and width 3B (Case 3)

The settlement analysis was carried out for varying load intensity and for various width of footing resting on different stabilized soil mixtures. In the present section, the strip footing resting on a stabilized zone of depth 2B and width 3B. For these variables (width, applied load intensity and stabilized mixes), the PLAXIS modelling is performed and analyzed. The results from the analysis are tabulated in Table 6.10 (B=1m), Table 6.11 (B=1.5m) and Table 6.12 (B=2m) and the correspond graphical presentation is made in Figure 6.11, Figure 6.12 and Figure 6.13 for quarry dust treated stabilized zone. Similarly. For slag treated stabilized zone, the results are tabulated in Table 6.13 (B=1m), Table 6.14 (B=1.5m) and Table 6.15 (B=2m) and corresponding graphical presentation is made in Figure 6.14, Figure 6.15 and Figure 6.16.

The Tables and figures conclude that the increase in load intensities and footing widths increases the settlement of the strip footing. Upon stabilization using quarry dust/GBFS with or without cement, the settlement decreases. For an applied load intensity of 100 kPa and width 1.5m, the settlement of footing resting on untreated marine clay is 103.6 mm, which is reduced to 68.34 mm and 42.99 mm upon soil treated by optimum quarry dust and optimum GBFS, respectively. Percentage reduction in the settlement is 34% and 59% for ground replaced by optimum quarry dust-soil and optimum GBFS-soil respectively. With the dosage of cement to optimum soil granular mix, the settlement reduces. Further, for the same load intensity of 100 kPa, the settlement reduced by 64% and 70% when 4% cement is added to optimum quarry dust and optimum GBFS mix. Similar trends can be seen for all applied load intensities and for all blends. From the analysis, it is clear that GBFS treated stabilized zone have performed better compared to quarry dust treated stabilized zone.

It can be observed by comparing the results of Case 3 and Case 2 that, with the increase in the area of the stabilized zone, the settlement of the footing has decreased. This decrement is due to the reduction in stress on natural soil. The decrease in the stress in the soil is due to the widespread of the applied load intensity and reduction of further stress with the depth of stabilized zone.

Table 6.10 Settlement of strip footing of width B=1m resting on stabilized soil zone of depth 2B and width 3B considering various soil-quarry dust blends

Load intensity (kPa)	Settlement (mm) (QD)				
	Untreated Marine clay	Optimum QD-soil	Optimum QD-soil + 2% cement	Optimum QD-soil + 4% cement	Optimum QD-soil + 6% cement
10	7.14	6.58	5.35	4.88	4.19
50	26.13	16.32	12.94	11.69	9.85
100	59.7	35.92	23.24	21.04	17.73
150	188.8	136.1	35.67	32.49	27.66
200	--	--	46.7	42.49	36.19
250	--	--	59.58	52.82	44.45
300	--	--	77.72	64.06	53.27
350	--	--	128.6	76.14	63.03
400	--	--	--	89.25	73.34

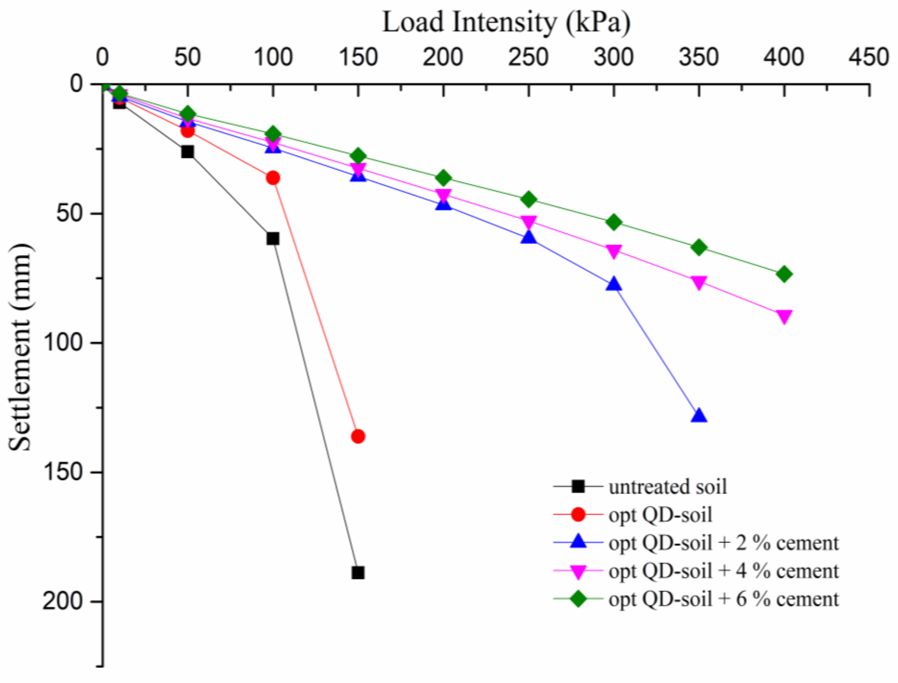


Fig. 6.11 Load intensity-settlement of B=1m wide strip footing resting on stabilized soil of depth 2B and width 3B for various soil-QD-cement mixtures

Table 6.11 Settlement of strip footing of width B=1.5m resting on stabilized soil zone of depth 2B and width 3B considering various soil-quarry dust blends

Load intensity (kPa)	Settlement (mm) (QD)				
	Untreated Marine clay	Optimum QD-soil	Optimum QD-soil + 2% cement	Optimum QD-soil + 4% cement	Optimum QD-soil + 6% cement
10	11.43	8.623	7.53	7.12	6.51
50	44.47	25.67	21.06	19.38	16.89
100	103.6	68.34	38.88	35.56	30.68
150	334.7	235.1	58.38	53.32	46.00
200	--	--	78.71	71.73	61.33
250	--	--	100.5	90.48	77.14
300	--	--	128.2	109.4	93.04
350	--	--	227.6	128.8	109.1
400	--	--	--	150.2	126.3

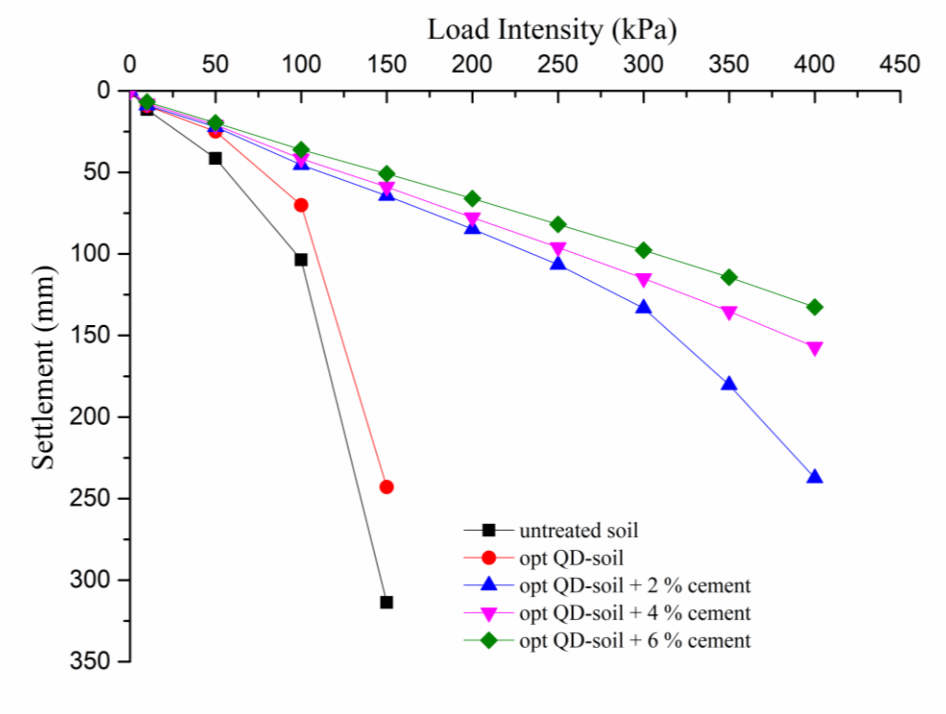


Fig. 6.12 Load intensity-settlement of B=1.5m wide strip footing resting on stabilized soil of depth 2B and width 3B for various soil-QD-cement mixtures

Table 6.12 Settlement of strip footing of width B=2m resting on stabilized soil zone of depth 2B and width 3B considering various soil-quarry dust blends

Load intensity (kPa)	Settlement (mm) (QD)				
	Untreated Marine clay	Optimum QD-soil	Optimum QD-soil + 2% cement	Optimum QD-soil + 4% cement	Optimum QD-soil + 6% cement
10	15.27	12.58	10.26	9.67	8.29
50	62.3	36.07	28.93	25.61	23.89
100	156	103.8	67.83	62.36	54.27
150	523.7	384.6	95.82	87.88	76.17
200	--	--	124.8	114.2	98.63
250	--	--	156.7	141	121.3
300	--	--	199.5	168.3	144.2
350	--	--	379.4	197	168
400	--	--	--	228.5	193.8

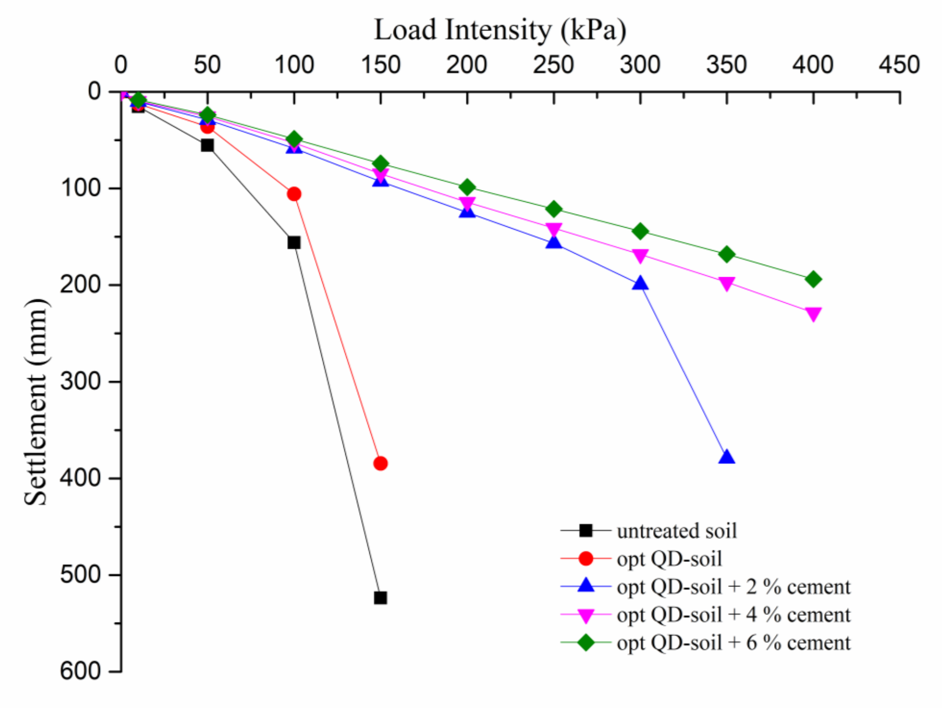


Fig. 6.13 Load intensity-settlement of B=2m wide strip footing resting on stabilized soil of depth 2B and width 3B for various soil-QD-cement mixtures

Table 6.13 Settlement of strip footing of width B=1m resting on stabilized soil zone of depth 2B and width 3B considering various soil-GBFS blends

Load intensity (kPa)	Settlement (mm)				
	Untreated Marine clay	Optimum GBFS-soil	Optimum GBFS-soil + 2% cement	Optimum GBFS-soil + 4% cement	Optimum GBFS-soil + 6% cement
10	7.14	5.34	4.27	4.03	3.5
50	26.13	14.61	11.14	10.31	8.73
100	59.7	26.92	20.54	18.97	16.04
150	188.8	41.15	32.09	29.78	25.37
200	--	55.29	42.13	39.23	33.35
250	--	70.63	52.91	48.6	41.27
300	--	87.17	64.26	58.99	48.96
350	--	106.5	76.72	70.01	57.99
400	--	147	90.34	81.81	67.52

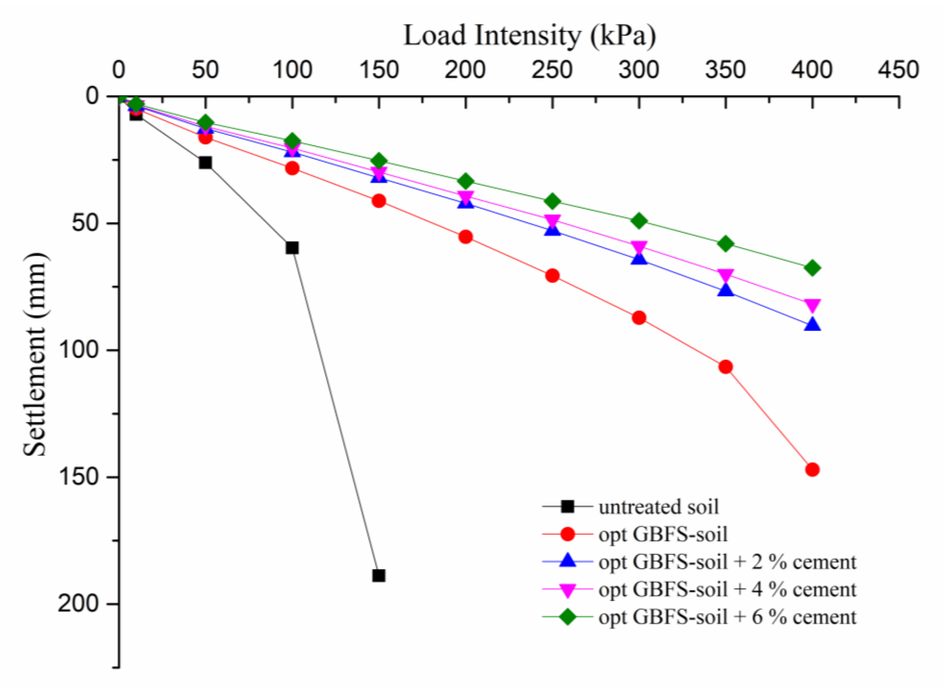


Fig. 6.14 Load intensity-settlement of B=1m wide strip footing resting on stabilized soil of depth 2B and width 3B for various soil-GBFS-cement mixtures

Table 6.14 Settlement of strip footing of width B=1.5m resting on stabilized soil zone of depth 2B and width 3B considering various soil-GBFS blends

Load intensity (kPa)	Settlement (mm)				
	Untreated Marine clay	Optimum GBFS-soil	Optimum GBFS-soil + 2% cement	Optimum GBFS-soil + 4% cement	Optimum GBFS-soil + 6% cement
10	11.43	5.46	5.13	5.19	4.91
50	44.47	21.74	17.49	16.56	14.59
100	103.6	42.99	33.79	31.62	27.43
150	334.7	66.07	51.70	48.24	41.84
200	--	90.48	73.39	65.49	56.12
250	--	115.5	89.46	83.09	70.88
300	--	142.3	108.7	100.8	85.76
350	--	172.1	128.4	118.8	100.7
400	--	230.3	149.9	137.7	116.4

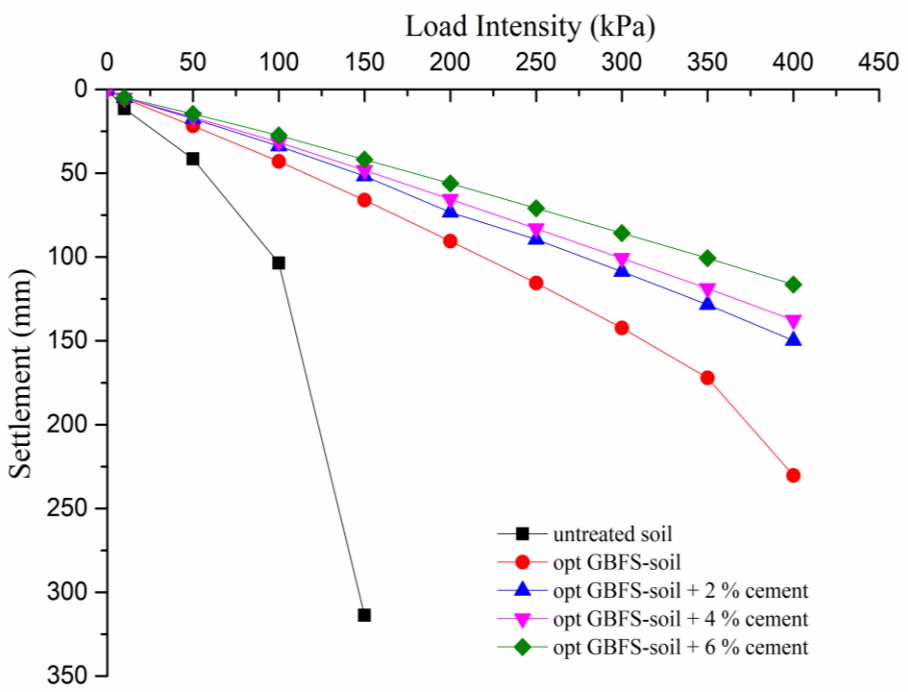


Fig. 6.15 Load intensity-settlement of B=1.5m wide strip footing resting on stabilized soil of depth 2B and width 3B for various soil-GBFS-cement mixtures

Table 6.15 Settlement of strip footing of width B=2m resting on stabilized soil zone of depth 2B and width 3B considering various soil-GBFS blends

Load intensity (kPa)	Settlement (mm)				
	Untreated Marine clay	Optimum GBFS-soil	Optimum GBFS-soil + 2% cement	Optimum GBFS-soil + 4% cement	Optimum GBFS-soil + 6% cement
10	15.27	10.04	8.98	8.19	7.59
50	62.3	30.21	24.77	23.97	22.13
100	156	73.74	58.69	55.22	48.43
150	523.7	104	81.35	76.01	66.96
200	--	141.2	111	103.8	89.94
250	--	176.9	138.2	128.9	111.3
300	--	216.6	165.8	154.3	132.7
350	--	263.2	194.8	180.8	154.8
400	--	357.7	227.3	209.8	178.1

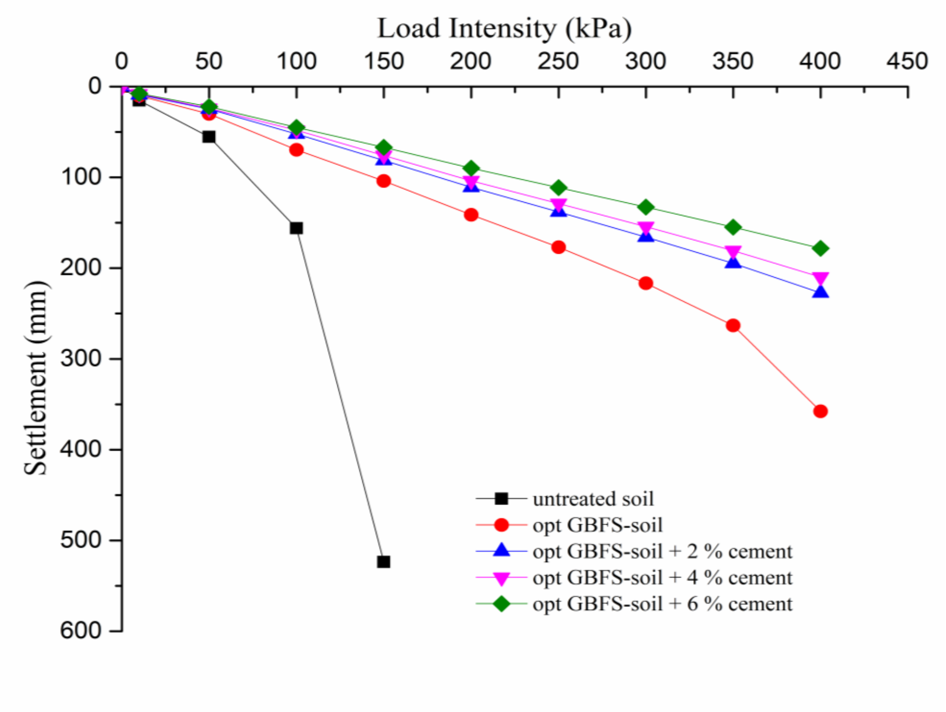


Fig. 6.16 Load intensity-settlement of B=2m wide strip footing resting on stabilized soil of depth 2B and width 3B for various soil-GBFS-cement mixtures

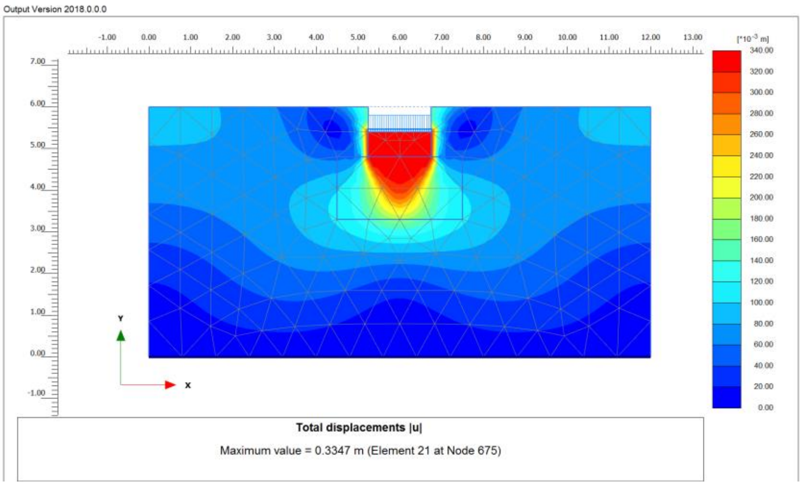


Fig.6.17 Settlement of strip footing resting on natural marine clay

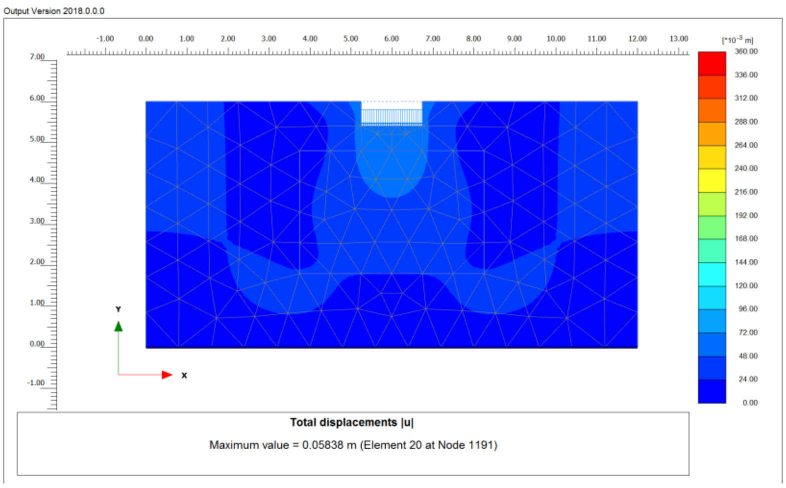


Fig.6.18 Settlement of strip footing resting on QD and cement stabilized marine clay

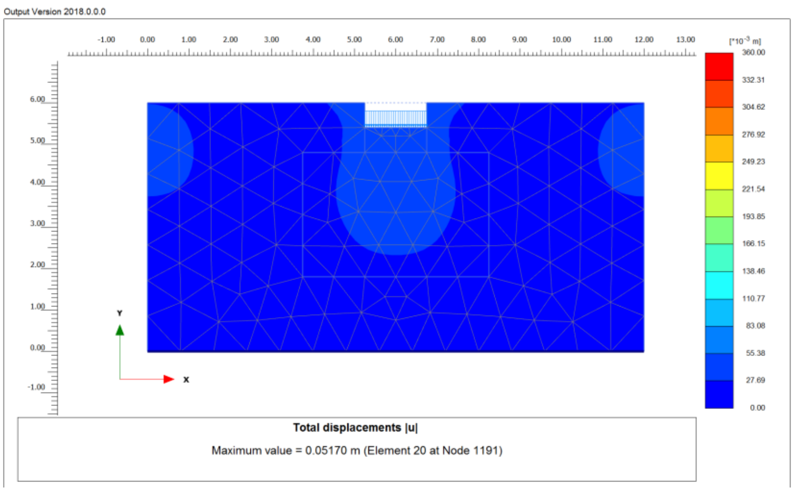


Fig.6.19 Settlement of strip footing resting on GBFS and cement stabilized marine clay

Figure 6.17 is the settlement contours for 2m wide strip footing resting on untreated natural marine clay for an applied pressure of 150 kPa. Figure 6.18 corresponds to settlement contours of 2m wide strip footing with applied pressure of 150 kPa but resting on 2% addition to optimum quarry dust stabilized soil mix of stabilized zone of 2B depth and 3B width. Figure 6.19 corresponds to settlement contours of same 2m wide strip footing with applied pressure of 150 kPa but resting on 2% addition to optimum granulated blast furnace-marine clay stabilized soil mix of stabilized zone of 2B depth and 3B width. From the later figures, the absence of high settlements (red scale) is observed. This absence is resulted from the presence of stabilized zone that spreads the applied pressure to a larger area and also absorbs the intensity (Mosallanezhad and Moayedi 2017).

Comparing Figure 6.19 with Figure 6.18, the settlements contours turned out to lesser value (dark blue) upon treating the stabilized zone with granulated blast furnace and cement than by quarry dust and cement. Hence GBFS treated marine clay performs well compared with quarry dust treated marine clay.

6.4 Net allowable pressure for treated marine clay

Table 6.16 shows the net allowable pressure on strip footing which is resting on quarry dust and cement treated marine clay mixtures. The stabilized zone is of depth 1B and width 2B. The net allowable pressure corresponds to an allowable settlement of 25mm. Table 6.17 represents the net allowable pressure for footing resting on GBFS and cement treated marine clay mixtures of stabilized zone of depth 1B and width 2B. The allowable pressure decreases with the increase in the width of the strip footing. With the optimum quarry dust-marine clay mix the allowable pressure improves. For the strip footing of width 1.5m, the allowable pressure increases from 25.07 kPa (untreated soil) to 41.67 kPa with introduction of optimum quarry dust-marine clay mix of zone 1B depth and 2B width. Further 4% cement addition to optimum quarry dust soil mix showed a net allowable pressure of 51.5 kPa, the percentage increase in net allowable pressure observed is 105% compared with the allowable pressure when strip footing resting on untreated marine clay. Similar kind of improvement is seen with GBFS incorporation. 103% and 126% of increase in net allowable pressure is observed for strip footing resting on optimum GBFS-marine clay and 4% cement addition to optimum GBFS-marine clay blend of 1B width 2B depth stabilized zone respectively.

Table 6.16 Net allowable pressure of strip footing resting on quarry dust and cement treated marine clay mixtures of stabilized zone of depth 1B and width 2B for an allowable settlement of 25mm

Combination	1m wide footing		1.5 m wide footing		2m wide footing	
	kPa	% increase	kPa	% increase	kPa	% increase
Untreated soil	47.63		25.07		18.27	
Opt QD-soil	57.4	20.6	41.67	66.0	25.2	37.4
Opt QD-soil +2% cement	70.36	47.8	50	99.2	32.0	74.9
Opt QD-soil +4% cement	73.85	55.1	51.5	105.1	37.1	102.6
Opt QD-soil +6% cement	81.39	71.0	53.08	111.5	41.7	127.8

Table 6.17 Net allowable pressure of strip footing resting on GBFS and cement treated marine clay mixtures of stabilized zone of depth 1B and width 2B for an allowable settlement of 25mm

Combination	1m wide footing		1.5 m wide footing		2m wide footing	
	kPa	% increase	kPa	% increase	kPa	% increase
Untreated soil	47.63		25.07		18.27	
Opt GBFS-soil	66.52	39.66	51.12	103.4	32.96	80.3
Opt GBFS-soil +2% cement	75.53	58.58	56.05	122.4	44.93	145.4
Opt GBFS-soil +4% cement	78.2	64.18	57	126.3	48.08	162.8
Opt GBFS-soil +6% cement	84.42	77.24	59.3	135.7	50.59	176.5

Table 6.18 shows the net allowable pressure on strip footing which is resting on quarry dust and cement treated marine clay mixtures. The stabilized zone is of depth 2B and width 3B. The allowable pressure corresponds to an allowable settlement of 25mm. Table 6.19 represents the net allowable pressure for footing resting on GBFS and cement treated marine clay mixtures of same stabilized zone. For the strip footing of width 1.5m resting on optimum quarry dust-soil stabilized zone 2B depth and 3B width, the allowable pressure increases from 25.1 kPa (untreated soil) to 48.4 kPa with introduction of optimum quarry dust-marine clay mix of 1B depth and 2B width. Further 4% cement addition to optimum quarry dust-soil mix showed a net allowable pressure of 79.4 kPa, the percentage increase in net allowable pressure observed is 168% compared with the allowable pressure when strip footing resting on untreated marine clay. Similar kind of improvement is seen with GBFS incorporation. 130% and 211% of increase in net allowable pressure is observed for strip footing resting on optimum GBFS-marine clay and 4% cement addition to optimum GBFS-marine clay blend of 1B width 2B depth stabilized zone respectively.

With the increase in area of stabilized zone from 1B depth x 2B width to 2B depth x 3B width, the allowable pressure has increased. The percentage of improvement in allowable pressure is found to increase from 136% to 261% with increasing the stabilized zone from 1B depth-2B width to 2B depth-3B width of strip footing of width 1.5 m and stabilized zone of combination 6% cement addition to optimum GBFS-marine clay. The trend is the same for all the other mixtures and widths of the strip footing (C. Sekhar 2017).

Comparing the settlement values and the allowable pressure for all combination of load intensity and widths of the strip footing, GBFS in comparison with quarry dust (with or without cement) exhibited less settlement and more allowable pressure values. This lower settlement and higher allowable load attribute GBFS as better granular stabilizer in comparing with quarry dust.

6.5 Summary

The settlement analysis performed using PLAXIS 2D on various combinations of treated soil, zones of treatment and load intensities infers that resistance towards loads is higher for treated soil compared to untreated soil. Also, the settlement of strip footing is less for GBFS treated marine clay compared to QD treated marine clay with/without cement for all combination of loads and depth of treatment.

Table 6.18 Net allowable pressure of strip footing resting on quarry dust and cement treated marine clay mixtures of stabilized zone of depth 2B and width 3B for an allowable settlement of 25mm

Combination	1m wide footing		1.5m wide footing		2m wide footing	
	kPa	% increase	kPa	% increase	kPa	% increase
Untreated soil	47.63		25.07		18.27	
Opt QD-soil	72.1	51.6	48.4	92.9	31.1	70.2
Opt QD-soil +2% cement	107.1	125.0	61.1	143.2	41.6	127.2
Opt QD-soil +4% cement	117.3	146.4	67.4	168.4	48.5	164.9
Opt QD-soil +6% cement	136.6	187.0	79.4	216.4	51.8	183.2

Table 6.19 Net allowable pressure of strip footing resting on GBFS and cement treated marine clay mixtures of stabilized zone of depth 2B and width 3B for an allowable settlement of 25mm

Combination	1m wide footing		1.5 m wide footing		2m wide footing	
	kPa	% increase	kPa	% increase	kPa	% increase
Untreated soil	47.63		25.07		18.3	
Opt GBFS-soil	92.2	93.7	57.7	129.8	39.7	116.8
Opt GBFS-soil +2% cement	119.3	150.6	73.0	191.0	50.3	175.0
Opt GBFS-soil +4% cement	127.9	168.7	78.0	210.8	51.6	182.1
Opt GBFS-soil +6% cement	148.0	211.0	90.5	260.7	55.4	202.5

CHAPTER 7

MANUFACTURE AND TESTING OF COMPRESSED STABILIZED EARTH BLOCKS

7.1 General

The utilization of soil/earth as a building construction material has been initiated since human civilization is evidenced. Dating back to the period of the Mesopotamia and Indus valley civilizations, the use of earth to fulfill the basic needs of humans has been advocated. The availability of soil in abundance has made it one of the best natural resources to be used as an effective construction material notably in the form of burnt bricks. But the production of these conventional burnt bricks requires quality material and high energy expenditure which is a global concern. The environmental impact and ecological concern have made technologists invent a new alternative to fired bricks. Compressed Stabilized Earth Blocks (CSEB) can claim many advantages over conventional burnt bricks. It utilizes locally available materials, takes up the least time, and requires fewer skilled labours for its production. Also, as it is caste in-site it resulted in the most economic and fastest rate of construction (Nagaraj et al. 2014).

During the past 5 decades, CSEBs were used in load-bearing masonry construction in various parts of the globe. CSEBs are produced by compressing a wet mixture of soil and suitable additives in a manually operated press to a possible high-dense block. CSEBs possess good mechanical strength, insulating capacity, and high thermal resistance which results in fewer carbon footprints and less embodied energy in the production stage. The development in technology and research has recommended the use of industrial by-products and waste in the manufacture of CSEBs making it a low-cost building material in the housing sector (Akinyemi et al. 2020).

Though various factors are influencing the strength, durability, and quality of CSEBs; the admixture dosage and the density attainment of the block are the noteworthy parameters.

7.2 Experimental program

For the preparation of the earth block, the MARDINI block making machine was used. A manual toggle lever-based ram generating a pressure of 2-3 MPa was attached to compress the mixture to a block of size 230mm x 108mm x 75 mm (Figure 7.1). The calculated amount of optimum soil-granular materials and cement were dry mixed and spread on a big tray. Water is added to this dry mix and blended well. This wet mix is transferred to MARDINI and compressed. The compressed block is ejected, labelled and cured for 28 days (Figures 7.2-7.4).

Fig. 7.1 Compressing the block by pulling down the lever arm

Fig. 7.2 Green block samples

Fig. 7.3 Curing of CSEBs

Fig. 7.4 Oven drying of CSEBs

7.3 Results and discussions

7.3.1 Compressive Strength

The dry compressive strength (Figure 7.5) of the marine clay soil block is 0.62 MPa. Similarly, compressive strength of 1.47 MPa and 1.78 MPa were obtained for optimum quarry dust-marine clay and optimum GBFS-marine clay blocks respectively. Upon immersion of these samples in water for the determination of its wet compressive strength and other properties, the blocks disintegrated and failed due to a lack of binding between the particles of the block. Hence cement is added to enhance the adhesion of constituents of the block (Reddy and Gupta 2006).

Fig. 7.5 Compression test of CSEB

The compressive (wet and dry) strength increases with the dosage of cement to the various optimum mixes. This improvement is due to the coating and binding of hydration products in the soil matrix of the block resulting from the cement-hydration reaction. This reaction enhances the stiffness and integrity of the blocks. Higher the cement addition more is the soil matrix bond connectivity leading to the formation of an intact rigid block. In the case of untreated soil and optimum granular material treated soil blocks, the high pore pressure exerted due to the water immersion creates the liquefaction of soil constituents resulting in null resistance for the wet compression load. Upon cement addition, the hydration products fill the voids space with an interconnecting bond between the constituents of the block and thus improve the block capacity against the pore water pressure. Hence the cement-treated optimum granular soil wet blocks show resistance towards the compressive load applied. Due to the presence of CaO in GBFS, blocks prepared using GBFS (Table 7.1) exhibited the best compressive strength compared to quarry dust added to the soil (Table 7.2). The rough texture and angular nature of quarry dust particles have resulted in better resistance to external compressive load in case of quarry dust added soil blocks.

Finally, in the view of the wet compressive capacity criteria (wet strength > 3.5 MPa), the blocks made of 10% addition of cement to the optimum soil-quarry dust mix (Figure 7.6) and 8% addition of cement to optimum soil-GBFS mix (Figure 7.7) can be used in the construction of load-bearing walls as per Indian Standards (IS).

Table 7.1 Compressive strength of quarry dust stabilized CSEBs

Sl. No.	Percentage cement addition to optimum QD-soil mix	Compressive strength (MPa)	
		Dry	Wet
1	0%	1.47	---
2	6%	3.57	2.77
3	8%	4.19	3.21
4	10%	5.03	3.72
5	12%	5.83	4.31
6	14%	6.13	4.97

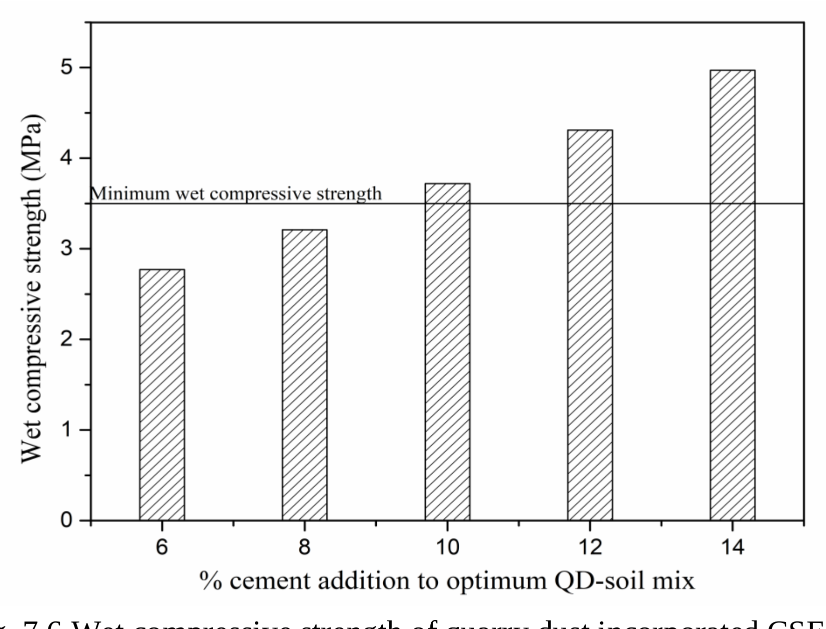


Fig. 7.6 Wet compressive strength of quarry dust incorporated CSEBs

Table 7.2 Compressive strength of GBFS stabilized CSEBs

Sl. No.	Percentage cement addition to optimum GBFS-soil mix	Compressive strength (MPa)	
		Dry	Wet
1	0%	1.78	---
2	6%	4.31	3.32
3	8%	4.66	3.61
4	10%	5.39	4.22
5	12%	6.74	5.16
6	14%	7.01	5.47

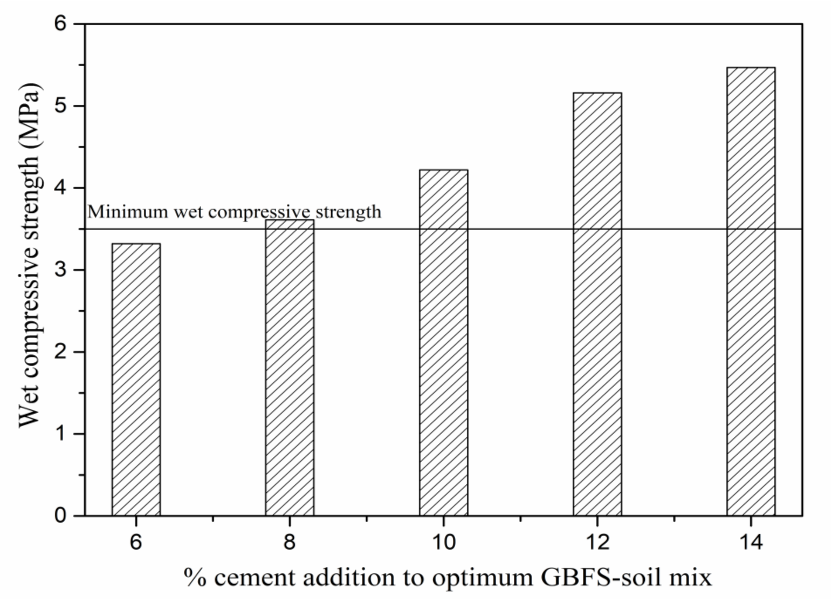


Fig. 7.7 Wet compressive strength of GBFS incorporated CSEBs

7.3.2 Water absorption and rate of water absorption

The average values of water absorption (Figure 7.8) for various combinations are shown in Table 7.3. The primary observation from the test data is that the water absorption decreases with the increased dosage of cement to optimum granular soil mixes. The higher cement content leads to the reduction of interconnecting voids in the matrix as the hydration products bind the block constituents together and harden the soil matrix. These reaction products due to cement hydration have contributed to the micro-level alterations in the matrix and this interaction, in turn, leads to the reduction of void space and the consequent decrement in the water absorption of the blocks. As GBFS is a crystalline foamy material, the water affinity is comparatively on the higher side. Interestingly, all the blocks prepared with cement and various granular additives have shown water absorption values lesser than the upper limit of 15% as per the Indian Standard specifications. Hence, all the blocks treated with granular material and cement has passed the water absorption criteria (Nagaraj et al. 2016).

Fig. 7.8 Water absorption test on CSEBs

The rate of water absorption of the combinations satisfying the minimum strength criteria for the load-bearing walls is shown in Figure 7.9. The absorption rate is high for the first 60 minutes of soaking, but the rate declines for the blocks as time passes. Better performance with regards to the rate of water absorption was observed in samples with cement added to optimum quarry dust compared to samples with cement added to optimum GBFS (Riza et al. 2010).

Table 7.3.Water absorption of CSEBs

Sl. No.	Percentage of cement addition	Water absorption (%)	
		optimum quarry dust-soil mix	optimum GBFS-soil mix
1	6%	11.32	12.31
2	8%	10.61	11.98
3	10%	10.32	10.13
4	12%	9.21	9.37
5	14%	9.06	9.21

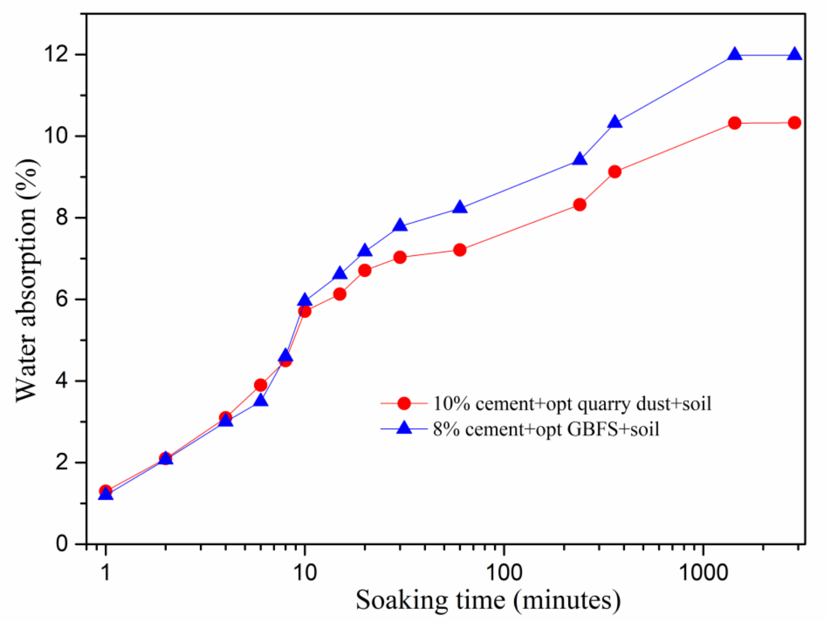


Fig. 7.9 Rate of water absorption of the CSEBs satisfying the compressive strength criteria

7.3.3 Initial rate of water absorption

The variation of the initial rate of water absorption (IRA) (Figure 7.10) with varying percentages of cement added to various granular additives is shown in Table 7.4. IRA declines with an increase in the amount of cement added to soil mixes. Quarry dust-replaced soil showed better performance compared to soil with GBFS additive. The lower IRA values of the block are attributed to a high-density matrix and the lower surface porosity property.

Fig. 7.10 Initial rate of water absorption test

The IRA value of 0.217, and 0.301 g/min/cm^2 are observed for 10% cement added to optimum quarry dust and 8% cement added to optimum GBFS for marine clay respectively.

Table 7.4 Initial rate of water absorption of CSEBs

Sl. No.	Percentage of cement addition	Initial rate of water absorption (g/min/cm^2)	
		optimum quarry dust-soil mix	optimum GBFS-soil mix
1	6%	0.293	0.356
2	8%	0.275	0.301
3	10%	0.217	0.262
4	12%	0.172	2.237
5	14%	0.129	0.156

7.3.4 Alternate wetting and drying

The variation of loss in mass after the 12 cycles of alternate wetting and drying for blocks casted using granular additive and cement is shown in Table 7.5. From the table, it is observed that the mass loss decreases with increasing cement percentage. Also, from the graph, it can be concluded that the blocks prepared using GBFS are more durable than blocks casted using quarry dust as an additive. The blocks satisfying the minimum wet strength criteria have passed the durability upper limit of mass loss criteria (<3%) as per Indian standard specification.

Table 7.5 Mass loss of CSEBs

Sl. No.	Percentage of cement addition	Mass loss (%)	
		optimum quarry dust-soil mix	optimum GBFS-soil mix
1	6%	7.83	4.15
2	8%	4.31	2.83
3	10%	2.90	2.55
4	12%	2.78	2.19
5	14%	2.32	2.36

Percentage mass losses of 2.9%, and 2.83% mass loss were observed for 10% cement addition to optimum quarry dust-soil mix, and 8% cement addition to optimum GBFS-soil mix respectively.

The blocks subjected to durability testing were later tested for their wet compressive strength. The wet compressive strength was found to be more than the values obtained from the table for their corresponding combination. A percentage increase in strength of 13%, and 34% of wet strength were obtained for 10% cement to optimum quarry dust, and 8% cement added to optimum GBFS to marine clay. The incremental strength is due to the accelerated cementitious pozzolanic reaction of the cement with block constituents with the slightly elevated temperature during the durability test. Interestingly, the increased wet strength is more for GBFS treated soil than soil treated with quarry dust, which proved that GBFS with activators performs better with accelerated curing (Nagaraj et al. 2016).

7.4 Summary

The chapter emphasizes the potential benefit of utilizing the treated marine clay in the manufacture of earth blocks. Compressed stabilized earth blocks casted with 10% cement+optimum Marine clay-QD and 8% cement+optimum Marine clay-GBFS mix can be used for load-bearing walls. These blocks satisfies the wet compressive strength, water absorption and durability criteria as per IS codal provision. As the cement requirement for GBFS incorporated blocks are less compared to quarry dust treated blocks, it can be inferred that usage of GBFS as additive is more economical and durable than quarry dust as granular additive.

CHAPTER 8

TEMPERATURE EFFECT ON PROPERTIES OF MARINE CLAY

8.1 General

The alteration in the physical and engineering properties of clay due to thermal variation is vital. It has a significant effect in sample preparation prior to testing and also in the field of ground and geotechnical applications. Marine clay was dried using an oven maintained at 40°C, 60°C, 80°C, 100°C and 150°C temperature and also by air (room temperature). The influence of drying temperature on the geotechnical properties of marine clay is exhibited in this chapter.

8.2 Effect of geotechnical properties on marine clay

Two methods of drying, namely air drying and oven drying were adopted. Moist soil was placed in the oven and subjected to different temperatures (40°C, 60°C, 80°C, 100°C and 150°C) for drying. The results obtained from the detailed laboratory tests conducted on the dried marine clay are presented in Table 8.1 and Figures 8.1 to 8.3. The variation in the geotechnical properties of the dried soil are analyzed and discussed in the subsequent sub-sections.

Table 8.1 Index and strength values of marine clay at different temperature

Sl. No	Drying temperature	Liquid limit (%)	Plastic limit (%)	MDU (kN/m^3)	OMC (%)	UCS (kPa)
1	Air	91	33	13.6	27.4	98
2	40°C	86	32	13.7	26.4	101
3	60°C	81	31	14.6	25.8	120
4	80°C	76	30	15	23.6	143
5	100°C	69	30	15.2	21.5	156
6	150°C	74	32	13.3	26.6	90

8.3 Effect of drying temperature on particles size

The effect of temperature variation on the amount of clay, silt and sand fractions of marine clay is shown in Fig. 8.1. Upon soil drying from room temperature to100°C, the clay size particles reduce from 36% to 27%. The amount of clay fractions remains constant with a further increase in drying temperature to 150°C.The decrease in clay size particles till 100°C temperature dried soil is attributed to the increase in silt particles from 33% to 39% and sand particles from 31% to 34%. This is due to the loss of inter-particular water and partial loss of adsorbed water upon drying (Tan et al. 2004; Villar and Lloret 2004). This loss of water facilitates interparticle attraction and fine particle aggregation leading to the capillary stresses enhancement upon drying at the higher temperature (Rao et al. 1989). Thus, the capillary stress enhancement creates a close contact of fine particles, leading to the growth of bulk particles resulting in a robust Van der Waal force and coulombic bonds, which are not easily separable (Pandian et al. 1993).

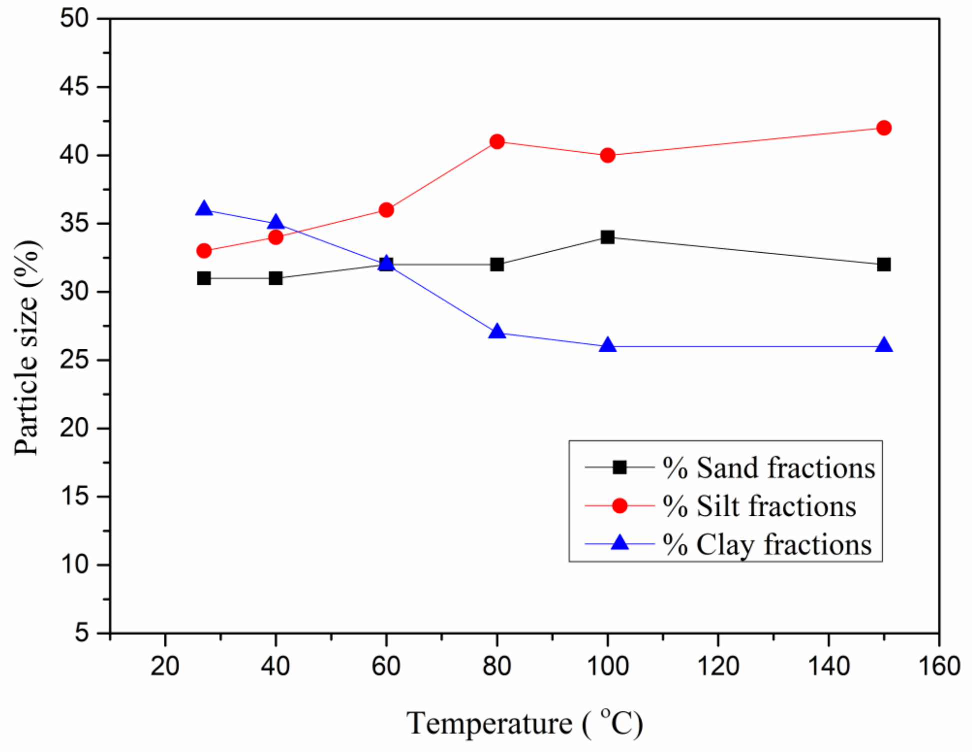


Fig.8.1 Variation of particle fractions of marine clay with drying temperature

8.4 Effect of drying temperature on consistency limits

The influence of the drying temperature on the consistency limits is shown in Table 8.1. It is noticed that there is a drop in the liquid limit value from 91% to 69% on drying the soil from 27°C to 100°C. Soil drying at a temperature of 100°C has more effect on the consistency limits than soil dried at lower or higher than 100°C. This is due to the fact that with an increase in drying temperature, the interparticle interactions are high (Rao et al. 1989). It leads to the lowering of specific surface area and hence reduces the liquid limit, which is also justified by the results of particle size analysis (Chen et al. 2016). Similarly, the plastic limit decreases with the increase in drying temperature till 100°C. A marginal change in the plastic limit is observed with 150°C drying temperature. The plasticity index decreases from 58% to 39% upon drying the soil from room temperature to 100°C. The drastic decrease in liquid limit reflected in the decrease in the plasticity index as the plasticity index is directly proportional to the liquid limit (Pandian et al. 1991).

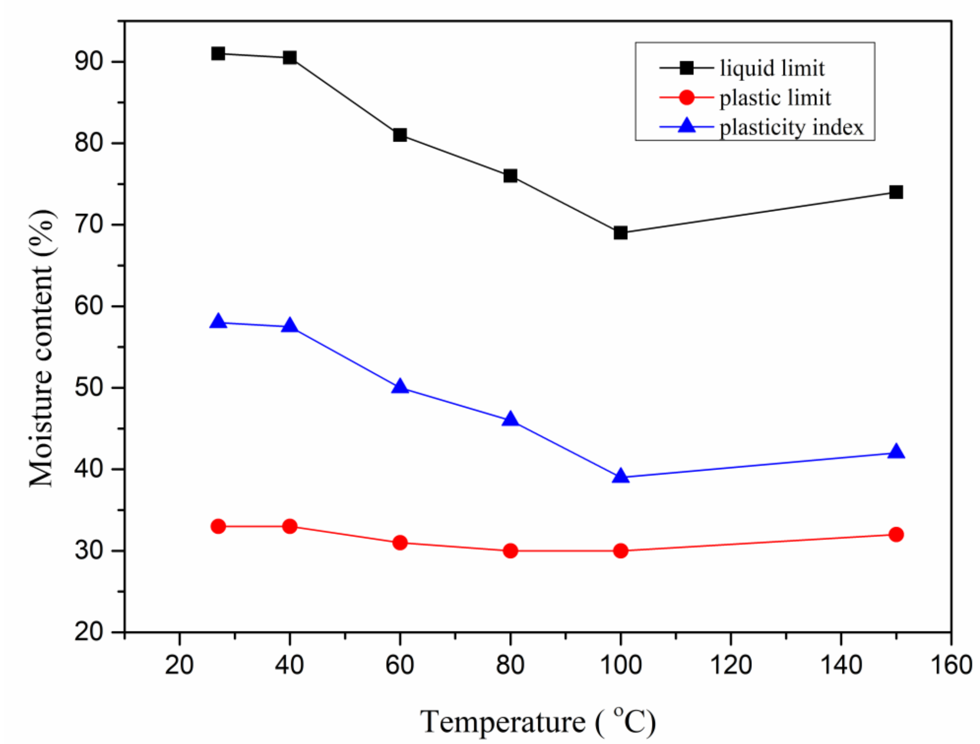


Fig. 8.2 Variation in consistency limits of marine clay with drying temperature

8.5 Effect of drying temperature on compaction characteristics

Maximum dry unit weight (MDU) has increased from 13.6 kN/m^3 (air-dried soil) to15.2 kN/m^3 upon drying the marine clay at a temperature of 100°C. This decrease is because of a reduction in the number of micropores and the formation of spatial homogenous compacted particles(Chen et al. 2016). The soil dried at 100°C, the MDU increases by 12% compared with the air-dried sample.

The MDU drastically decreases to 13.3 kN/m^3 with a further increase in drying temperature (150°C). This decrease in MDU is due to the development of microcracks (Gadzama et al. 2017). Fig 8.3 shows the variation of MDU of marine clay with drying temperatures.

The optimum moisture content (OMC) from the light compaction test decreases with the increase in drying temperature till 100°C. Further, at 150°C drying temperature, OMC increases. The decrease in OMC till 100°C drying temperature is due to the reduction in the clay size particle and agglomeration of this clay size to form silt size and sand size particle which leads to a decrease in the specific surface area of the soil. A 21% decrease in OMC of marine clay was found when dried at 100°C compared with the OMC of an air-dried sample (Fig 8.3).

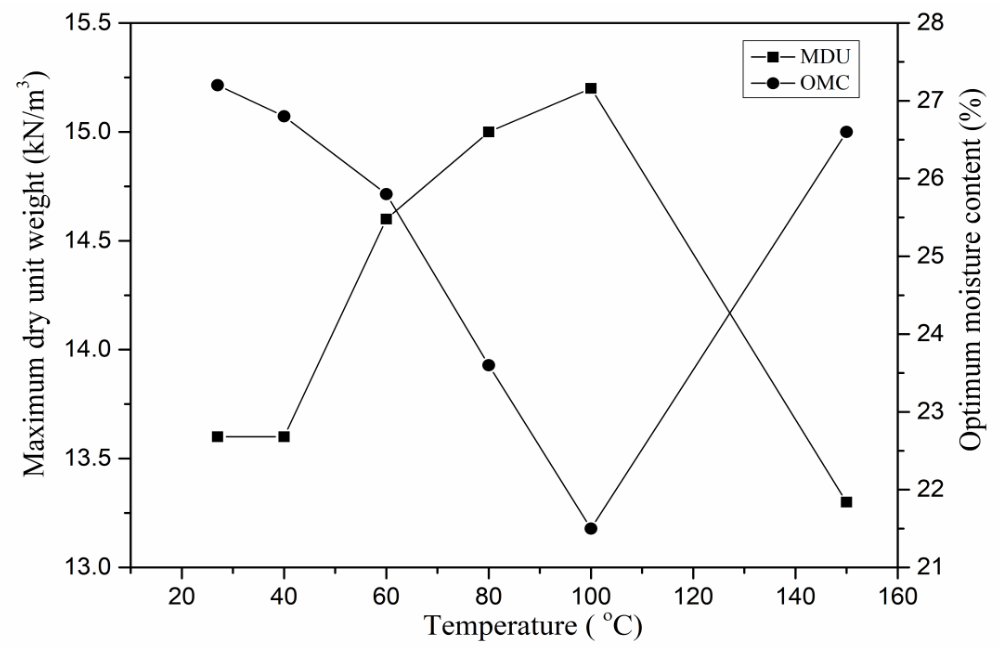


Fig. 8.3 Variation in compaction characteristics of marine clay with drying temperature

8.6 Effect of drying temperature on unconfined compression strength

The UCS test result for marine clay dried at various temperatures is shown in Table 8.1.The strength increases from 98 kPa (air-dried soil) to 156 kPa for soil dried at 100°C.Further, the strength decreased to 90 kPa for 150°C dried soil. The increased MDU and decreased OMC resulted in the strength enhancement for 100°C dried soil.

8.7 Effect of drying temperature on organic matter and *pH*

The precipitation and decomposition of organic matter facilitates the development of acidity in soil. Upon drying, the organic matter decreased and thus the pH decreases (Sunil and Krishnappa 2012). The aggregation of finer particle is also contributed by the organic content in the soil (Pandian and Nagaraj1993), as the drying temperature increases the organic matter decreases leading to the aggregating of fine particles to form bulk particles. From Figs. 6 and 7, it is evident that the organic matter and pH decreased with the increase in drying temperature.

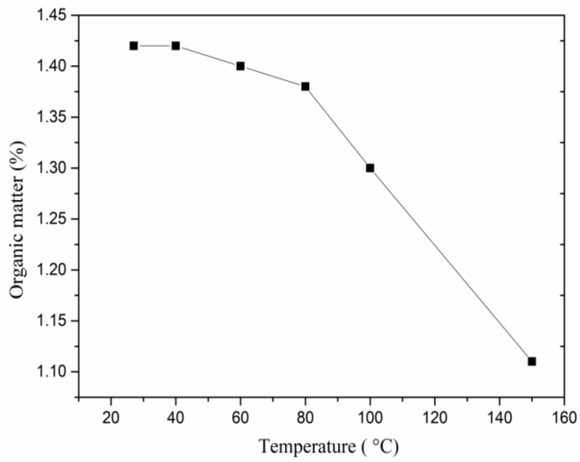


Fig.8.4 Variation of organic matter of marine clay with drying temperature

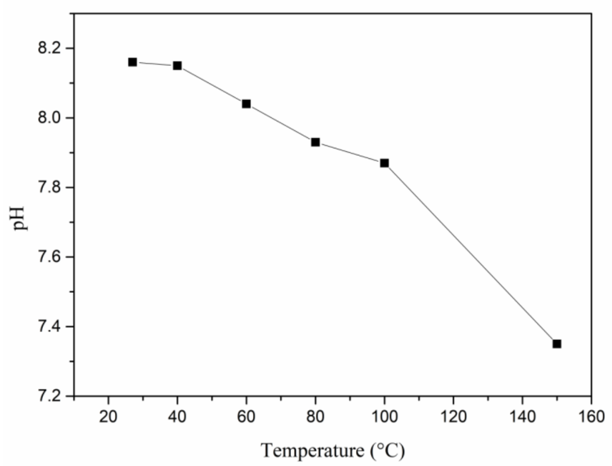


Fig. 8.5 Variation of pH of marine clay with drying temperature

8.8 Effect of rewetting on the engineering properties of dried soil

The air-dried and different temperature oven-dried marine clay was soaked for 7and 28 days in water and tested for various its consistency limits. Air-dried samples have shown no change in their consistency limits on rewetting (Table 8.2). There was a minimal increase in the liquid limit and no change in the plastic limit for different temperature-dried soil upon rewetting for 7 days (Pandian 1993).

Table 8.2 Engineering properties of dried marine clay upon rewetting

Sl. No.	Rewetting days	Liquid limit (%)		Plastic limit (%)		Plasticity index (%)	
		7	28	7	28	7	28
1	Air dried (27 °C)	91	91	33	33	58	58
2	40°C	90	90	33	33	57	57
3	60°C	83	84	31	31	52	53
4	80°C	78	78	31	31	47	47
5	100°C	70	71	30	31	40	40
6	150°C	75	75	32	32	43	43

There was a minimal increase in the liquid limit and no change in the plastic limit for different temperature-dried soil upon rewetting for 7 days. There observed a small change in liquid limit and plastic limit for 28-day rewetted marine clay samples. This is due to the fact that, with an increase in drying temperature, there is a loss of inter-particular water and partial loss of adsorbed water (Rao et al. 1990), facilitating the high interparticle interactions (Chandrakaran 1989). This creates a close contact of fine particles leading to the growth of bulk particles resulting in a strong Van der Waal force and coulombic bonds which are not easily separable. The properties and their values after rewetting for 7 and 28 days are shown in Table 8.2.

8.9 Summary

This chapter emphasizes the influence of drying temperature on the geotechnical properties of marine clay. Experimental results shows a decrease in clay size fraction and increase in silt and sand size particles with the increase in drying temperature. The geotechnical properties have improved till the soil dried to 100 °C beyond this drying temperature the trend reversed. There is no significant effect on index properties upon rewetting the dried samples compared with the properties of dried samples before rewetting.

CHAPTER 9

CONCLUSIONS

9.1 Closure

In the present research work, the potential utilization of industrial granular by-products in stabilizing fine-grained marine clay is attempted. The stabilized mixture was examined for few geotechnical applications.

The high liquid limit, high plasticity index, poor shear strength and high compressibility character of marine clay paves the path for stabilization. Also, marine clay is sensitive to temperature changes. The variation of index and compaction characteristics of marine clay was analyzed through laboratory tests. The primary objective of the work is to improve the shear strength properties using industrial by-products and cement. The granular industrial by-products used are quarry dust and granulated blast furnace slag. Quarry dust is the output generated in the granite stone crushers and granulated blast furnace slag is obtained from the ferrous industries. Experiential studies evaluate the potential usage of granular additives in the stabilization of marine clay. In this regard, marine clay was replaced by quarry dust/granulated blast furnace slag in various percentages and tested for geotechnical performance. Through the unconfined compressive test results, the optimum replacement percentage was established. Hence 65% marine clay+35% quarry dust is termed as optimum quarry dust-soil mix and 60% marine clay+40% GBFS is termed as optimum GBFS-soil mix. The rough texture and annular flaky particles of quarry dust impart friction between the constituent parties of the mix and hence the shear strength of the mix increases. The presence of CaO in GBFS initiated the pozzolanic reaction and the reaction products generated binds the constituent parties of the mix. Also, the rough glassy granular particles of GBFS induce friction in the mix and hence the friction angle increases. The enhanced cohesion and angle of internal friction due to GBFS supplementation will improve the shear strength of the mix.

Further, to improve the cohesion of the optimum-soil granular mix, Cement is added in various proportions and analyzed for its geotechnical performance. The hydration reaction of cement produces cementitious products and these products increase the shear strength of the mix. These cementitious products generated due to pozzolanic and hydration reactions were justified through SEM and XRD analyses. CSH, CAH, CASH were the major cementitious compounds observed in the analysis and they were responsible for binding the constituent soil particles of the mix. Thus the efficacy of marine clay stabilized using the granular industrial by-products and cement is worked out in the present research.

9.2 Conclusions

Based on the detailed investigation carried out in the present work, the following conclusions are drawn:

- Quarry dust and GBFS are waste/by-product accumulated in quarries and iron industry respectively. GBFS containing a certain amount of CaO that can impart binding property in the presence of water. The MDU increases and OMC decreases with the addition of either quarry dust or GBFS to marine clay.
- 35% quarry dust replacing marine clay showed a maximum UCS strength compared to other soil-quarry dust mixes. Hence, 35% quarry dust+65% marine clay is considered as the optimum quarry dust-soil mix. Similarly, in the case of GBFS upon curing, 40% GBFS+60% soil exhibited maximum UCS and was considered as optimum GBFS-soil mix. Optimum quarry dust-soil showed an increase in UCS of 187% compared to untreated soil. For optimum GBFS-soil mix, the increase in UCS of 497% is observed compared to untreated soil when cured for 28 days.
- With the addition of quarry dust or GBFS to the soil, liquid limit decreased due to the reduction of specific surface area. The plasticity index decreased by 41% for optimum quarry dust-soil mix and 39% for optimum GBFS-soil mix.
- Cohesion increases by 247% for optimum GBFS-soil mix for 28 days of curing. The angle of internal friction improves with the addition of coarser

(quarry dust or GBFS) material. The increased cohesion at 28 days curing in case of GBFS addition is due to the pozzolanic reaction between free lime and silica in the mix.

- With the addition of cement to optimum granular-soil mix (quarry dust/GBFS), the UCS value tremendously increases. Cement coats and binds the constituent particles of soil-granular composite and gives a rigid structure. This dual effect of improved gradation (by granular material addition) and binding (cement addition) provides improved friction and cohesion to the marine clay.
- CBR value increases with the addition of granular additive (QD/GBFS). GBFS stabilized marine clay showed better performance than quarry dust stabilized marine clay. From the durability studies, it can be inferred that optimum GBFS stabilized Marine clay with cement is more durable than optimum QD stabilized Marine clay with cement.
- The XRD studies conclude that GBFS addition to marine clay in the presence of water has produced cementitious products, which are responsible for the better strength gain. CSH, CASH and other cementitious compounds were observed in the cement stabilized optimum granular mixes. The formation of cementitious mass is identified in the SEM image of GBFS stabilized marine clay. This cementitious mass is due to the reaction between free lime in slag and reactive silica in soil. Rigid compacted matrix with cementitious products is observed in the SEM micrographs of cement added to optimum granular mixes.
- The load intensity-settlement analysis infers that, with the increase in the area of stabilized zone, the net allowable pressure increases. Also, with the higher addition of cement to the optimum granular-soil mix, the net allowable pressure increases. The settlement of GBFS incorporated mix showed lesser values compared to quarry dust incorporated mix. Thus GBFS performed well compared to quarry dust as a stabilizer to marine clay.
- Compressed stabilized earth blocks cast using 10% cement to the optimum quarry dust-soil mix can be used as construction units in a load-bearing wall.

Similarly, CSEB made with the addition of 8% cement to the optimum soil GBFS-soil mix can be used for load-bearing walls. These blocks satisfy the minimum wet compressive strength as per IS codal provision. The water absorption and durability performance of these blocks are also satisfied according to IS requirements.

- Marine clay is sensitive to temperature change. The index and strength properties vary with the temperature change. Upon rewetting the dried marine clay, changes in index properties are found to be irreversible.

9.3 Scope for future work

Present investigation has focused on stabilizing marine clay using quarry dust, granulated blast furnace slag and cement.

There is scope for further exploration in the following areas

- The stabilization of marine clay using granular additives and lime.
- The stabilization of marine clay using granular additives and alkali activated ground granulated blast furnace slag.

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LIST OF PUBLICATIONS

Sl. No.	Title of the paper	Authors	Name of the Journal/ Conference, Vol., No., Pages	Month & Year of Publication
1	Geotechnical Investigations on Marine Clay Stabilized Using Granulated Blast Furnace Slag and Cement	H. K. Preetham and Sitaram Nayak	International Journal of Geosynthetics and Ground Engineering, Vol 5 (4), pp. 28	October 2019
2	Experimental investigation on the stabilization of soft clay using granulated blast furnace slag	H. K. Preetham, Sitaram Nayak and Surya E V	IOP Conference Series: Materials Science and Engineering, vol. 561, no. 1, p. 012047(1-6)	November 2019
3	Effect of Drying Temperature and Rewetting on the Engineering Properties of Marine Clay	Sitaram Nayak and H. K. Preetham	Transportation Infrastructure Geotechnology Vol 7(4), pp. 517-534	March 2020
4	Efficacy of Granular Industrial By-Products as an Alternate to Sand in the Manufacture of Compressed Stabilized Earth Blocks	H. K. Preetham and Sitaram Nayak	KSCE Journal of Civil Engineering (Manuscript No: KSCE-D-21-01353)	Under Review

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