# A STUDY ON GEOPOLYMER STABILIZED AND FIBER REINFORCED SOFT CLAY COLUMNS

*A thesis submitted in partial fulfilment of the requirements for the award of the degree of*

**DOCTOR OF PHILOSOPHY**

*in*

**CIVIL ENGINEERING**

*by*

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# WARANGAL

## CERTIFICATE

This is to certify that the thesis entitled "**A STUDY ON GEOPOLYMER STABILIZED AND FIBER REINFORCED SOFT CLAY COLUMNS**" being submitted by **Mrs. V. Bhavita Chowdary** for the award of the degree of **DOCTOR OF PHILOSOPHY** to the Faculty of Engineering and Technology of **NATIONAL INSTITUTE OF TECHNOLOGY, WARANGAL** is a record of bonafide research work carried out by her under my supervision and it has not been submitted elsewhere for award of any degree.

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This Thesis entitled "**A STUDY ON GEOPOLYMER STABILIZED AND FIBER REINFORCED SOFT CLAY COLUMNS**" by **Mrs. V. Bhavita Chowdary** is approved for the degree of Doctor of Philosophy.

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# ABSTRACT

The soft clay deposits ($c_u$ < 25 kPa) occur along the coastlines and estuaries of several world nations including India. In view of the enormous economic activity along the coastlines, it is imperative to take up huge infrastructure building (such as transportation routes, ports and harbour structures, multi-storeyed structures, residential and industrial utilities, etc.) over these unsuitable deposits inevitably. The sustained research by various investigators across the globe enabled the engineering community to develop remedial techniques such as soil replacement, stone columns, preloading with vertical drains, electro osmosis and soil-lime or soil-cement piles by deep soil mixing in order to make these deposits viable for construction activity. Among these techniques, only deep mixing method could modify the ground within short time and the remaining techniques require considerable time periods before the expected level of improvement could be achieved. In view of this, for time-bound projects, deep mixing method becomes an inevitable choice. This technique essentially consists of installing soil-binder columns by mixing binder (dry or wet) in the existing soft soil with the help of augers below ground surface. The soil-binder columns thus formed act as reinforcement for the soft ground improving its overall performance (increased bearing capacity and reduced settlements) to support low to medium load structures. The lime and cement have been traditionally used as binders for this purpose and currently, it is felt that the use of these traditional cementing materials with high carbon footprint are to be discouraged. Also, lower durability of these materials is reported. As an alternative to these conventional binding materials, geopolymer technology has been introduced and continuously being investigated for concrete making as well as for soil stabilization. The major difference in concrete making and soil stabilization using geopolymerisation arises from the fact that the entire geopolymerisation mechanism in soil stabilization, which is susceptible to aspects like silica and alumina supply, alkaline concentration, water content, etc., can be altered by the existence of soil. In view of the high degree of variability of soft clay deposits, especially the natural water content, several uncertainties arise, for which systematic investigations are inevitable in order to build confidence in the construction industry. This technology basically involves the preparation of an inorganic alumina silicate material formed by combining reference materials called ‘precursors’ possessing high amorphous silica (Si) and alumina (Al) with alkaline solutions called alkali activators or reactants of required concentration to get the target strength and durability requirements. Out of the various industrial by-products and wastes that are used as

precursors, fly ash, ground granulated blast furnace slag (GGBS) and metakaolin are most widely used. The general inference from these studies is that these precursors in the presence of $NaOH+Na_2SiO_3$ impart high strength and durability to the treated soil. Also, among the several hydroxide and silicate combinations rich in soluble metals like sodium (Na) and potassium (K) as potential alkaline medium, Sodium Hydroxide (NaOH) and Sodium Silicate ($Na_2SiO_3$) combination ($NaOH+Na_2SiO_3$) proved to be the most effective one and was broadly accepted by the cement and concrete industry researchers. However, contradictory results are also reported by previous researchers where geopolymers made with GGBS impart high early strength at ambient temperature curing, unlike geopolymers made of fly ash which require vigorous working environment and high temperatures of curing (60 °C– 200 °C) to initiate the reactions.

Further, it is indicated that the use of $Na_2SiO_3$ in the geopolymerisation process is suggested to be discouraged in view of its higher carbon footprint during its manufacture and transportation. As per this, some researchers have attempted to use lower alkali concentrations in their studies. However, the use of geopolymers at higher binder contents with high alkali concentrations becomes inevitable to satisfy the target UCS ranging from 1034 kPa to 4137 kPa of soil-binder columns required for DSM treatment of soft soils for wide range of applications. A detailed laboratory testing is taken up to understand the strength behaviour of soil-geopolymers using $NaOH$ alone as alkali activator in the present work along with GGBS as precursor in view of its higher strength gain at ambient temperature. As the soil-geopolymer mixes become brittle at high alkali concentrations and binder contents, polypropylene fibers are incorporated into the mixes to improve their ductility as suggested by the researchers. Also, considering the case of intrusion or extrusion of water (moisture fluctuations) from the soil surrounding the soil-geopolymer columns, the durability (against wetting and drying) of the soil-geopolymer mix specimens needs to be studied. Although fibers do not influence the changes in the hardened soil-geopolymer matrix due to wetting and drying, their role is still crucial in arresting the propagation of micro cracks formed during wetting and drying cycles, thus reducing the mass loss and volume change.

The scope of the present study is to synthesize an appropriate geopolymer (GP) binder with GGBS and $NaOH$ reinforced with polypropylene (PP) fibers to stabilize a highly plastic soft clay at high water contents (around liquid limit) and testing its efficacy with respect to strength and durability characteristics of the stabilized soft clay in deep mixing applications. The objectives of the present research work are kept as follows:

1. To study the strength aspects of geopolymer stabilized soft clay with $NaOH$ as a sole alkali at higher concentrations and GGBS as binder at its higher contents by performing laboratory tests by varying the mix proportions.

2. To assess the effect of polypropylene fiber inclusion on the strength and stiffness of soil-geopolymer mix specimens by varying fiber dosages.

3. To perform durability studies on the soil-geopolymer mix specimens of suitable proportions with and without fiber reinforcement and assess their mass loss, volume change and residual strength when subjected to wetting and drying cycles.

4. To estimate the load capacity of model soft clay bed reinforced with end bearing and floating soil-geopolymer columns with and without polypropylene fiber inclusion.

The experimental investigations are planned in line with the above objectives to determine the strength in terms of unconfined compressive strength (UCS) and flexural strength for different soil - geopolymer mix specimens with and without fiber reinfocement cured at ambient temperature for 3, 7, 14 and 28 days. The results from these tests were compared with that of the soil-cement specimens. The microstructure of selected treated soil samples after the strength tests was studied with the help of Scanning Electron Microscopy (SEM) and EDAX to understand the mechanism of strength improvement. The selected soil-geopolymer mix specimens with and without fiber reinforcement were subjected to durability test against wetting and drying cycles after 28 days of curing. The load tests on model soft clay bed with end bearing and floating soil – geopolymer columns, without and with fiber reinforcement were carried out to understand the load carrying capacity under axial loading.

From the present study it is understood that the specimens treated with GP showed higher UCS values compared to cement-treated specimens for the same dosage, and this may be due to the combined effect of pozzolanic and geopolymeric reactions of GP. To meet the target strength requirement for DSM applications, a binder dosage of greater than or equal to 20% and A/B ratio of greater than or equal to 0.75 are required. With increase in initial soil moisture content (higher than liquid limit), the strength of the treated specimens under unconfined compression and flexure is reduced thus requiring higher binder dosage to meet the DSM requirements at higher water contents. Out of the various combinations of the mixes tried, the geopolymer treated soil mixes with binder content of 30% and A/B ratio of 0.75 reinforced with 1% PP fibers by dry weight of soil could satisfy the strength and durability requirements

and hence found to be the optimum mix combination for deep soil mixing applications for soils with liquid limit in the range about 68%. For end bearing columns condition with any area ratio, the fiber reinforcement has shown improved load-deformation behavior as compared to the unreinforced system. For floating columns condition, the soil-geopolymer column reinforced soil bed has shown a block failure pattern and hence, the effect of high column strength and fiber reinforcement has insignificant effect on its load capacity. To attain a particular UCS, the Geopolymer stabilisation with GGBS and $NaOH$ is found to be economical compared to the Cement stabilisation as per the prevailing market rates.

**Key words:** Soft clays; Deep Soil Mixing; Geopolymer stabilization; Ground granulated blast furnace slag; Polypropylene fibers; Column studies.

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## List of Notations and Abbreviations

| | |
|---|---|
| $CO_2$ | Carbon dioxide |
| Si | Silica |
| Al | Alumina |
| NaOH | Sodium hydroxide |
| $Na_2SiO_3$ | Sodium silicate |
| Na | Sodium |
| K | Potassium |
| $E$ | Modulus of elasticity or Young's modulus |
| $E_{50}$ | Stiffness or Secant modulus of mix specimens |
| $a_p$ or $A_r$ or $\alpha$ | Improvement area ratio |
| $A_t$ | Net area of deep mixing |
| $A$ | Total area to be improved |
| $w_c$/C or $w$/C | water/cement ratio |
| M | Alkali metal cation such as Na, K, or Ca |
| $n$ | Degree of polycondensation |
| $z$ | Silicon to aluminum (Si:Al) ratio (usually 1, 2, or 3) |
| $w$ | Molar water amount |
| $R_{fcs}$ | Ratio of the flexural-compression strength |
| $q_{ult}$ | Ultimate bearing capacity |
| $q_{uc}$ | Unconfined compressive strength |
| $q_{us}$ | Unconfined compressive strength of the surrounding soil |
| $c_u$ | Undrained shear strength |
| $c_{uc}$ | Undrained shear strength of the column |
| $c_{us}$ | Undrained shear strength of the soft soil |
| $H_c$, $B$, $L$ | Height, width, and length of the DSM column group |
| $c_{av}$ | Average shear strength along the assumed failure surface |
| $p_t$ | Stress on the soil-cement columns |
| $N_c$ | Bearing capacity factor for shallow foundation on clay |

| | |
|---|---|
| $w_L$ | Soil water content at Liquid Limit |
| $EA_C$ | Energy Absorption in Compression |
| TI or $I_T$ | Toughness Index |
| $UCS_{28}$ | UCS after 28 days curing period |
| $UCS_D$ | UCS after any other specific curing period |
| $UCS_f$ | UCS of the specimens at any fiber dosage from 0.25 – 1.0 % |
| $UCS_0$ | UCS of GP specimens without fibers |
| $F_f$ | Flexural strength at any fiber dosage from 0.25 – 1.0 % |
| $F_0$ | Flexural strength of GP specimens without fibers |
| DSM | Deep Soil Mixing |
| FA | Fly ash |
| GGBS | Ground Granulated Blast Furnace Slag |
| S | Slag |
| OPC | Ordinary Portland Cement |
| AAMs | Alkali activated materials |
| AACs | Alkali-activated cements |
| AAB | Alkali activated binder |
| CS | Cemented soil |
| BCS | Black cotton soil |
| FRSC | Fiber Reinforced Soil Cement |
| FRCMS | Fiber reinforced cement mortar soil |
| PPF or PP Fibers | Polypropylene Fibers |
| GF | Glass fibers |
| UCS | Unconfined Compressive Strength |
| FS | Flexural Strength |
| LL | Liquid limit |
| PL | Plastic limit |
| CH | Clay with high plasticity |
| CL | Clay with low plasticity |
| kPa | Kilo Pascals |
| MPa | Mega Pascals |

| | |
|---|---|
| psi | Pounds per square inch |
| GJ | GigaJoules |
| kWh | Kilo Watts |
| PS | poly(sialate) with Si:Al ratio of 1 |
| PSS | poly(sialate-siloxo) with Si:Al ratio of 2 |
| PSDS | poly(sialate-disiloxo) with Si:Al ratio of 3 |
| L | Liquid alkaline medium |
| SEM | Scanning Electron Microscopy |
| EDS | Energy Dispersive X-ray Spectroscopy |
| XRD | X-ray Diffraction |
| NASH | Sodium Aluminosilicate Hydrate |
| CSH | Calcium Silicate Hydrate |
| CASH | Calcium Aluminosilicate Hydrate |
| OMC | Optimum Moisture Content |
| MDD | Maximum Dry Density |
| $G_s$ | Specific gravity of soil |
| COV | Coefficient of variation |
| A/B ratio | Alkali/Binder ratio |
| $R$ | Modulus of Rupture |
| w-d | wetting-drying |
| S | Soil with Soft consistency |
| V | Soil with Very Soft consistency |
| L | Soil with Liquid consistency |
| GP | Geopolymer / unreinforced soil-geopolymer |
| FGP | Fiber Reinforced Soil–geopolymer |

# Chapter – 1
# Introduction

## 1.1. General

This chapter includes a brief introduction about the background and need for the present study. It also contains scope and objectives of the present work. The chapter ends with the organization of the thesis.

## 1.2. Background of the study

The presence of soft clay deposits can be observed in the coastal areas and estuaries of various countries worldwide, including India (Kitazume and Terashi, 2013). These deposits exhibit a notable abundance of natural water, leading to significant compressibility and minimal shear strength (< 25 kPa), rendering them unsuitable for civil engineering projects (Broms, 1991; Porbaha, 1998, Lin, C. et al., 2014; Disu et al., 2021). Significant infrastructure development, including transportation routes, ports and harbour structures, multi-storied buildings, residential and industrial utilities, etc., must be built over these unsuitable deposits due to the significant economic activity along the coastlines. The extensive research conducted by several researchers worldwide (Hughes and Withers, 1974; Sridharan, 1990; Broms, 1991; Ando et al., 1995; Porbaha, 1998; Zheng and Qin, 2003; Puppala and Musenda, 2007; Hassan, 2009; Ornek et al., 2012; Han, 2014; Sol-Sánchez et al., 2016) has allowed the engineering community to develop remedial procedures to make these deposits viable for construction purposes.

These procedures comprise of techniques like soil replacement, stone columns, preloading with vertical drains, electro osmosis, and soil-lime or soil-cement columns formed by deep soil mixing. The applicability of these techniques varies depending upon the specific ground conditions. Among the various techniques available, only the deep soil mixing method has the capability to modify the ground in a relatively short period of time. The remaining methods necessitate substantial time periods to attain the anticipated level of improvement (Topolnicki, 2004). Considering this, for projects with strict time constraints, the deep mixing method becomes an unavoidable option. This method primarily involves the installation of soil-binder columns by combining binder (either in a dry or wet form) with the pre-existing soft soil using augers below the ground surface. The formation of soil-binder columns serves as a reinforcement for the soft ground, enhancing its overall performance by increasing its bearing capacity and reducing settlements. This reinforcement action enables the ground to support structures with low to medium loads (Porbaha, 1998; Kitazume and Terashi, 2013; Puppala et al., 2017).

Although cement and lime have historically been employed as binders for this purpose, it is considered that the production of these traditional cementing materials releases a significant quantity of carbon dioxide ($CO_2$) into the atmosphere (Zhang et al., 2013; Chen, H. et al., 2020; Disu et al., 2021). Additionally, there have been reports of reduced durability for these materials (Arulrajah et al., 2018). As a result, geopolymer technology has been developed and is still being researched as a possible replacement for cement and lime associated with high amounts of $CO_2$ generation. Geopolymer concrete was created as a substitute for traditional cement concrete and has been promoted for its broad range of applications (Provis and Van Deventer, 2009; Deb and Sarker, 2017). Similarly, various researchers (Cristelo et al., 2012; Liu et al., 2016; Phummiphan et al., 2016) have been focusing on various aspects of soil stabilisation with geopolymers, and as a result, this technique was extended to soft clay modification also. The primary distinction between concrete production and soil stabilisation through geopolymerization lies in the fact that the entire process of geopolymerization, which is influenced by the factors such as the availability of silica and alumina, alkaline concentration, water content, etc., can be significantly affected by the in-situ soil condition (Ayub and Khan, 2023). Given the significant heterogeneity of soft clay deposits, particularly in terms of natural water content, numerous uncertainties arise. Therefore, it is essential to conduct systematic investigations to instil confidence in the construction sector.

Few researchers (Sargent et al., 2017; Yaghoubi et al., 2019) have been focusing on the durability and strength characteristics of soils stabilised with geopolymer. Fundamentally, this technology entails the synthesis of an inorganic alumina silicate material by combining precursors, which are reference materials abundant in amorphous silica (Si) and alumina (Al), with alkaline solutions known as alkali activators or reactants in the concentrations necessary to achieve the desired strength and durability (Davidovits, 2002). The most commonly used precursors are fly ash, ground granulated blast furnace slag (GGBS) and metakaolin (Arulrajah et al. 2018; Pourakbar and Huat 2017) among other industrial residues and wastes (Huang et al., 2021). In light of the various studies, it can be generally inferred that when these precursors are combined with $NaOH$ and $Na_2SiO_3$, they enhance the treated soil's strength and durability. Furthermore, among the various combinations of hydroxides and silicates that are abundant in soluble metals such as potassium (K) and sodium (Na) that were considered as potential alkaline media, the combination of sodium hydroxide ($NaOH$) and sodium silicate ($Na_2SiO_3$), i.e., $NaOH$ +$Na_2SiO_3$, demonstrated the highest efficacy and was widely endorsed by researchers in the cement and concrete industry (Duxson et al., 2007). In contrast to fly ash geopolymers, which necessitate a high curing temperature (60 – 200 °C) and a vigorous working environment to initiate the reactions, geopolymers produced with GGBS impart high early strength during curing at ambient temperatures (Davidovits, 2008). Previous researchers have also reported contradictory findings (Sargent et al., 2017; Yaghoubi et al., 2019). Furthermore, it is imperative to minimise the utilisation of $Na_2SiO_3$ in order to drastically reduce global warming, given that its production and transportation generate substantial quantities of $CO_2$, which highlights the critical necessity for additional research on geopolymers utilising $NaOH$ exclusively as an alkaline medium (Reddy and Murugan, 2020).

Further, to achieve the desired UCS of 1034 kPa to 4137 kPa (Bruce, 2001; Puppala et al., 2008) for a wide variety of DSM applications, the use of high binder contents (>10%) in soil-binder columns is essential that raises the material's brittleness, making it susceptible to abrupt brittle failure under compression, tension, shear, bending, and rotation (Broms, 1999; Filz and Navin, 2006; Han, J., 2014). Some researchers are currently examining the application of fiber reinforcement to stabilised soil columns as a means to reduce the expansion of micro-cracks without compromising the columns' strength (Zhang, M. X., et al., 2008; Sukontasukkul, P., & Jamsawang, P., 2012; Correia et al., 2015; Yi, W. J., et al., 2018). However, additional research is necessary to validate these findings for DSM applications (Ruan,B., et al., 2021).

## 1.3. Research gaps and need for the present work

Previous literature indicates that most of the researchers have commonly utilised $NaOH$ and $Na_2SiO_3$ as alkalis in different combinations to produce geopolymers for soil stabilisation. However, there is a need to mitigate the use of $Na_2SiO_3$ due to significant $CO_2$ emissions associated with its production and transportation. Many researchers have examined the strength characteristics of geopolymer stabilised soils with lower alkali concentrations to understand the patterns in strength development. However, it is necessary to employ geopolymers with increased amounts of binder and higher alkali concentrations to achieve the desired unconfined compressive strength (UCS) of soil-binder columns, which should range from 1034 kPa to 4137 kPa, i.e., 150 psi to 600 psi (Bruce, 2001; Puppala et al., 2008) for a wide range of DSM applications. This is essential for the effective treatment of soft soils through deep soil mixing (DSM) in various applications. There are very few studies available that address the patterns of strength gain and the corresponding behaviour that need to be understood at such higher dosages. To account for the various soft soil conditions in the field, a comprehensive laboratory testing has been carried out to understand the strength characteristics of soil-geopolymers using $NaOH$ as the sole alkali activator in the current study. Geopolymers developed with GGBS have demonstrated the ability to provide significant early strength at ambient curing temperatures, unlike geopolymers made with flyash which require high temperature curing. Therefore, GGBS is employed as a precursor in the preparation of geopolymer mixes in this work. The researchers indicated that the inclusion of fibers into soil-geopolymer mixes can enhance their ductility, particularly when the mixes become brittle due to higher alkali concentrations and binder contents. Therefore, due to their low modulus, polypropylene (PP) fibers are utilized in the current investigation to examine their impact on the strength and stiffness of the soil-geopolymer blends at different fiber dosages. Furthermore, it is necessary to investigate the durability of the soil-geopolymer mix specimens in relation to the intrusion or extrusion of water (moisture variations) from the surrounding soil. While fibers do not directly affect changes in the hardened soil-geopolymer matrix caused by wetting and drying, they play an essential role in arresting the propagation of micro cracks that occur during these cycles. As a result, they help to minimize the mass loss and volume change. Therefore, it is necessary to investigate the resistance to wetting and drying of the soil-geopolymer mix specimens reinforced with PP fiber, in addition to the specimens without fiber reinforcement.

## 1.4. Scope and Objectives of the study

The scope of this study is to synthesize an appropriate geopolymer binder using GGBS and $NaOH$, along with polypropylene fibers, to stabilize a highly plastic soft clay with high water content (close to the liquid limit). The efficacy of this developed geopolymer binder will be tested by evaluating the strength and durability properties of the stabilized soft clay in deep mixing applications.

The objectives of the current research work are as follows:

1. To study the strength aspects of geopolymer stabilized soft clay with only $NaOH$ as an alkali activator at higher concentrations and GGBS as binder at its higher contents and compare them with those obtained by cement stabilization.
2. To assess the effect of polypropylene fiber inclusion on the strength and stiffness of soil-geopolymer mix specimens by varying fiber dosages.
3. To perform durability studies on the soil-geopolymer mix specimens of suitable proportions with and without fiber reinforcement and assess their mass loss, volume change and residual strength when subjected to twelve wetting and drying cycles.
4. To estimate the bearing capacity of model soft clay bed reinforced with end bearing and floating soil-geopolymer columns with and without polypropylene fiber inclusion.

## 1.5. Organization of the thesis

This thesis is the final outcome of this research work and is divided into seven chapters. This section presents a brief description of the organization and contents of this dissertation.

Chapter 1 introduces the research work with some background illuminating the necessity and relevance of this work. It also defines the scope and objectives of the thesis.

Chapter 2 provides a review of existing literature on soft soils, conventional soil stabilization techniques and their limitations, description of deep soil mixing technique, geopolymers and their characteristics, their use as soil stabilizers in the past and fibers as reinforcement in stabilized soils.

Chapter 3 details the materials used and experimental methodology adopted to accomplish the research objectives of this study. This chapter elaborates the test procedures used to conduct engineering characterization tests for the determination of the strength and durability characteristics of unreinforced and polypropylene fiber reinforced geopolymer stabilized soft clay specimens. It also describes the methodology adopted and stepwise procedure for the preparation of model soft clay bed and column installation for the laboratory scale model tests conducted in this study.

Chapter 4 presents the results of compressive strength and flexural strength tests on cement-treated and geopolymer-treated soil specimens and analyses improvement of strength and stress-strain behaviour of geopolymer-treated soil specimens compared to the cement-treated ones. Additionally, economical aspects of stabilization of soft soils with geopolymer are also discussed in this chapter. The influence of polypropylene fiber reinforcement on the stress-strain behaviour, strength and ductility of soft clay specimens stabilized with optimized geopolymer mixes is also discussed. It also elucidates the durability characteristics of unreinforced and polypropylene fiber reinforced geopolymer stabilized soft clay against wetting and drying cycles. This chapter also discusses the results of small-scale laboratory model study conducted on a model soft clay bed reinforced with soil-cement and soil-geopolymer columns with and without polypropylene fiber reinforcement and attempts to validate the experimental findings with the laboratory model tank study.

Chapter 5 summarizes the major findings and conclusions from this study, in addition to addressing future research needs and recommendations.

# Chapter – 2
# Literature Review

## 2.1. Introduction

This chapter presents a comprehensive overview of soft soil and the most prevalent techniques for deep soft soil stabilization. The fundamental principles and mechanisms and limitations of the currently available techniques are critically reviewed in order to find the scope for taking up the relevant research work in the emerging stabilization techniques.

## 2.2. Difficult soils

For a variety of socio-economic considerations such as, construction of various infrastructure projects like transport facilities (railway lines, highways, etc.), multi-storeyed structures, bridge abutments, reservoirs, hydro and thermal power plants, etc., foundation soil is of utmost importance. There are numerous types of soils ranging from hard and dense large fragment rocks, gravel, sand, silt and clay to soft organic deposits. However, climate, organisms, geography, parent materials, and the time of weathering and erosion are only a few of the many factors that influence the type and behaviour of soil (Mitchell and Soga, 2005). The existence of a uniform language for soil description and identification is critical. Several systems for soil classification have been used depending on the purpose of soil utilization that specify symbol, group and recommended name for each soil type depending on the soil grain size, percentage of fines and liquid limit (Bunga et al., 2011; Budhu, 2010; Hartemink, 2015). Though most of the soil types are highly advantageous as a foundation material for the construction of structures, there are few soil types which are proven to be problematic for such purposes. The collapsible soils (quick clays, loose sands in a saturated state, unsaturated primarily granular soils with clay particles at intergranular contacts, chemically weathered rocks) (Murthy, V.N.S., 2003), dispersive soils with unstable structure (Mitchell and Soga, 2005) and expansive soils (Wang, Y. et al., 2014; Jha and Sivapullaiah, 2016; Dang et al., 2016) are the most common types of such soils. Soft clay deposits with shear strength less than 25 kPa are referred as soft clays which are the focus of the present work.

### 2.2.1. Origin and distribution of soft clays

Soft clay soils are primarily found in estuarine channels or specific areas near the sea. Soft clay soils were typically formed during the Holocene Epoch, spanning the last 11,700 years after the conclusion of the major glacial period in the Pleistocene Epoch. The Pleistocene was characterised by a period of reduced sea levels worldwide and extensive ice in the northern hemisphere, as well as in certain regions of the southern hemisphere. During the Pleistocene Epoch, as sea levels decreased, rivers eroded the coastal materials, creating channels and bays of all shapes and widths. During the rapid warming after the last glacial maximum, sea levels increased quickly, leading to the deposition of sediments in the channels and bays formed in the coastal area. Small changes in water level caused newly deposited material to be uncovered, dried off, eroded again, and redeposited (J. Ameratunga et al., 2021).

The particle size of material deposited in a body of water is determined by the energy from the currents and waves in that environment. When varied depositional settings are combined with changing sea levels, it is clear that there will be significant variation in the distribution of material types. Soft clay soils are created by the deposition of extremely small soil particles in water. The transport capacity of flowing water for a specific particle is determined by the water velocity, the level of frictional forces between particles, and between particles and the bed of the water body. Clay particles are tiny (less than 0.002 mm), stay suspended for extended periods, and are easily carried by sluggish water. Clay particles need to be suspended in an environment with minimal energy for deposition to take place. These habitats consist of deeper coastal waters shielded from near shore activities, quiet places within a river system, or lakes. Each depositional setting might see rapid changes in conditions compared to the lengthy timespan needed to create substantial soft clay deposits.

The spatial distribution of clay in a certain region is typically intricate, with clay zones interspersed with sand, peat, or other elements. Geologists and coastal scientists identify many environments where clay deposition can take place. These environments include tide dominated estuaries, shallow marine environments, lagoon barrier and strand plain systems, some parts of river systems, sheltered bays, and freshwater and saltwater lakes (J. Ameratunga et al., 2021).

The main regions with soft clay are the Nordic countries (except Denmark), Canada and northern United States (Chicago and Boston), where deposits of soft glacial and postglacial clays are often more than 100 m thick. Clays in these regions often have a high sensitivity and a low shear strength; they are called quick in the Scandinavian countries when the sensitivity ratio, St, (i.e., the ratio of the undisturbed and remoulded shear strengths) exceeds 50. Other areas where deep deposits of soft clay occur are Mexico City, with its volcanic clay, and the deltas of the major rivers of the world, such as the Nile, Mississippi, Rhine around Rotterdam in the Netherlands, Elbe around Hamburg in Germany, Neva around Leningrad in USSR, Eufrates and Tigris Rivers in Iraq, Ganges around Calcutta in India, and the Yangtze River around Shanghai in China (Brand and Brenner, 1981).

India has long coastal region having complex deep seated thick deposits of soft clay extended to a depth of 10 m to 30 m which undergoes consolidation settlements of high magnitudes after the application of permanent loads from facilities constructed on it (S.P. Bhosle et al., 2015). Soft clay distribution and origins in India have been analysed in different areas. (Prithviraj, 1990) reported that the inner shelf sediments in central Kerala have high levels of kaolinite and montmorillonite, which are presumably derived from the aluminous laterites found along the coast. (Reddy, 1989) stated that kaolinite and chlorite in the central eastern continental shelf originate from coastal red sediments and Precambrian khondalites. (Rao, V. P. et al., 1995) pinpointed three main origins of sediments in the western continental edge, originating from a collection, rich in smectite and kaolinite from the Gneissic Province. (Nair, R. R. et al., 1982) identified four distinct clay-mineral regions on the western continental shelf, emphasising the significance of source-rock impact above physical transportation. The results indicate that soft clays in India are affected by both terrestrial and marine activities.

### 2.2.2. Problems associated with soft clays

Soft clays differ significantly from other weak geomaterials like loose sands, which necessitate ground improvement. The soft clay improvement has emerged as a significant geotechnical challenge in civil engineering over the past thirty years. Civil engineers, planners, architects, consultants, and contractors are all now knowledgeable with the characteristics of soft clays and the potential hazards involved in construction projects in such locations.

Two primary considerations of soft clays are strength and compressibility. These materials could cause excessive settlements and instability under imposed loads if not built incorrectly. Generally, the word "soft clay" is linked to the cohesion of clay. Clays are often classified according to their undrained shear strength, $c_u$, which is the standard method used by most Standards, such as the Canadian Geotechnical Society, 2006 and AS1726, 2017.

Soft clays have low permeability (less than $10^{-7}$ cm/sec), high compressibility, and low undrained shear strength (varies with depth) of 5 to 20 kPa. Additionally, it possesses the characteristics like low modulus of elasticity, E (< 500 kPa), primarily fine-grained, high natural water content (possibly higher than the liquid limit) and time-sensitive strength development. Thus, they pose numerous difficulties, including insufficient bearing capacity, significant deformation and instability arise while building on these soils. Therefore, such soft ground has to be improved before the construction of any structure over it.

References to ancient ground improvement techniques for soft clay engineering are mentioned by (WANG Yuan-zhan, et al., 2015) and (Barends, 2011). Recent historical information from the last century is available in (Brand and Brenner, 1981) and (Han, 2015). Significant progress has been made in the last forty years through the improvement of existing procedures and the creation of new innovative approaches. The achievements were motivated by necessity, as engineers had to improve deleterious grounds due to the scarcity of better ones. They are often low-lying and need significant filling to elevate them above flood levels. Furthermore, there is a global trend towards expanding land area through reclamation, resulting in the creation of additional land for construction. However, this process often leads to the presence of soft, compressible soils that can cause settlement and stability issues.

The direct approach to address this issue involves excavating the soft soil and replacing it with a substantial amount of appropriate fill material. However, this approach is laborious and leads to a significant depletion of natural resources, consequently giving rise to environmental issues. Also, incomplete replacement of soil may lead to excessive differential settlements in the future and accordingly, this approach is limited to shallow depths of soft clays. Hence, altering the engineering characteristics of the soft soil in its natural location proves to be a more effective approach. Consequently, it is imperative to adopt technical measures to enhance the soft soil properties (Kitazume & Terashi, 2013).

### 2.2.3. Deep stabilization techniques in soft clays

Listing all ground improvement methods suitable for soft clay is challenging due to the vast quantity and numerous variations. SHRP2 2014 enlists and defines the ground improvement techniques suitable for all soil types. Table 2.1 illustrates various methods for enhancing soft clays, some of which share similarities in materials and construction procedures with modest variations.

**Table 2.1. Ground improvement methods applicable to soft clays as per SHRP2 2014 (Schaefer, V. R., & Berg, R. R. 2014)**

| | | |
|---|---|---|
| Aggregate Columns | Excavation and replacement | Lightweight fill |
| Bio-Treatment for subgrade stabilization (Emerging technology) | Geocell Confinement in Pavement Systems | Mass Mixing Methods |
| Chemical stabilization of subgrades and bases | Geosynthetic reinforced construction platforms | Mechanical stabilization of subgrades and bases |
| Column-supported embankments | Geosynthetic reinforced embankments | Micropiles |
| Combined soil stabilization with vertical columns | Geosynthetic separation in pavement systems | Onsite use of recycled pavement materials |
| Compaction grouting | Geotextile encased columns | Preloading and prefabricated vertical drains (PVDs) |
| Continuous flight auger (CFA) piles | Hydraulic fill with Geocomposite and Vacuum consolidation | Sand compaction piles |
| Deep mixing methods | Injected lightweight foam fill | Vacuum preloading with and without PVDs |
| Electro-Osmosis | Jet grouting | Vibro-Concrete columns |

Many of these techniques have been proposed by several other researchers also and implemented globally, based on site-specific conditions, for mitigating the challenges posed by soft clays characterized by high compressibility and low shear strength. The hydraulic techniques of preloading and electro-osmotic drainage are primarily designed on the principles of consolidation. Though they are efficient in improving the stability of the soft ground and reducing settlements, the wide use of these methods is limited due to the various concerns such as requirement of sufficient time for consolidation to occur, application and removal of large quantities of surcharge loads, requirement of additional measures to prevent clogging of the vertical drains and low effectiveness in highly organic and sensitive clays. Stone columns are constructed in soft clays to provide vertical drainage and soil reinforcement. Though they enhance soil stability by improving load bearing capacity and by reducing settlement, they possess certain limitations such as higher cost compared to other methods and consumption of large quantities of natural resources (quarried stone). The practice of chemical stabilisation is commonly adopted for the purpose of stabilising soils to shallow depths, as it involves low water content mixing and usage of large amounts of chemical admixtures like lime and cement. Thus, its application is limited to shallow stabilization of subgrade or foundation soil for various structures such as roadways, airfields, and other similar constructions.

From past several decades (since 1954), the Deep Soil Mixing (DSM) technique is being widely used in practice to improve the overall performance of soft ground with improved bearing capacity and reduced settlements. The ease of operation, less time consumption, wide range of applications, arrangement of columns in several patterns, make this technique advantageous over the other deep stabilization techniques (Bruce, 2000). Figure 2.1 shows the superior performance of deep mixing method compared to other soft soil improvement techniques based on environment friendliness, high economy and reliability (Ando et al., 1995).

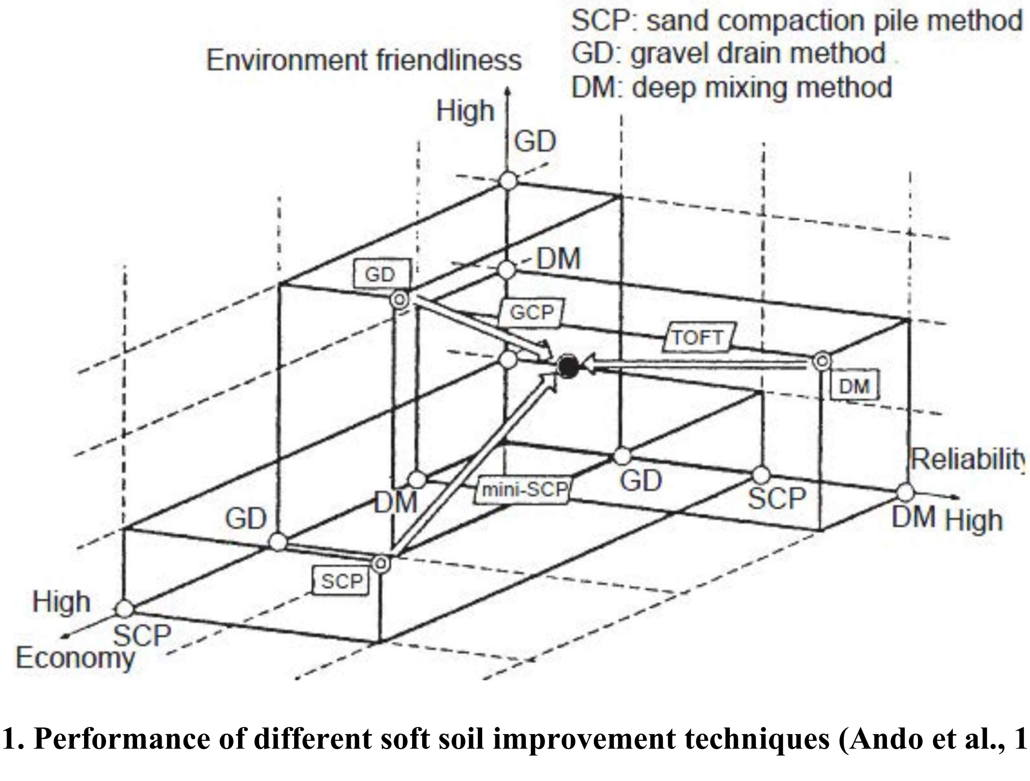



**Fig. 2.1. Performance of different soft soil improvement techniques (Ando et al., 1995)**

## 2.3. Deep Soil Mixing method

This technique consists of in-situ mixing of soil with cementitious binders, commonly lime, cement and their blends, to form soil-binder columns (Figures 2.2 and 2.3) at a particular position in the site with the help of augers until required depth is reached. The commonly used augers for DSM are shown in Figure 2.4.

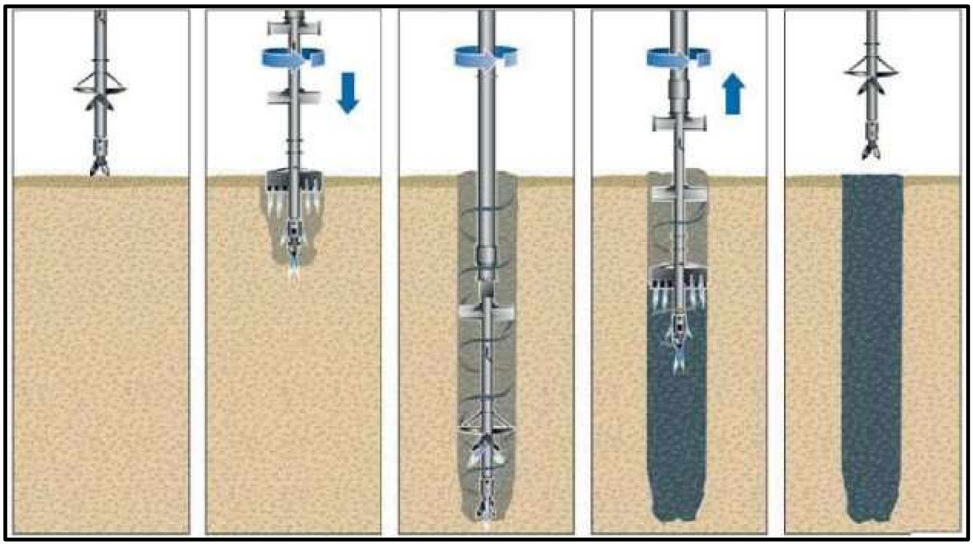


**Fig. 2.2. Deep mixing column installation process (Liebherr, 2012)**

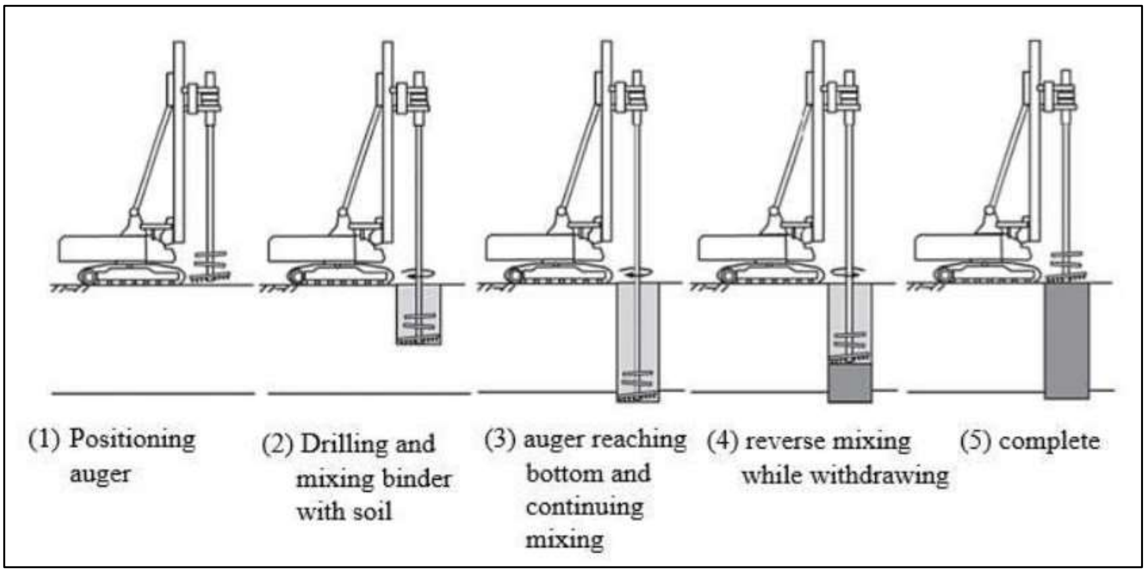



**Fig. 2.3. Deep mixing column installation process (Keller, 2013)**

Depending on the purpose of DSM columns, site conditions and costs of soil improvement, several patterns of installation of columns, as shown in Figure 2.5, are used to achieve the desired result by using spaced or overlapped and single or combined or group columns. Improvement area ratio (Equation 2.1), $a_p$ or $A_r$ or $\alpha$, is used to compare various column patterns in terms of the treated area (Topolnicki, 2004).

$$a_p = A_t/A = \text{net area of deep mixing / respective total area} \qquad \text{----- Eq. 2.1}$$

(a) (b) (c)

**Fig. 2.4. Dry mixing augers (a) standard, (b) modified (Larsson, 2005) and Wet mixing auger (c) DSM auger (Keller, 2013)**

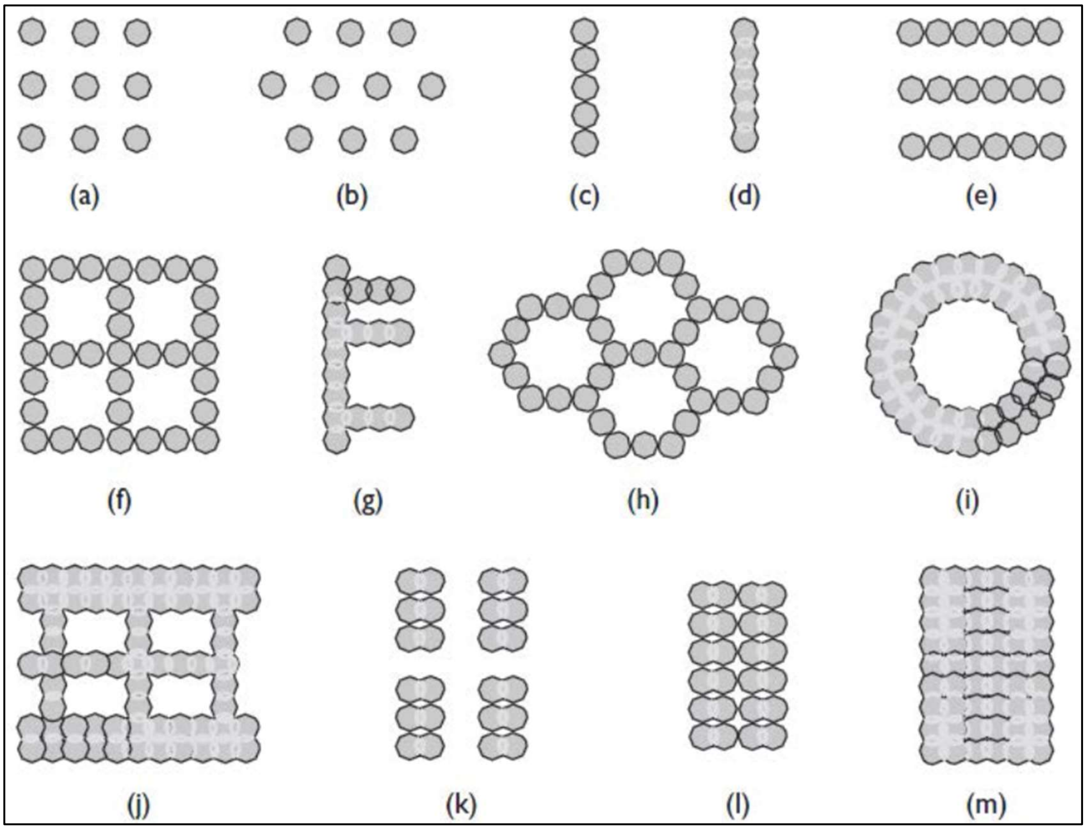



**Fig. 2.5. Patterns of DSM (a) column-type (square arrangement), (b) column-type (triangular arrangement), (c) tangent wall, (d) overlapped wall, (e) tangent walls, (f) tangent grid, (g) overlapped wall with buttresses, (h) tangent cells, (i) ring, (j) lattice, (k) group columns, (l) group columns in contact, (m) block pattern (Topolnicki, 2004)**

Extensive research has been conducted on the implementation of the DSM technique in Southeast Asia, specifically in Japan and Sweden since its introduction in the 1970s. This deep ground improvement method has been the subject of research and the development of guidelines (Bruce, 2001; Puppala et al., 2008). Several of these previous research studies, conducted until the late 1990s, were not widely disseminated (Porbaha, 1998). The DSM method, originally referred to as the "deep lime mixing" method due to the use of lime as the binder, transitioned to using cement in slurry form in the mid-1970s, leading to the development of the name "wet method" of deep mixing, whereas the addition of binder in powder form to the soil and the introduction of the term "dry method" of deep mixing occurred in the early 1980s (Porbaha, 1998). Okumura and Terashi (1975) employed the deep lime mixing method to enhance the quality of soft coastal clays in Japan. The significant progress in the development of mixing techniques and mix designs took place in the mid-1980s (Porbaha, 1998).

### 2.3.1. Lime and cement as binders for DSM

There has been a great deal of research in DSM technology and the most noteworthy studies are discussed below.

Uddin et al. (1997) performed a series of tests on a soft clay with a high-water content that had been treated with Portland cement. The purpose of the study was to examine the strength and compressibility characteristics of the treated soil. The soil exhibited a high plasticity, characterised by a clayey texture, with liquid limit (LL) and plastic limit (PL) values of 103% and 43%, respectively. The soil was treated with cement contents of up to 40% and a cement slurry with a water/cement ratio of 0.25. The original water content of the soil was around 80% (≈ 0.8 LL). The specimens underwent a curing period of 0 – 24 weeks before conducting various tests, such as unconfined compressive strength (UCS), consolidation, and triaxial testing. The study findings indicate that the most effective cement content for enhancing strength is between 10 to 20%, while for improving compressibility characteristics, it is within the range of 10 to 25%. The optimal cement content for the latter is determined to be 15%. Furthermore, it was determined that the ideal duration for curing was 28 days in every instance.

Miura,N. et al. (2001) examined the alterations in the engineering properties of a silty clay with a significant amount of water when treated with cement for application in the DSM process. The soil exhibited a liquid limit (LL) of 120% and a plastic limit (PL) of 57%. They performed oedometer, unconfined compressive strength (UCS), and triaxial tests on samples of various soil-cement combinations cured for durations of 7 and 28 days. The water content ranged from 120% to 250% (about 1.0 – 2.0 LL), whereas the cement content fell between 8% and 33% (based on the dry mass of soil). They found that the clay-water/cement ratio ($w_c$/C) had a significant impact on the behaviour of the mixtures in all the tests. According to the analysis, decreased water to cement ratio ($w_c$/C) values between 7.5 and 15.0 led to increased yield stress, unconfined compressive strength (UCS), and shear strength. In addition, mixes with high $w_c$/C ratio and high initial water content (250%) exhibited low shear modulus and significant volumetric strain.

Horpibulsuk et al. (2003, 2004 and 2005) conducted research on the engineering properties of a clay with high water content that was treated with cement for the purpose of deep soil mixing (DSM). They performed a range of laboratory and field tests. The soil was CH clay, with LL of 120% and PL of 57%. The study considered various water contents ranging from 0.7 to 2.0 LL, cement concentrations ranging from 7.5% to 33%, *w*/C ratios ranging from 0.6 to 15, curing periods ranging from 3 to 28 days, and mixing factors including penetration and slurry injection rate. While all the elements examined were shown to have a substantial impact on the strength and consolidation behaviour of the composite ground (consisting of soil and columns), it was determined that the water-to-cement ratio (*w*/C) was the primary and most crucial component. The study focused on developing models to accurately forecast the strength and compressibility of cement-treated clays. These models considered many parameters, with a particular emphasis on the *w*/C ratio. Furthermore, it is mentioned that there exists a correlation between the goal strength and the *w*/C ratio. This implies that if the total water content needs to be modified for reasons like mixing technique, wet mixing, or grouting, the correlation can be utilised to make appropriate adjustments to the cement content.

Lorenzo and Bergado (2003, 2004 and 2006) performed experiments in both laboratory and field settings to establish models for predicting the consolidation and strength characteristics of a deep cement mixed soft clay with a high-water content. The soil was a CH clay with LL of 103% and PL of 43%. The soil was remoulded to a soil moisture content of 0.8 – 2.0 LL before being combined with cement slurry at a water-to-cement ratio of 0.6. The cement percentage ranged from 5% to 20% of the dry soil, and the samples were subjected to curing for 7, 14, and 28 days. The test findings revealed that soil-cement mixes with water contents at 1.0 LL exhibited greater strength and reduced compressibility. The study found that the water content, cement content, and curing time were the key factors influencing the strength enhancement of high-water content clays treated with cement. A formula was developed to estimate the strength improvement by considering the ratio of void ratio after curing to cement content, which accounts for the combined effect of these parameters. It is reported that an increase in this ratio resulted in a drop in the strength and formation of negative excess pore water pressure, while increasing the ductility of the mixes. Moreover, the cement content influenced both the position and slope of the post-yield compression line in consolidation. Additionally, the after-curing void ratio controlled the pre-consolidation stress.

Jongpradist et al. (2010) and Bushra and Robinson (2013) performed unconfined compression strength (UCS) tests on clays with high water content that were blended with cement and/or fly ash (FA) to examine the effect of adding FA. Jongpradist et al. (2010) employed cement amounts ranging from 5% to 40%, FA contents ranging from 5% to 30%, and clay water content ranging from 1.1 to 1.9 LL. The study performed by Bushra and Robinson (2013) reported these values as 10–20%, 10–30%, and 0.8–1.8 LL respectively. The water-to-cement ratio ($w_c$/C) was determined to be a significant factor in enhancing strength in both trials, with the optimal water content being approximately 1.2 LL. While the strength and stiffness improved as the FA content increased in both trials, Bushra and Robinson (2013) observed that the mixtures exhibited a brittle behaviour and recommended an FA content of 20% as being effective.

Cong et al. (2014) performed a series of unconfined compressive strength (UCS) tests on a clay material that was treated with Portland cement and/or chemical additives for the purpose of stabilising it for use in deep soil mixing (DSM) applications. The additions consisted of sodium silicate and a combination of sodium hydroxide and calcium chloride in a 1:1 ratio. The soil was a CL clay, characterised by a LL of 42% and PL of 24%. The cement content ranged from 10% to 80%, whereas the water content varied from 75% to 110% (about 1.8–2.6 LL), resulting in a $w_c$/C ratio of 1.38 – 7.50. The additives constituted up to 6% of the total content, and the samples were subjected to curing periods of 7, 28, 60, and 90 days. The inclusion of additives resulted in a rise in both the UCS and secant modulus ($E_{50}$) of the cement-treated soil. It was recommended to utilise these additives to minimise the amount of cement used, which offers environmental and financial benefits. Furthermore, the $w_c$/C ratio was found to play a significant influence on the engineering properties of the cement-treated soil mixes.

In their study, Pakbaz and Farzi (2015) examined the impact of two different mixing methods, wet and dry mixing, on the behaviour of soil treated with cement and/or lime and having a high-water content. The soil had LL of 130% and PL of 70%, and binder dosages ranging from 2% to 10% were utilised. The water content of the mixtures was adjusted to LL of the soil for both wet and dry mixed specimens. The samples were then cured for 7, 14, and 28 days. The findings indicated that wet mixing had a greater influence on the improvement of

UCS of cement-treated mixtures, while the lime-treated mixtures had a greater increase in the UCS when made using the dry mixing method. The same principle was applicable to cement-lime treated mixes, but to a lesser degree. In addition, the process of dry mixing resulted in increased elastic moduli for all treated mixes.

Jamsawang et al. (2016) employed a numerical model to examine a highway embankment that was reinforced with deep cement mixed columns. The settlement, stress concentration, excess pore water pressure, and lateral movement in field, which are caused by axial compression and lateral loading were examined and compared by using mechanical and hydraulic numerical models in conjunction with PLAXIS software. The soil in the field was a highly plastic clay (CH), with liquid limit (LL), plastic limit (PL), and field soil moisture content measuring approximately 100%, 30%, and 100% respectively. The columns had a diameter of 0.6 m and were spaced at 1.5 m from each other centre to centre. The slurry used had a $w_c$/C ratio of 1.5 and a cement dosage of 150 kg/m$^3$ of soil. The study found a strong correlation between the field observations and the model predictions. The stress concentration was found to be around 2.0, indicating that the majority of the imposed load was borne and transmitted to the more rigid layers below through the columns. Moreover, it was shown that the bending failure is a crucial factor that must be taken into consideration while conducting tests and designing DSM columns. Prior research has shown the significance of bending failure in deep columns. Larsson et al. (2012) highlighted the importance of considering bending failure in DSM columns. They achieved this by conducting extensive direct shear tests and numerical modelling using Abaqus software on clay improved with cement-lime columns. Yapage et al. (2013) conducted numerical modelling on deep cement columns in soft soils and made similar observations.

Although cement and lime have historically been employed successfully as binders for soil stabilization and other applications, the environmental and durability concerns from their use made the research community to search for alternate sustainable binders such as Geopolymers.

### 2.3.2. Need for Geopolymers

Cement is a prominent worldwide substance that offers substantial benefits in civil engineering sectors and soil stabilisation. Nevertheless, it has several disadvantages that have sparked worldwide discussions on decreasing cement manufacturing. Cement manufacturing has a significant environmental effect, accounting for about 6.0% of worldwide $CO_2$ emissions, a key factor in contributing to global warming (Mikulčić et al., 2016; Song, D. et al., 2016; Zhang et al., 2017). The construction sector has had significant global expansion, averaging 6.95% annually. The greatest growth rate was 9.0% in 2010 and 2011, followed by a decrease to 3.0% in 2012 (MR&CL, 2013). The worldwide cement market is forecasted to grow at a rate of 5% per year, with China leading in cement production (van Ruijven et al., 2016).

The process of cement manufacture depletes natural resources via the significant amounts of raw materials and fuel needed for electricity generation. Cement is mostly made up of cement clinker, which is created by mixing and grinding limestone and clay or other minerals like shales. This process is detailed in several sources (British Geological survey, 2005; CEMBUREAU, 2009; García-Gusano et al., 2015; Zhang et al., 2017a). Producing one tonne of cement needs 1.5 to 1.8 tonnes of limestone and 0.4 tonne of clay. Cement production is highly energy-intensive because it involves incineration processes that require high temperatures ranging from 1400 °C to 1500 °C to produce cement clinker, which is a crucial component of cement. Energy consumption in cement manufacture represents 50% - 60% of its production expenses (Liu et al., 2015). The sector is expected to be the second-largest consumer of energy globally, behind the steel industry, with between 12% to 15% of global industrial energy consumption (Van Ruijven et al., 2016). Producing one tonne of cement clinker necessitates between 3.0 and 6.5 GigaJoules (GJ) of energy, used for incineration processes and thermal energy (Rahman et al., 2015; Horsley et al., 2016). The grinding step in contemporary cement factories requires an estimated 110 – 120 kWh of electrical energy per tonne of cement (Rahman et al., 2015; Diego et al., 2016).

The cement and construction industries are under enormous pressure to reduce their $CO_2$ emissions by creating more environmentally and financially sustainable alternatives to cement. Alkali activated industrial by-products and wastes such as ground granulated blast furnace slag (GGBS), fly ash (FA), metakaolin, palm oil fuel ash, etc., referred to as geopolymers, are considered as possible substitutes. These are beneficial since they eliminate the necessity of transferring the industrial by-products and wastes to landfills, and are abundant, and have minimal to zero production costs. Geopolymers can decrease greenhouse gas emissions by as much as 64%.

# 2.4. Geopolymers

## 2.4.1. Brief introduction

Geopolymers are a novel kind of binding materials that rely on alkali-aluminosilicate reactions, exhibiting qualities similar to Ordinary Portland Cement (OPC) but with much reduced carbon emissions. Geopolymers are created by the alkali activation of materials rich in aluminosilicate. They are characterised by extensive three-dimensional networks of covalently bound alumino-silicates and are recognised for their notable compressive strength, little shrinkage, and long-lasting qualities (Duxson et al. 2007a). Alkali activated materials can be created from low-cost aluminosilicate precursors such as clay, metakaolin, fly ash, and others. They solidify quickly at room temperature and are viewed as a more environmentally friendly and enduring option compared to traditional building materials (Davidovits 1991, van Jaarsveld et al. 2002, Cheng and Chiu 2003, Gordon et al. 2005).

## 2.4.2. History of Geopolymers

In 1908, German researcher Kuhl patented the formation of alkali-activated materials (AAMs) similar to Portland cement. Purdon further developed the fundamentals of AAMs by testing blast furnace slags activated by sodium hydroxide and calcium hydroxide solutions. In the 1950s, Glukhovsky discovered that alkali-activated binder materials could be created using low-calcium or calcium-free aluminosilicates (clay), known as 'soil cements' and 'soil silicates', based on their similarity to natural minerals. This discovery is believed to be the first recorded synthesis of geopolymers. In the 1980s, French material scientist Joseph Davidovits developed geopolymer binders by alkali activating naturally occurring materials like kaolinite, limestone, and dolomite. His patents sparked interest in geopolymers, which have since been explored in various disciplines, including chemistry, mineralogy, material sciences, and engineering applications. Geopolymers have various uses, including fire-resistant materials, thermal insulation, containment of radioactive materials, corrosion-resistant coatings, adhesives, cements, concretes, and infrastructure composites (Davidovits 1991, Van Jaarsveld et al. 1999, Duxson et al. 2007a, Provis and van Deventer 2009, Temuujin et al. 2009).

## 2.4.3. Geopolymer terminology

Davidovits coined the term 'geopolymer' in the 1980s, which refers to a class of inorganic, alumino-silicate-based ceramics that are rigid gels made under ambient conditions to form near-net dimension bodies (Bell et al., 2009). Geopolymers are a subset of AAMs, which are materials formed by combining an aluminosilicate precursor and an alkaline activator. Geopolymers are essentially AAM binders, formed with little to no calcium and often using metakaolin or fly ash as the aluminosilicate precursor (Provis and van Deventer 2014). Research on geopolymer binders has been ongoing for decades, but there is still confusion about their correct terminology. These materials are commonly referred to as alkali-activated materials (AAMs), inorganic polymers, or geopolymers. Van Deventer's categorization of AAMs is widely acknowledged, with darker shading indicating higher concentrations of Na and/or K. Geopolymers are considered a subset of inorganic polymers, which are in turn subsets of AAMs (Figure 2.6).

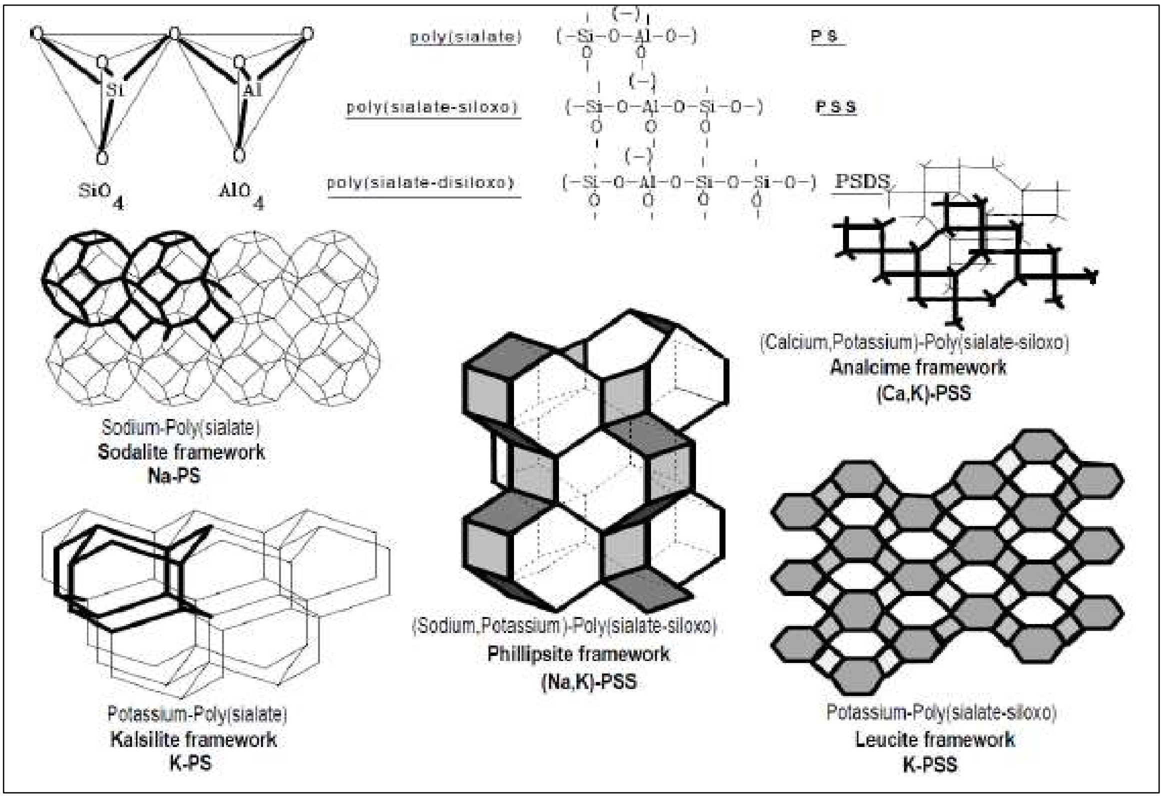



**Fig. 2.6. Molecular framework and the associated crystalline structures of geopolymers (Davidovits 1991)**

### 2.4.4. Geopolymer structure

Geopolymers, chemically known as polysialates, are chain or ring polymers with $Si^{4+}$ and $Al^{3+}$ in IV-fold coordination with oxygen (Figure 2.6). They range from amorphous to semi-crystalline in nature (Davidovits 1991) and can be represented using the empirical formula of polysialates, as shown in Equation 2.2.

$$M_n\,[-(SiO_2)_z-AlO_2]\,n.\,wH_2O \text{ ---- Eq. 2.2.}$$

where, 'M' is the alkali metal cation (such as Na, K, or Ca), 'n' is the degree of polycondensation, 'z' is the silicon to aluminum (Si:Al) ratio (usually 1, 2, or 3), and 'w' is the molar water amount. Polysialates, categorized by Si:Al atomic ratios of 1, 2, and 3, are referred to as poly(sialate) (PS), poly(sialate-siloxo) (PSS), and poly(sialate-disiloxo) (PSDS) respectively (Figure 2.6). The aluminosilicates that are commonly derived from fly ash, metakaolin, and slag consist of reactive forms of silicon (Si) and aluminium (Al). Alkaline activators commonly consist of solutions containing sodium hydroxide (NaOH), potassium hydroxide (KOH), and sodium silicate ($Na_2SiO_3$). When the aluminosilicate materials are combined with the alkaline solution, the first step is the dissolving of silicon (Si) and aluminium (Al) from the solid substance. The alkaline environment disrupts the Si-O-Si and Si-O-Al interactions, liberating silicate ($SiO_4$) and aluminate ($AlO_4$) species into the solution. The silicate and aluminate species in solution undergo polycondensation, resulting in the formation of a three-dimensional network. This process entails the condensation of Si-OH and Al-OH groups, resulting in the formation of Si-O-Si and Si-O-Al bonds, while simultaneously releasing water as a by-product. The polymerization process leads to the creation of a geopolymer gel, composed of an amorphous or semi-crystalline aluminosilicate network. The gel undergoes progressive condensation and solidification as time progresses. The ongoing reorganisation of the structure results in heightened mechanical robustness and stability. Calcium has the ability to affect the process, resulting in the creation of calcium-alumino-silicate-hydrate (C-A-S-H) gel. This gel can exist alongside the geopolymer gel and play a role in determining the qualities of the material. The ultimate geopolymer structure consists of an intricate and interconnected arrangement of Si-O-Si and Si-O-Al chemical linkages. This structure imparts the material with its distinctive mechanical robustness, chemical resilience, and long-lasting nature (Provis, J. L. et al, 2009)

### 2.4.5. Geopolymerization process

Geopolymers are eco-friendly construction materials formed by the reaction of aluminosilicate sources with an alkaline solution, resulting in a gel called N-A-S-H that dictates the material's properties. In his study, Duxson et al. (2007a) discussed the composition of geopolymers, and their synthesis process, which involves the dissolution of aluminosilicates and the formation of a gel through polymerization. The authors highlighted the importance of the Si/Al ratio and the role of alkaline activators in the production of geopolymers. The alkaline activator plays a crucial role in the synthesis and characteristics of geopolymers. It is responsible for the dissolution of the aluminosilicates present in the raw material, which is a critical step in the geopolymerization process. The choice of alkaline activator and its concentration can significantly affect the properties of the resulting geopolymer. . For instance, the most commonly used alkaline activators are silicate solutions and alkaline hydroxides, such as sodium hydroxide (NaOH) and potassium hydroxide (KOH). The use of different activators can produce geopolymers with varying properties, so the selection of the activator depends on the desired characteristics of the final product. The concentration of the alkaline activator is also important. Higher concentrations of the activator have been shown to lead to increased compressive strength of the geopolymer. However, there is a limit to this effect, as excessively high concentrations can lead to precipitation and reduced workability. In addition, the ratio of silicate solution to hydroxide (SS/NaOH ratio) is a critical parameter for the development of good compressive strength in geopolymers. The combined use of NaOH and silicate solution is often more cost-effective and can produce geopolymers with better mechanical properties than using either activator alone. The alkaline activator also affects the rheology of the geopolymer paste, with silicate solutions generally being more viscous than hydroxide solutions. This can impact the workability and processing of the geopolymer mixture. In summary, the alkaline activator is essential for the dissolution of aluminosilicates and the subsequent formation of the N-A-S-H gel in geopolymers. Its type, concentration, and ratio with other activators can significantly influence the mechanical strength, rheological behavior, and overall performance of the geopolymer. During the synthesis of geopolymers, the alkaline activator reacts with the aluminosilicate source, breaking down the aluminosilicate network and releasing aluminium and silicon monomers into the solution. These monomers then undergo polymerization to form the N-A-S-H gel, which is the main binding phase in geopolymers. They also discuss the raw materials used, including fly ash, metakaolin, and mining tailings, which are rich in alumina and silica. This review covers the history of geopolymers, tracing back to Joseph Davidovits' research in the 1970s, and their development as a 3D amorphous-semi-crystalline material. The authors note that the mechanical strength of geopolymers is influenced by the Si/Al ratio, water content, curing temperature, and the type of alkaline activator used.

According to Bhat and Kandagor (2014) geopolymers are formed through an alkali-activated polycondensation reaction process, where monomers bond, releasing water or other condensed molecules. Glukhovsky proposed a mechanism in the 1950s, consisting of destruction-coagulation, coagulation-condensation, and condensation-crystallization. Over time, researchers have expanded on this process due to technological advancements, providing better insight into the process. The process involves three major steps: destruction-coagulation, coagulation-condensation, and condensation-crystallization (Davidovits 1991, Provis and van Deventer 2009). Figure 2.7 illustrates the geopolymerization process, which can be broken down into five stages: dissolution, speciation equilibrium, gelation, reorganization, and polymerization and hardening. These stages are successive reactions, but they are generally overlapping and occur concurrently.

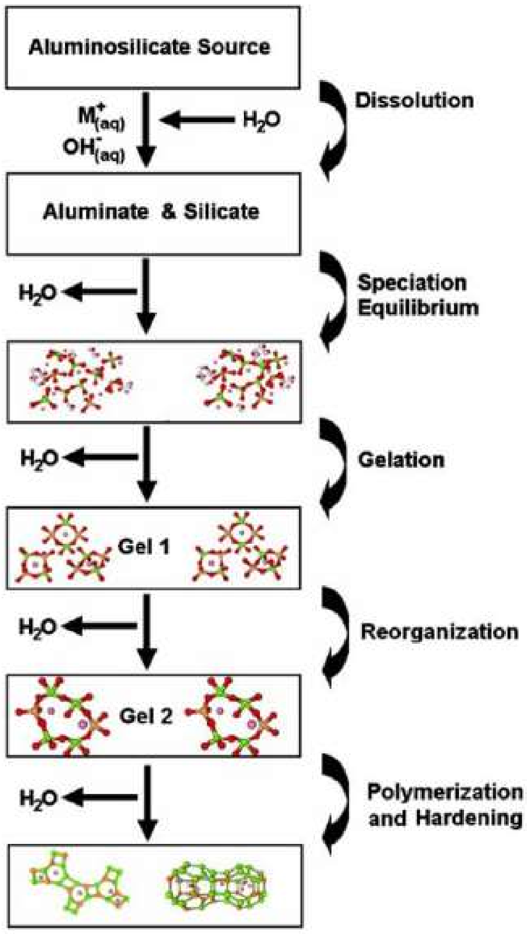



**Fig. 2.7. Schematic of process of geopolymerisation (Duxson et al. 2007)**

The dissolution process of aluminosilicate begins when the precursor is mixed with an alkaline activator solution containing metal cation, water, and silica. At high pH, water dissolves the precursor, forming monomeric aluminate and silicate species through hydrolysis. This results in a complex supersaturated aluminosilicate solution, which undergoes polycondensation reactions to form large networks or chains, resulting in gelation. The water released during this process is not chemically bound to the geopolymer structure (Duxson et al. 2007a). The gel system then reorganizes into a complex 3-D structure with extensive aluminosilicate networks, indicating geopolymers. Further curing results in hardening and the formation of evolved polymeric networks that eventually crystallize. The gelation of dissolved aluminosilicate species is dependent on factors such as the concentration of reactive species, raw material type and quality, processing conditions, and time.

### 2.4.6. Geopolymers in DSM

Despite the use of Flyash and Slag as substitutes for Portland cement in ground development endeavours, researchers have identified certain limitations. Notably, it has been observed that Flyash and Slag, when employed independently, exhibit inferior strength compared to Portland cement. The issue could be resolved by employing alkaline activation (geopolymerisation) on these wastes, resulting in the production of geopolymer binders with significantly enhanced strengths (Cristelo et al., 2013; Du et al., 2017; Rios et al., 2017). Geopolymers were initially presented in the 1970s, but it was not until the early 1990s that researchers began to give them more significant consideration (Davidovits, 1991).

### 2.4.7. Flyash and Slag as precursors

The most commonly used precursors are fly ash, ground granulated blast furnace slag (GGBS), and metakaolin (Pourakbar and Huat, 2017; Arulrajah et al., 2018), among other industrial residues and wastes (Huang et al., 2022). This sub-section will examine some of the most noteworthy studies conducted on the utilisation of Flyash and Slag based geopolymers.

Yip et al. (2005, 2008) conducted experiments to examine the strength and chemical properties of several calcium silicate based geopolymers. Various calcium silicate variants, include metakaolin, cement, powdered granulated blast furnace slag, and combinations of $NaOH$ and $Na_2SiO_3$ as liquid alkaline medium (L), either with or without fine washed sand as aggregate, was utilised. The metakaolin/metakaolin+calcium silicate ratios ranged from 0 to 1, with a fixed sand/metakaolin+calcium ratio of 3. The L/metakaolin+calcium silicate ratios were between 1.45 and 1.65, while the water/binder ratio ranged from 0.39 to 0.44. Specimens were subjected to curing for a duration ranging from 1 to 240 days. Subsequently, various analyses including compressive strength testing, scanning electron microscopy (SEM), and X-ray diffraction (XRD) were performed on the specimens. The findings demonstrated that the inclusion of a 20% mixture of metakaolin and calcium silicate resulted in the greatest level of strength. Furthermore, the combination of ground granulated blast furnace slag and cement produced higher strengths compared to other calcium silicates. Furthermore, nearly all the combinations attained their ultimate potency within a week of the curing process, with only slight enhancements in strength observed thereafter. Li and Liu (2007) and Kumar et al. (2010) have similarly noticed a reduction in curing time when Slag is added to Flyash-based geopolymers. Yip et al. (2005, 2008) found that an L/metakaolin+calcium silicate ratio of 1.45 led to stronger combinations. In addition, the presence of both sodium aluminosilicate hydrate (NASH) gel and calcium silicate hydrate (CSH) was found, which resulted from the presence of calcium. Kumar et al. (2010), Yang et al. (2012), and Ismail et al. (2014) similarly observed this cohabitation while including Slag into Flyash-based geopolymer. According to Yang et al. (2012), increasing the Slag content up to 80% improves the strength. This finding confirms the effectiveness of the coexistence of Flyash and Slag.

Chen and Chang (2007) performed compression and flexural strength experiments on geopolymers made from fly ash (FA) and slag (S). The researchers utilised calcined clay, FA, S, and sand in various combinations, together with $NaOH$+$Na_2SiO_3$ (with a ratio of $Na_2SiO_3$/ $NaOH$ = 2.4) as the liquid alkaline medium (L). The water content fell within the range of 20-45%. The cure durations were 7, 28, and 90 days. The maximum strength was seen in the mixture containing 33% sand, 34% calcined clay, and 34% S. Furthermore, it was determined that the flexural strength is approximately 10% of the compressive strength. In addition, the specimens achieved 80% of their strength during a period of 7 days, and there was minimal increase in strength after 28 days.

Rattanasak and Chindaprasirt (2009) conducted an experiment where they created mortars utilising fly ash (FA) based geopolymers mixed with river sand. The purpose of the experiment was to examine how the addition of sodium hydroxide ($NaOH$) affected the strength of the mortars. Solutions of $NaOH$ with molarities of 5, 10, and 15 were combined with $Na_2SiO_3$ at $Na_2SiO_3$/ $NaOH$ ratios ranging from 0.5 to 2.0. The FA was mixed with sand with 36% content, and the resulting specimens were subjected to a curing process at a temperature of 65°C for a duration of 2 days. The findings indicated that higher strengths were achieved by increasing the molarity of $NaOH$ and the $Na_2SiO_3$/ $NaOH$ ratios within the range of 1.5 – 2.0.

Prior studies (Heah et al., 2011; Liew et al., 2016; Park et al., 2016; Suksiripattanapong et al., 2015) have documented a reduction in the compressive strength of kaolin, metakaolin, FA+S based geopolymers, and water treatment sludge stabilised with FA based geopolymer after a specific curing time at temperatures above 80°C. Nevertheless, there was no significant increase in strength reported in FA based geopolymer concrete even with extended curing when subjected to heat-curing (Hardjito and Rangan, 2005). Previous research has shown that the compressive strengths of FA based geopolymer and soft soil stabilised with FA based geopolymer can continue to rise for up to one year, even when cured at a temperature of 85°C. This has been shown in various experiments conducted by Criado et al. (2007), Cristelo et al. (2011), and Cristelo et al. (2013). This suggests that the influence of curing temperature on the advancement of strength is heavily contingent upon the specific characteristics of the material. While there have been numerous studies conducted on curing temperatures above room temperature, research on temperatures below room temperature has been few.

Cristelo et al. (2011) examined the utilisation of FA in conjunction with liquid activator (L) for enhancing deep soft soil using laboratory and field compression testing. They then compared the outcomes with those obtained from Portland cement treatment. The soil consisted of clay with a liquid limit (LL) of 32% and a plastic limit (PL) of 10%. The study utilised cement amounts ranging from 10% to 30%, FA contents ranging from 20% to 50%, and an L/FA ratio of around 1.0. The curing time extended up to one year. The L solution consisted of a combination of $Na_2SiO_3$ and $NaOH$ with molar concentrations of 10.0, 12.5, and 15.0. The ratio of $Na_2SiO_3$ to $NaOH$ in the solution was 2:1. The study found a rise in the unconfined compressive strength (UCS) of the samples as the content of fly ash (FA) and the duration of time increased. Furthermore, although the cement exhibited rapid strength development within 28 days, the soil treated with activated FA had a consistent and continuous increase in strength up to 1 year. In addition, the soil treated with activated FA exhibited greater strength values compared to the soil treated with cement after a period of three months. While alkali activated FA has been identified as a suitable substitute for cement, additional research is advised to find methods for reducing the duration required for strength development.

Cristelo et al. (2013) conducted a comparison between the strength development of a soft soil stabilised with Portland cement and that of a soil stabilised with a geopolymer based on fly ash (FA). The soil had a sandy clay composition with poor plasticity, characterised by a liquid limit (LL) of 32.4%, a plastic limit (PL) of 10.5%, and a sand content of 50%. The soil was stabilized with both cement and FA separately in proportions of 20%, 30%, and 40% of the total solid components, which correspond to 25%, 43%, and 67% of the soil in dry state. The geopolymeric specimens were prepared using a combination of $NaOH$ solutions with different molar concentrations (10.0, 12.5, and 15.0) and $Na_2SiO_3$ solution with a $Na_2SiO_3/ NaOH$ ratio of 2.0. The ratio of the liquid alkali (L) to the fly ash (FA) was adjusted between 1.0 and 2.5. The cement treated specimens were subjected to water/cement ratios ranging from 0.5 to 1.5. Time periods ranging from 7 days to 1 year were taken into account for the curing process. The test results indicated that decreasing the L/FA ratio would be advantageous in terms of both strength enhancement and cost considerations. The L/FA ratio of 1.0 yielded the highest strength among all geopolymeric specimens. The cement-treated specimens obtained their highest strengths with water contents ranging from 1.0 to 1.3 liquid limits (LL). In addition, the cement-treated samples achieved over 90% of their ultimate strength within 28 days, whereas the FA-geopolymeric samples only attained 20–40% of their one-year strength in 28 days.

The water-to-cementitious material ratio ($w_c$/C ratio) is the primary factor that significantly affects the strength development in the process of cement stabilisation of clays. Given a fixed ratio for a certain goal strength, the cement content can be modified according to the water content in order to attain the desired strength (Bushra and Robinson, 2013; Cong et al., 2014; Horpibulsuk et al., 2011a; Horpibulsuk et al., 2011b; Lorenzo and Bergado, 2004). Generally, the water content in coastal areas is significantly elevated. Consequently, a substantial quantity of cement is necessary to meet the specified strength criteria. When it comes to geopolymer stabilisation, the L/precursor ratio has a greater influence than the water/precursor ratio. An advantage of utilising geopolymer binders instead of Portland cement in DSM can be acknowledged (Cristelo et al., 2013).

In their study, Sukmak et al. (2013) examined the impact of several factors, such as the L/FA ratio ranging from 0.3 to 0.8, the $Na_2SiO_3/ NaOH$ ratio ranging from 0.4 to 2.3, the curing temperature ranging from 65 to 85 °C, and the curing time of up to 90 days, on a geopolymer stabilised soil using FA as a foundation material. The soil consisted of a clay type known as CH, with liquid limit (LL) and plastic limit (PL) values of 54% and 28%, respectively. The samples were subjected to a curing process at room temperature for 24 hours, followed by curing at temperatures of 65, 75, and 85 °C for durations of 24, 48, and 72 hours respectively. Finally, the samples were allowed to cure at room temperature again. The clay's water content was adjusted to the optimal water content determined by modified compaction testing. The study found that the optimal ratios of L/FA and $Na_2SiO_3/ NaOH$ were 0.6 and 0.7, respectively. The optimal strength was attained at a curing temperature of 75 °C, but any further increase in temperature resulted in a decrease in strength. Furthermore, the specimens that were cured at a temperature of 65 °C achieved a point of maximum strength development after 28 days of curing.

In their study, Phetchuay et al. (2016) performed a series of unconfined compressive strength (UCS) tests on clay stabilised with geopolymer made from fly ash (FA). The purpose was to examine how several factors, such as FA content, L/FA ratio, water content, curing time and temperature, and the ratio of sodium silicate ($Na_2SiO_3$) to sodium hydroxide ($NaOH$), influenced the development of strength in the material. The parameters utilised were as follows: FA levels ranging from 25% to 45%, L/FA ratios ranging from 0.5 to 2.0, water content equivalent to 1-2 liquid limits of soil, curing times ranging from 3 to 28 days, curing temperatures ranging from 25 to 40 °C, and $Na_2SiO_3$/ $NaOH$ ratios ranging from 1.5 to 9.0. The test findings indicated that the optimal values for strength growth were L/FA = 1, water content = 1 LL, and $Na_2SiO_3$/ $NaOH$ = 2.33. Furthermore, the strength exhibited a positive correlation with both the curing temperature and time; notably, the relationship was logarithmic for the latter. Furthermore, the potency was heightened with the rise in FA concentration. In a previous study, Phetchuay et al. (2014) determined that the ideal amount of FA to enhance the characteristics of a CH clay using FA based geopolymers is 15%.

Pourakbar et al. (2016), Rios et al. (2017), and Sukmak et al. (2017) utilised palm oil ash and FA-based geopolymers to stabilise clayey and sandy soils, respectively. They observed significant long-term strength improvements in both cases. However, it is important to note that the studies mentioned had a relatively low water content and utilised dynamic compaction for sample preparation. In contrast, soils found in coastal locations typically have high water contents, and dynamic compaction is not employed in the process of Deep Soil Mixing (DSM). FA-based geopolymers have been utilised for stabilising soft soil. However, it has been observed that FA requires a longer curing time compared to a Portland cement binder in order to obtain the desired strength. This finding has been published by Cristelo et al. in 2011 and 2013, as well as by Phetchuay et al. in 2016. According to Kumar et al. (2010), the inclusion of S has been found to decrease the amount of time it takes for FA based geopolymers to solidify and increase their initial strength.

Sargent et al. (2016) discussed the challenges of soft alluvial soils, the effectiveness of GGBS+$NaOH$ at dosages of 2.5 to 10 % in improving strength and durability, the need for more sustainable replacement binders, and the challenges related to transportation distances between sourcing plants and stabilisation sites. Stabilizing alluvial soil with sodium hydroxide

($NaOH$) activated GGBS produced significant strength and durability improvements, particularly when using binder dosages >7.5% by dry weight, which comfortably met or surpassed those exhibited by samples stabilized with equivalent quantities of lime or cement and met criteria of minimum UCS >300 kPa defined by EuroSoilStab (2002). The GGBS+$NaOH$ binder potentially has an impressive level of engineering practicality, exceeding that of lime, cement, and other industrial wastes-based binders. The use of GGBS+$NaOH$ has the potential of becoming a more sustainable alternative than the continued use of lime and cement, thereby promoting its commercialization potential.

Jhonathan F. Rivera (2020) studied the utilisation of Alkali-activated cements (AACs) for soil improvement. AAC were prepared using granulated blast furnace slag (GBFS) and lime (L) as calcium sources. From their study it can be understood that AACs using industrial by-products can significantly increase the compressive strength of soil under soaked conditions. Along with that, further the mass loss percentage after wetting and drying cycles was within the allowed limits by Colombian specifications for stabilized soil, indicating the potential for using this type of fly ash as a sustainable alternative to replace Portland cement in soil stabilization for road construction. However, further studies on the long-term performance of AACs stabilized soils, and the cost of applying AACs in soil stabilization are worth being conducted.

Arul Arulrajah et al. (2018) investigated the fly ash (FA) and slag (S) as alternative green binders in ground improvement projects to reduce the carbon footprint by showing that they can be effective alternatives to traditional cement or lime binders. Unconfined compression strength (UCS), Flexural Strength (FS) and Scanning Electron Microscopy (SEM) imaging tests were conducted on the specimens. The use of fly ash and slag binders significantly increased the strength and stiffness of the soft clay, making them viable alternatives to traditional cement or lime binders. The optimum binder content was found to be 20%, with a specific mixture of Coode Island Silt (CIS) + 5%FA + 15%S identified as the optimum combination. The ground improvement industry has been exploring environmentally friendly alternatives with low carbon dioxide emission, and their study's results support the use of these new binders for this purpose.

Mohammadjavad Yaghoubi et.al. (2019) discusses the use of geopolymers as sustainable binders in deep soil mixing (DSM) projects, highlighting their potential environmental benefits. The combination of 30% $NaOH$ with 70% $Na_2SiO_3$ achieved the highest strengths in the soil when used as binders. Increasing the slag content resulted in significant improvements in strength. The results showed an excellent correlation between strength and stiffness, which is expected to aid in the development of relationships for strength prediction of these green binders in geotechnical applications.

Abdullah (2020) highlighted the use of FA-based geopolymers for enhancing soil properties, Fly-ash geopolymer can be used successfully as a binder for soil stabilisation, but further research is needed to realize its full potential. The utilization of geopolymers for soil stabilisation is not widely recognized by the geotechnical engineering industry. The use of FA-based geopolymer through the (N, C)-A-S-H model for soil stabilisation at ambient temperature is still not widely recognized by the geotechnical industry. There is a limited study on enhanced geopolymer mixtures for clay treatment, need for further testing and interpretation, lack of constitutive models, limited industry recognition, requirement for further research, and scarcity of practical procedures.

Abdila (2022) focused on the combination of fly ash and ground granulated blast furnace slag based geopolymer for soil stabilisation. Geopolymers using GGBFS and fly ash can successfully improve the mechanical and physical qualities of clayey soils for soil stabilization in road construction applications. Low-strength soil layers, particularly clayey soils, pose significant challenges for civil engineers and construction projects. The study highlights the potential of GGBFS and FA-based geopolymers as effective binders for soil stabilization. It is reported that there is a need for further studies on the impact of geopolymer treatment, constitutive models, and practical procedures.

In contrast to fly ash geopolymers, which necessitate a high curing temperature (60 – 200 °C) and a vigorous working environment to initiate the reactions, geopolymers produced with GGBS impart high early strength during curing at ambient temperatures (Davidovits, 2008). Also, in light of the various studies, it can be generally inferred that when these precursors are combined with $NaOH$ and $Na_2SiO_3$, they enhance the strength and durability of the treated soil. Furthermore, among the various combinations of hydroxides and silicates that are abundant in soluble metals such as potassium (K) and sodium (Na) that were considered as potential alkaline media, the combination of sodium hydroxide ($NaOH$) and sodium silicate ($Na_2SiO_3$), i.e., $NaOH$+$Na_2SiO_3$, demonstrated the highest efficacy and was widely endorsed by researchers in the cement and concrete industry (Duxson et al., 2007). However, it is imperative to minimise the utilisation of $Na_2SiO_3$ to reduce global warming drastically, given that its production and transportation generate substantial quantities of $CO_2$. This highlights the critical necessity for additional research on geopolymers utilising $NaOH$ exclusively as an alkaline medium (Reddy and Murugan, 2020).

While there have been several studies conducted on the use of FA and GGBS as precursors and $NaOH$+$Na_2SiO_3$ as alkaline activators, research on the use of only GGBS and $NaOH$ based geopolymers for DSM applications has been few (Sargent et al., 2017; Yaghoubi et al., 2019), in which their study was either limited to binder dosage of up to 10% or usage of GGBS+$NaOH$ was a small part of an elaborate study comprising of other combinations of precursors and alkaline activators. Thus, there is a need to synthesize a geopolymer binder with GGBS and $NaOH$ and evaluate its efficacy for use in DSM applications. Also, to achieve the desired UCS of 1034 kPa to 4137 kPa, i.e., 150 psi to 600 psi (Bruce, 2001; Puppala et al., 2008) for a wide range of DSM applications, the use of high binder contents (>10%) and high alkali concentrations in soil-binder columns is essential that raises the material's brittleness, making it susceptible to abrupt brittle failure under compression, tension, shear, bending, and rotation (Broms, 1999; Filz and Navin, 2006; Jie Han, 2015). To prevent sudden failure and reduce their brittleness, the column material can be reinforced with randomly distributed fibers.

## 2.5. Fiber reinforcement in deep mixed columns

Some researchers are currently examining the application of fiber reinforcement to stabilised soil columns to reduce the expansion of microcracks without compromising the columns' strength (Zhang et al., 2008; Sukontasukkul, P., & Jamsawang, P., 2012; Correia et al., 2015; Syed et al., 2020). However, additional research is necessary to validate these findings for DSM applications (Bo Ruan et al., 2021).

In their study, Tang et al. (2007) investigated the effects of polypropylene fiber on the strength and mechanical behavior of uncemented and cemented clayey soil. Inclusion of PP-fiber reinforcement increased soil strength and changed its behavior. Bond strength and friction at the interface were identified as dominant mechanisms controlling reinforcement benefit. Reinforcing fiber has further improved the micromechanical properties at the fiber/matrix interface. Their study mentioned that behavior at the interface differed between fiber-reinforced uncemented and cemented soil.

Sukontasukkul and Jamsawang (2012) explored the use of steel and polypropylene fibers in soil-cement piles to enhance their flexural performance. The deep cement mixing technique, used in Thailand, has been used to improve soft clay strength, but it has poor tensile and flexural strengths. The fiber reinforced soil cement (FRSC) was produced using polypropylene and steel fibers at different volume fractions, with polypropylene fibers performing better than steel fibers. The study also introduced a technique of mixing short fibers to enhance the flexural strength and to reduce the brittleness of the soil-cement mixture. The toughness of the treated mixes also improved with increase in the volume fraction of fibers. The FRSC's flexural performance was assessed according to ASTM C1609.

Correia et al. (2015) examined the effect of binder and PP fiber dosages on the mechanical properties of "Baixo Mondego" soft soil. Their study employed four different types of tests to assess compressive strength and tensile strength. Higher binder content resulted in greater stiffness, compressive strength, and tensile strength, with a reduced effect on specimens reinforced with fibers. The relationship between the fiber content and stiffness, compressive, and tensile strength was non-linear, suggesting that mechanical properties do not increase proportionally with the addition of fibers to the paste. Adding low fiber content had minimal effect on compressive and direct tensile strength, decreased stiffness and strength loss, and shifted behaviour of the specimens from brittle to ductile. The influence of adding fibers to the treated soil mix varies based on the specific test and the level of strain at which failure occurs. The results of their study have shown that the effect of adding fibers on strength of treated soil mix varies depending on the strain mechanism employed in each test. Thus, inclusion of fibers has a substantial influence on flexural strength and minimal impact on direct tensile strength of the treated soil mixtures.

The research of Estabragh A. R. et al. (2017) provided insights into the methods of stabilizing clay soils including chemical and mechanical techniques, and discussed the effects of additives such as cement and fibers on the mechanical behaviour of the soil, aiming to provide useful insights for simulating real-life projects. Stabilization of clay soils through the addition of chemical additives like cement results in lower compressibility and higher strength compared to natural soil. The inclusion of fibers significantly increases the peak compressive strength, ductility, splitting tensile strength, and flexural toughness of clay soil and soil-cement. Reinforcing the soil-cement with polypropylene fibers reduces brittleness, increases ductility, and decreases stiffness. Jaiswal et al. (2022) and Tamassoki et al. (2022) mentioned similar findings in their review of research on fiber reinforced cemented soils.

Wei et al. (2018) investigated the mechanical properties of soil and lime-soil by solidifying it with wheat straw, rice straw, jute, and polypropylene fibers. The study found that all fiber types improved soil shear strength and deviatoric stress-strain properties. The optimal fiber content was found to be 0.2% or 0.25%, with the optimal fiber length being 30% or 40% of the sample diameter, with polypropylene fiber being the best for reinforcement. Fiber reinforcement significantly increased the cohesion and slightly improved the internal friction angle. All four kinds of fiber may improve the strength and deviatoric stress-strain properties of soil and lime-soil, with polypropylene fibers identified as the best for reinforcement of treated soil.

Syed et al. (2020) discussed the use of alkali activated binder (AAB) with polypropylene (PPF) and glass fibers (GF) to improve the geomechanical properties of expansive black cotton soil (BCS), addressing the low volumetric stability of BCS due to moisture imbalance and the environmental impact of traditional binders like lime and cement. The study reported improvement in strength properties with varying dosages of fibers in AAB treated BCS, and the higher bonding interaction and tensile resistance with 0.4% PPF reinforced AAB treated BCS. The study also proposed non-linear best-fit equations to relate experimental test results with model-predicted results for fiber reinforced AAB treated BCS.

Bo Ruan et al. (2021) evaluated the impact of polypropylene fiber on cement mortar soil defects through unconfined compressive strength and flexural strength tests. Increasing fiber content substantially increases the unconfined compressive strength, residual strength, and flexural strength of the fiber reinforced cement mortar soil (FRCMS). The peak strain and ratio of the flexural-compression strength ($R_{fcs}$) of the FRCMS have an optimal fiber content of 3.5%. The addition of an appropriate fiber dosage is suggested to substantially improve the plasticity and lateral stress capacity of the FRCMS. The brittleness index of the FRCMS is inversely proportional to fiber content, and the strength of the FRCMS improves with the increase in cement content, sand content, and curing age within a certain range.

Zhang J. et al. (2022) investigated the use of hybrid polypropylene fiber to reinforce cemented soil. The fiber reinforcement significantly improves the anti-flying property, anti-wear property, and crack resistance of cemented soil. The fiber content and fiber length have a significant impact on the properties of fiber reinforced cemented soil. The ideal combination of hybrid polypropylene fiber reinforced cemented soil is 0.3% coarse polypropylene fiber with the length of 38 mm and 0.3% fine polypropylene fiber with the length of 12 mm, with mechanical properties exceeding those of single polypropylene fiber reinforced cemented soil.

Pedroso, G. O. M et al. (2023) evaluated the flexural performance of a soil-cement pavement reinforced with polypropylene and steel fibers at three different curing times. Polypropylene and steel fibers were used at 0.5, 1.0, and 1.5% fractions by volume for three different curing times (3, 7, and 28 days) to assess the fiber effect in the cemented soil (CS) matrices. An evaluation of the material performance was carried out by using the 4-Point Flexural Test and the results show that steel fibers with 1.0% content improved initial strength and peak strength without affecting the material's flexural static modulus. Polypropylene fiber mixtures showed better performance in terms of ductility index, residual strength, and cracking control.

Hakan A. Kamiloğlu et al. (2024) explored the stabilization of silty soil using alkali-activated fly ash and fibers of different lengths 3 mm and 12 mm. Their study examined the impact of activator content and fly ash content on the UCS, Al/Si ratio, and $SiO_2/Na_2O$ ratio of stabilized samples. The study also investigates the effects of hybrid fiber length on UCS, secant modulus, flexural strength, toughness, and flexural load-deformation characteristics. Results showed that activator content significantly influences UCS value, with variations in fly ash content increasing UCS value up to 15%. Moreover, changing the activator content resulted in a maximum 12-fold increase in UCS value. Incorporating hybrid fibers for stabilization led to higher secant modulus (up to 30%), better flexural strength (up to 6%), and ductility without compromising UCS.

From the above literature study, it can be understood that polypropylene (PP) fibers, when used as reinforcement, improve the mechanical properties of the stabilized soils significantly. It can also be noted that several studies are reported on the use of PP fibers in cement/lime stabilized soils, whereas studies on their use in geopolymer stabilized soils, especially in DSM applications, are scarce. Hence, detailed study is needed to evaluate the effect of PP fiber reinforcement on the strength, stiffness and ductility of soils stabilized with geopolymers. Furthermore, it is necessary to investigate the durability of the fiber reinforced geopolymer treated soil mixes against wetting and drying in relation to the intrusion or extrusion of water (moisture variations) from the surrounding soil.

## 2.6. Bearing capacity of composite soft ground reinforced with deep mixed columns

Various methods have been suggested in the past to determine the bearing capacity of composite ground enhanced by deep soil mixing columns. Broms (2000), and Bouassida and Porbaha (2004) introduced an equation to determine the ultimate bearing capacity of composite ground improved by end-bearing columns, whereas Bergado et al. (1994) introduced equations for floating columns. The improvement area ratio ($A_r$ or $a_p$ or α) and the undrained shear strength of the soft soil and column ($c_{us}$ and $c_{uc}$ respectively) have been the main parameters considered for calculating the composite ground's ultimate bearing capacity. These methods and their equations are addressed below.

$$q_{ult} = c_{uc} \,.\, \alpha + (1 - \alpha) \,.\, c_{us} \quad \text{(Weighted method)} \quad \text{---- Eq. 2.3.}$$

$$q_{ult} = 0.7\, q_{uc} \,.\, \alpha + \lambda\, (1 - \alpha) \,.\, c_{us} \quad \text{(Broms method)} \quad \text{---- Eq. 2.4.}$$

where, $c_{uc}$ and $c_{us}$ are the undrained shear strength of the column and soft soil respectively, '$\alpha$' is the area ratio, $q_{uc}$ is the unconfined compressive strength of the column and '$\lambda$' is taken as 5.5 as proposed by Bergado et al. (1996).

Bergado et al. (1994) developed equations to determine the ultimate bearing capacity of composite ground improved by floating columns based on type of column failures. The ultimate load bearing capacity in relation to block failure and local shear failure is predicted using the following equations.

$$q_{ult} = \frac{2C_{us} \,.\, H_c \,.\, (B+L)}{BL} + (a)\, c_{us} \quad \text{(Block failure)} \quad \text{---- Eq. 2.5.}$$

$$q_{ult} = 5.5\, c_{av}\, (1 + 0.2\, \frac{B}{L}) \quad \text{(Local shear failure)} \quad \text{---- Eq. 2.6.}$$

where, $H_c$, B, L are height, width, and length of the DSM column group and '$a$' is a factor which is equal to 6 for rectangular column group with L > B or 9 for square column group and $c_{av}$ is the average shear strength along the assumed failure surface.

Bruce (2001) stated that in nations such as Scandinavia and the United States, the improvement area ratio ($A_r$), commonly used in the deep mixing approach typically ranges from

10% to 30%. An area ratio of 30 to 50% may be used in certain cases, particularly to prevent lateral deformation and slide failure caused by seismic events (Bergado et al. 1996).

Terashi and Tanaka (1981) conducted a study on the load-bearing ability of a composite ground enhanced by a combination of end-bearing and floating soil-cement columns through a 1 g physical model test. Casing pipes were used to install the columns within the consolidated model ground. An auger was used to extract the soil from the casing pipe. The empty casing was filled with a mixture of earth and cement, and then the casing pipes were removed. The ratio of the unconfined compressive strength of the column to that of the surrounding soil ($q_{uc}/q_{us}$) ranged from 11 to 173. The model studies indicated that the group of soil-cement columns displayed strain-softening behaviour. This shows that the column had brittle failure, resulting in a drop in residual strength post-failure. The ductile characteristic of the unimproved clay ground contrasted with the brittle behaviour of the composite ground, which was attributed to the presence of deep mixing columns. A distinct peak in the stress-strain curve was seen for a high-strength column with a compressive strength of 1040 kPa. The test findings indicated that the columns experienced gradual failures. The average maximum stress for the soil was almost equal to the carrying capacity of a shallow foundation on clay soil, whereas for columns, the value ranged from 55-80% of $q_{uc}$. They suggested an equation to calculate the ultimate bearing capacity of the enhanced ground, denoted as $q_{ult}$:

$$q_{ult} = A_{\mathrm{r}}\ p_t + (1 - A_{\mathrm{r}})\ N_c\ c_{\mathrm{us}} \qquad \text{---- Eq. 2.7.}$$

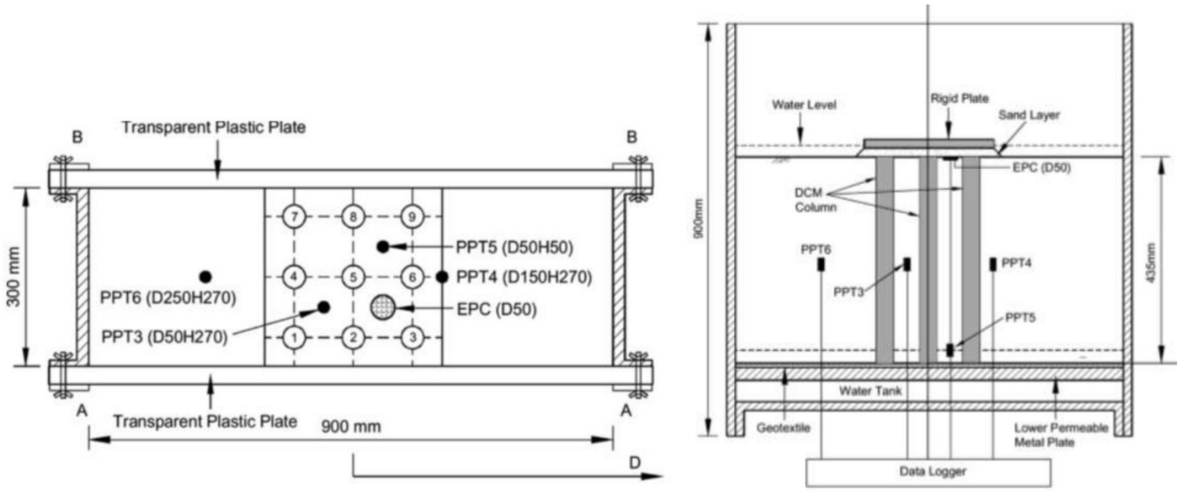
where, $p_t$ is the stress on the soil-cement columns and $N_c$ is the bearing capacity factor for shallow foundation on clay.

Kivelo (1998) conducted a static field load test on a 0.5 m diameter lime-cement column constructed in a soft, plastic clay soil with an undrained shear strength of around 18-20 kPa and a water content of 43%. The lime-cement mix ratio was 50 percent. The deformation in the column at varied depths was measured as the load increased, as well as the drop in the column's modulus of elasticity as a function of the applied force. It was discovered that increased axial strain resulted in a reduction in elastic modulus. The axial strain that develops in a lime-cement column over time is determined by how the column interacts with the surrounding soil. As a result, any changes in the surrounding native soil qualities, such as a change in saturation level, may have an impact on the column's long-term performance.

Rogers and Glendinning (1997) ascribed the improved soft clay soils' higher bearing capacity to the strength of the lime columns caused by confining pressure from the unimproved surrounding soil. They claim that the amount of lateral support grows significantly with depth, and that the way of creating these lime columns is determined by the degree of vertical confinement of the clay that is located near to the surface.

Bouassida and Porbaha (2004a, b) studied the ultimate bearing capacity of soft soil enhanced by a group of end-bearing columns with an area ratio of 18.8% by 1g physical modeling. Clay and soil cement columns with a diameter of 20 mm were used to create the model ground. The columns were built and allowed to harden away from the earth. The model was kept in completely saturated circumstances for two days before being loaded. The load-settlement behavior of columns was studied at various levels of column strength. The load values on the columns peaked at less than 10% of normalized displacement, suggesting a brittle failure of the columns. They have also investigated the ultimate bearing capacity of soft soil by implementing a set of end-bearing soil-cement columns based on the kinetic approach of yield design theory. A lower and upper limit solution for evaluating the carrying capacity of a soil increased by a series of end-bearing columns has been proposed. The findings obtained from the experimental research were validated using the predicted upper and lower limit solutions.

Yin and Fang (2010) researched the load-bearing ability and manner of collapse of a soft soil that was enhanced by a set of nine end-bearing DCM columns at a low area ratio of 12.6%. Figure 2.8 displays the model configuration. This research examined the wedge-shaped shear failure of the model ground. The bearing capacity of the model ground was computed using Weighted method and Broms methods and the values obtained were compared with the measured ones. Broms approach provides a more accurate assessment of the bearing capacity than the Weighted method. Observations were made on the coupling between columns before and after failure with that of consolidated soil using data of pore water pressures at various sites in soft soil.




(a) (b)

**Fig. 2.8. Model arrangement (a) plan view and (b) cross section (Yin and Fang 2010)**

Farouk and Shahien (2013) explored the load-bearing ability of a strip footing on Nile deltaic soil enhanced by soil-cement columns under plane-strain conditions with different area ratios of 8.7%, 10.4%, 13.9%, and 17.3%. The earth columns were positioned vertically using wooden forms. A 10 mm layer of untreated soil was placed on top of the improved ground, followed by a 480 mm x 100 mm x 20 mm rigid steel plate to mimic the performance of a strip footing on the improved soil. The steel plate was raised using a hydraulic jack until the soil column failed. An investigation was conducted on how the bearing capacity of stabilized soil is influenced by the area ratio, curing period, and length of the cemented column. Model experiments shown that the length and quantity of soil-cement columns are crucial foundation characteristics that impact the bearing capacity of the reinforced soil ground. By using the optimal length and number of columns, settlement may be decreased by up to 80%.

Dehghanbanadaki et al. (2016) studied the ultimate bearing capacity (UBC) of uniform peat ground enhanced by a series of end-bearing and floating soil-cement columns. Physical model tests were performed at several length/height ratios (0.25, 0.5, and 0.75) and three distinct area improvement ratios of 13.1%, 19.6%, and 26.2%. The footings' bearing capacity was calculated using several analytical approaches. The ultimate bearing capacity (UBC) of floating and end-bearing DCM columns rose by 60% and 223%, respectively, relative to the unimproved peat ground. The ultimate bearing capacity of floating columns on improved peat ground, calculated using Broms approach, closely corresponded with the findings of model testing. Brom's technique overestimated the ultimate bearing capacity by up to 25% for end-bearing columns.

Rashid et al. (2015a, b) studied the maximum load-bearing capacity of soft clay soil enhanced by end-bearing and floating columns using a sequence of 1 g physical model experiments. All columns were 24 mm in diameter and 200 mm in length and were put in the clay bed using the replacement technique. The impact of varying improvement area ratios (17%, 26%, and 35%) was analyzed. When using end-bearing columns on the ground, a 200% increase in bearing capacity was attained by employing an area ratio of 34.7%. The bearing capacity factor, $N_c$, showed a linear growth in relation to the area ratio. Ground with floating columns exhibits a 60 to 85% improvement in bearing capacity compared to untreated ground. A graph was created to compare the relative average shear strength of the improved ground under the footing to that of the surrounding soil, represented as $c_{uc}/c_{us}$, versus the bearing capacity factor. Rashid et al. (2015a, b) found that Nc reached a maximum value of 10 when the ratio of $c_{ua}/c_{us}$ exceeded 3.0.

Mamata Mohanty and J. T. Shahu (2020) investigated the effectiveness of soil-cement columns in improving the bearing capacity of soft soil. They tested soft soil with end-bearing and floating columns under axisymmetric conditions, evaluating the stiffness and failure stress. The influence of several group foundation factors, such as area ratio, column length and diameter, and binder concentration, is examined. From the study it was found that smaller-sized columns were more beneficial for both end-bearing and floating columns. The stiffness and failure stress increased with column length, but only marginally for those longer than 10 times the column diameter. The area ratio significantly influenced the failure pattern of end-bearing columns, with bending occurring at varying lengths. The study also found good agreement between the bearing capacity values obtained from the experiment and numerical analysis.

Despite extensive research in the field of DSM and use of Geopolymers for DSM applications has been carried out, a thorough review of the existing literature reveals that no previous research has been conducted pertaining to bearing capacity studies on DSM column improved composite soft ground using soil-geopolymer columns. Surprisingly, previous research lacks studies focusing on laboratory or field scale column studies on this aspect, as geopolymer technology is still in the early stages of its development and most of the studies were limited to the assessment of mechanical characteristics of stabilized soil with respect to various parameters like binder and alkali type and concentration, curing conditions and field conditions. Though Sireesh Saride and Vamsi N.K. Mypati (2024) have attempted laboratory scale DSM column studies using geopolymer and varying area ratios, their study was limited to analysis of swell-shrink behaviour of the DSM column reinforced expansive clay. This absence of prior work underscores the significance and novelty of the present study in filling this research gap and contributing to the existing body of knowledge in this field. Therefore, there is a need to conduct bearing capacity studies on a model soft clay ground improved with columns made of geopolymer synthesized in the present study and the same has been attempted.

## 2.7. Summary of Literature Review

From the above review of literature, it is evident that the DSM technique has been studied by various investigators in order to explore various parameters. Most of the previous works have used the lime and cement for the purpose of DSM and the use of these materials is discouraged in view of their high $CO_2$ contribution to environment. Hence, geopolymers have been studied as an alternative to lime and cement. Under this trend, mostly lower concentrations of $NaOH$ and GGBS were used so far and keeping in view the higher strength requirement for emerging DSM applications, the studies on the use of higher binder contents (>10%) and higher alkali concentrations in soil-geopolymer columns are required. Though few studies have used higher concentrations, this combination was limited to a small part of an elaborate study comprising of other combinations of precursors and alkaline activators and also, the brittle behaviour of stabilized material is reported, and thus fiber reinforcement has been attempted to maintain ductile behaviour. However, further studies are required to get comprehensive understanding of strength and durability of such fiber reinforced soil-geopolymer mixes for DSM applications. The bearing capacity analysis of column improved ground has been attempted by few investigators and geopolymer column stabilized soft ground requires further studies. Based on the above research gaps, suitable objectives are framed for the present work.

# Chapter – 3

# Materials and Experimental Methodology

## 3.1. Introduction

The experimental program at the outset aims at developing a sustainable alternative binder to stabilize soft clays by deep mixing. This study employs detailed laboratory testing to examine the potential of GGBS based geopolymer as an alternative to conventional binders in enhancing the engineering competence of soft clays. This chapter includes the details of materials used, their properties and the experimental procedures adopted to fulfil the research objectives.

## 3.2. Materials

The materials used in this investigation are Soft Clay, Ground Granulated Blast Furnace Slag (GGBS), Ordinary Portland Cement (OPC), Sodium Hydroxide (NaOH) and Polypropylene (PP) fibers.

### 3.2.1. Soft Clay

The locally available black cotton soil was collected from 1.0 m depth of soil bed on NIT Warangal campus, Hanamkonda district, Warangal, Telangana to prepare soft clay at the desired consistency. Basic soil tests were performed per the relevant Indian standards and the properties are presented in Table 3.1. According to IS soil Classification System, the soil is classified as 'Clay of High plasticity', i.e., CH. Table 3.2 provides the chemical composition of soil used in this study. It indicates that the soil is predominantly composed of oxides of silica (44.15%), alumina (22.2%) and iron oxide (14.9%). Figure 3.1 illustrates the morphology of the clay based on SEM analysis. The particles of clay used in this study presented almost clustered irregular shapes.

**Table 3.1. Soil properties**

| Parameters | | Value/Designation |
|---|---|---|
| Grain size distribution | | |
| | Gravel (%) | 2 |
| | Sand (%) | 21 |
| | Silt (%) | 34 |
| | Clay (%) | 43 |
| Atterberg limits | | |
| | Liquid Limit (%) | 68 |
| | Plastic Limit (%) | 22 |
| | Plasticity Index, PI (%) | 46 |
| Optimum Moisture Content, OMC (%) | | 24 |
| Maximum Dry Density, MDD (g/cc) | | 1.54 |
| Specific Gravity, $G_s$ | | 2.68 |
| IS Soil Classification | | CH |
| pH | | 7.4 |

**Table 3.2. Chemical composition of the soft soil**

| Oxides | Composition (%) |
|---|---|
| $SiO_2$ | 44.15 |
| $Al_2O_3$ | 22.2 |
| $Fe_2O_3$ | 14.9 |
| $MgO$ | 7.6 |
| $CaO$ | 7.4 |
| $Na_2O$ | 0.25 |
| $K_2O$ | 0.65 |
| $TiO_2$ | 1.75 |
| $P_2O_5$ | 0.27 |
| $MnO$ | 0.48 |
| $SO_3$ | 0.13 |

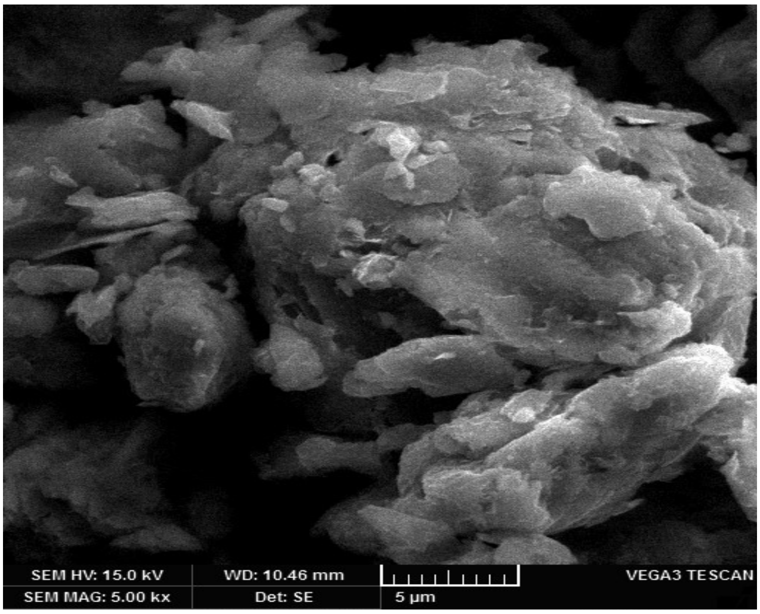



**Fig. 3.1. SEM image of the soil**

### 3.2.2. Ground granulated blast furnace slag (GGBS)

Ground granulated blast furnace slag (GGBS), a by-product from steel industry with good availability of calcium oxide, was used as the source of alumina and silica (precursors) for geopolymer binder. The GGBS powder, off-white in colour, was obtained commercially from the market. The chemical composition and physical properties of the GGBS used in the present study are presented in Tables 3.3 and 3.4 respectively.

**Table 3.3. Chemical composition of GGBS and cement**

| Oxide | | $SiO_2$ | $Al_2O_3$ | $Fe_2O_3$ | CaO | MgO | $SO_3$ | $Na_2O$ |
|---|---|---|---|---|---|---|---|---|
| Composition (%) | GGBS | 30.1 | 13.4 | 5.7 | 45.8 | 6.1 | 0 | 0.2 |
| | Cement | 20.5 | 6.3 | 4.1 | 64.1 | 3 | 1.5 | 0.4 |

**Table 3.4. Physical properties of GGBS and Cement**

| Property | GGBS | Cement |
|---|---|---|
| Specific gravity | 2.9 | 3.15 |
| Bulk density (kg/m$^3$) | 1250 | 1440 |
| Normal consistency (%) | 44 | 33 |
| Initial setting time (min.) | 20 | 42 |

### 3.2.3. Cement

53 grade Ordinary Portland Cement (OPC), obtained commercially from the market, with chemical composition as shown in Table 3.3 was used in the present study. The physical properties of the cement used are given in Table 3.4.

### 3.2.4. Sodium hydroxide (NaOH)

In the current study, Sodium hydroxide (NaOH) was chosen as the sole alkali to form geopolymer due to its superior efficacy in the dissolution of minerals compared to other alkalis. Sodium hydroxide, also known as Caustic Soda, is purchased from Fisher Scientific Pvt. Ltd. in laboratory grade pellets form with 98% purity (Fig. 3.2a). A desirable concentration range for NaOH in terms of strength improvement has been reported to be between 4.5M and 15M, with the most efficient range being between 8M and 12M (Nematollahi and Sanjayan, 2014; Rios et al., 2017). Thus, in the present study, NaOH solutions of 8M, 10M and 12M concentrations are used. The NaOH pellets are mixed with water of predetermined quantity to prepare NaOH solution of required molarity. To prepare 8M NaOH solution, 320 grams of NaOH pellets were weighed and dissolved in some distilled water in a glass jar or container and left for hydration to take place. The solution thus prepared is placed in a water bath until the heat of hydration is liberated and was kept undisturbed at room temperature of 25 ± 2 °C and relative humidity of 60 ± 5% for 24 hours before its use. Then, the solution thus obtained is transferred into a volumetric flask adding enough distilled water to it to make 1 liter of 8M NaOH solution as shown in Fig. 3.2b. The NaOH pellets used have a specific gravity of 2.1. Due to the unpredictable fluctuations in the field water content, the molarity of the alkali in the geopolymer would be altered when added to the soil (Cristelo et al., 2013; Heah et al., 2012; Phetchuay et al., 2016). Hence, in the present study different measurements of soil water content, alkali concentration and alkali content are employed, which are elaborated upon in subsequent chapters, to examine potential variations in the strength development.

(a) (b)

**Fig. 3.2. Sodium Hydroxide (NaOH) (a) pellets (b) 8M solution**

### 3.2.5. Polypropylene (PP) fibers

The Recron 3S PP fibers from Reliance Industries were used in the present study. The properties of the PP fibers as obtained from the manufacturer are given in Table 3.5. Figure 3.3 displays a picture of discrete PP fibers of 12 mm length used in this study. The PP fiber dosages used by previous researchers for soil improvement range from 0.1 to 1.2% by weight of dry soil (Khattak & Alrashidi, 2006; Fatahi et al., 2013; Olgun, 2013; Ayeldeen & Kitazume, 2017; Tripathi et al., 2020). The PP fiber contents of 0.25, 0.5, 0.75 and 1% by weight of dry soil are selected for the present study.

**Table 3.5. Properties of polypropylene fibers used**

| Property | Value |
|---|---:|
| Length (mm) | 12 |
| Diameter (mm) | 0.035 |
| Aspect Ratio | 343 |
| Tensile Strength (MPa) | 560 |
| Elastic Modulus (GPa) | 4.5 |
| Specific Gravity | 0.91 |

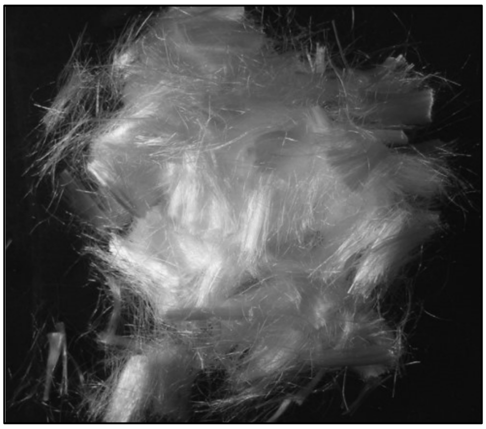


**Fig. 3.3 Polypropylene fibers**

## 3.3. Mix proportioning

To investigate the efficacy of the PP fiber reinforced GGBS geopolymer in stabilizing the soft soil for DSM applications, the experimental investigation was divided into four phases, as shown in Fig. 3.4. In first phase of the investigation, the efficiency of the GGBS geopolymer synthesized with binder contents ranging from 10 – 30%; NaOH molarity ranging from 8M to 12M; alkali/binder (A/B) ratio ranging from 0.5 – 1.0 and curing period ranging from 3 to 28 days, was studied by testing the stabilized specimens for unconfined compressive strength (UCS) and flexural strength (FS). The results thus obtained were compared with that of the specimens stabilized with cement at water/cement ($w$/C) ratio of 0.4 and cement content ranging from 10 – 30%. The optimum mix proportions from the first phase are used for the second and third phases of study. In second phase, the PP fibers, with fiber contents ranging from 0.25 – 1.0%, were added to the optimized GGBS geopolymer stabilized soil mixes and the specimens thus prepared and cured for 28 days were tested for UCS and FS. The results from the tests were compared with that of the unreinforced geopolymer stabilized soil mixes. The mix proportions used for the third phase of study are similar to the second phase. In the third phase of study, the unreinforced and PP fiber reinforced geopolymer stabilized soil mix specimens were tested for durability against wetting and drying. The initial soil water contents adopted for all the three phases were $0.75w_L$, $1.0w_L$ and $1.25w_L$ representing soft (S), very soft (V) and liquid (L) consistencies ($w_L$ is liquid limit of the soil). The mix designations adopted for all the mixes are given in Chapter 4. In the fourth phase of the study, the results from the three phases are substantiated by evaluating the load capacity of the model soft clay bed reinforced with single and group of soil-geopolymer (with and without fibers) and soil-cement columns in end bearing and floating conditions using test tank at soil water content of $0.75w_L$, NaOH molarity of 10M, A/B ratio of 0.75, fiber content of 1% and binder contents of 20 and 30%.

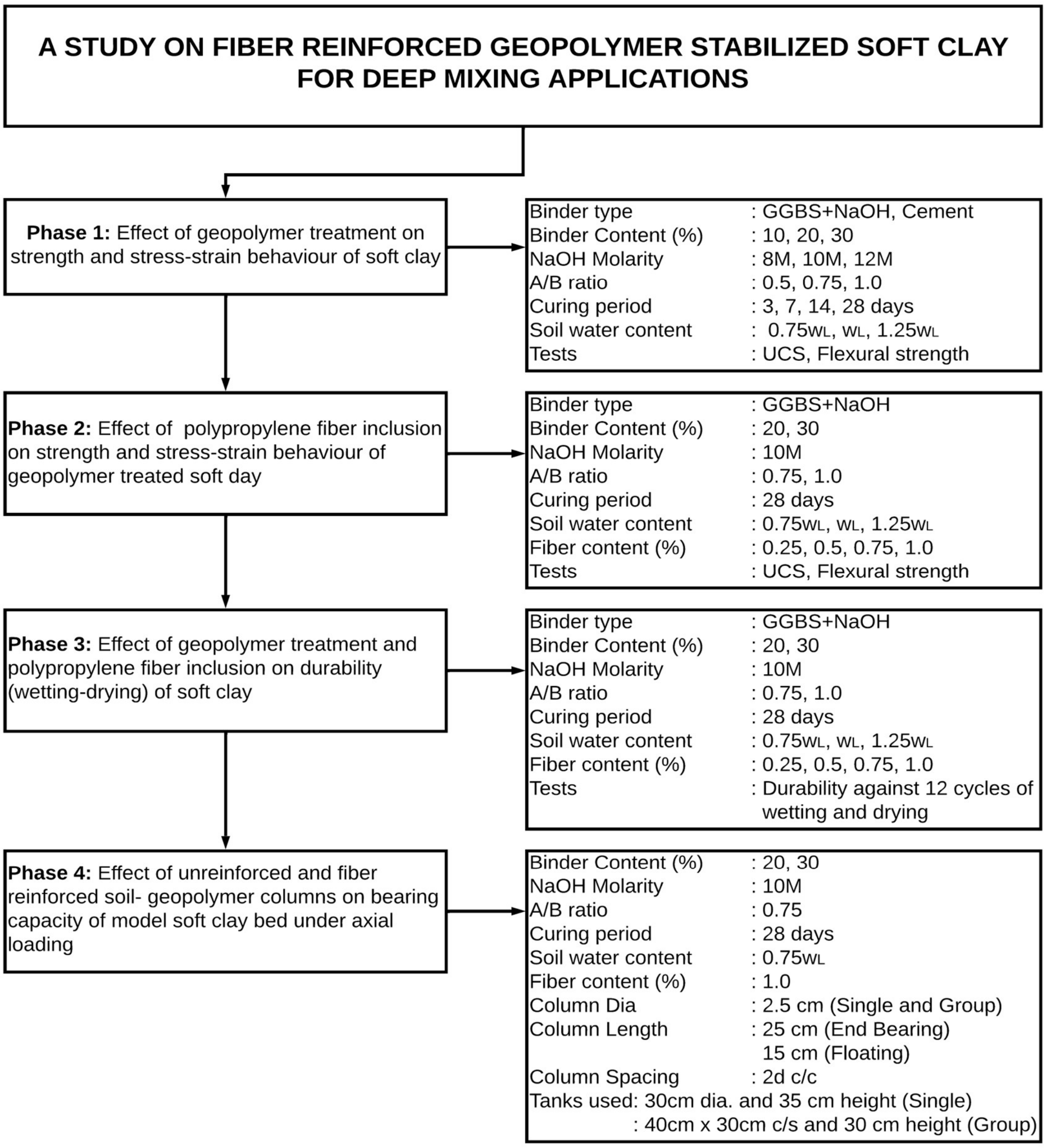



**Fig. 3.4. Methodology adopted for the present study**

## 3.4. Specimen preparation and curing

The mixing procedure employed was the same for preparing the stabilized test specimens for all the tests. Firstly, the dry soil (passing through 2 mm IS sieve) and potable water were weighed separately and mixed in a container which was then kept in an airtight container for 24 hours to prevent the escape of moisture and to ensure uniform water absorption by the soil. NaOH solutions of different molarities were prepared as mentioned earlier. At the time of preparation of samples, GGBS required for the desired mix is first added to the respective amount of the NaOH solution and blended thoroughly to avoid the formation of lumps, thus making a smooth paste or slurry. For example, to stabilize 100 grams of dry soil, 10 grams of GGBS and 10 grams of NaOH solution were mixed together representing 10% binder content and A/B ratio of 1.0. Then, the binder slurry is immediately transferred to the container or bowl containing previously prepared wet soil and mixed thoroughly until uniformity is reached. As soon as the GGBS comes in contact with the alkali, the reaction process starts and a change in the consistency (loose to thick) of the paste can be observed during the mixing. Also, thickening of the soft soil can be observed when the geopolymer paste is blended in it. This visual observation can be considered as the physical evidence for the initiation of reactions at an early stage. The soil mixtures thus prepared in various mix proportions of materials are filled in the respective greased moulds of the UCS, flexural strength, and durability tests. Cylindrical PVC moulds of 50 mm diameter and 100 mm length with length/diameter ratio of 2 were used to prepare specimens for UCS and durability tests, whereas rectangular steel moulds were used to prepare specimens of size 50 mm x 50 mm x 200 mm for flexural strength tests. For PP fiber reinforced soil-geopolymer specimens, PP fibers were added to the binder slurry and mixed thoroughly before blending it into the soft soil. The cement stabilized soil specimens were also prepared following the same procedure mentioned above by mixing cement paste (made with water/cement ratio, i.e. *w*/C ratio of 0.4) in the wet soil. Due to the higher water contents used in this study, the generated mixes were in a state of low viscosity. The mixes were poured into the moulds in two increments. After each placement, the moulds were tapped 25 times on the platform to eliminate any trapped air within the mixes. Some mixes exhibited a quasi-plastic condition as a result of the higher alkali concentration employed or lower water contents. The specimens of these mixes were made by placing the mixture into the mould in three layers, giving 25 blows for each layer with a small tamping rod, to attain unit weights that match those of the low-viscosity mixes. To ensure consistency, the unit weights of the specimens of all mixes were verified. After levelling the surface of the moulded samples, they were labelled and secured in sealed plastic covers and kept for curing in desiccators to avoid escape of moisture. After 24 hours, the treated samples extracted from the moulds are again placed in the desiccators for curing at average temperature and average humidity of 25 ± 2 °C and 60 ± 5%, respectively, for the required curing period. Fig. 3.5 shows the procedure adopted for preparation of test specimens.

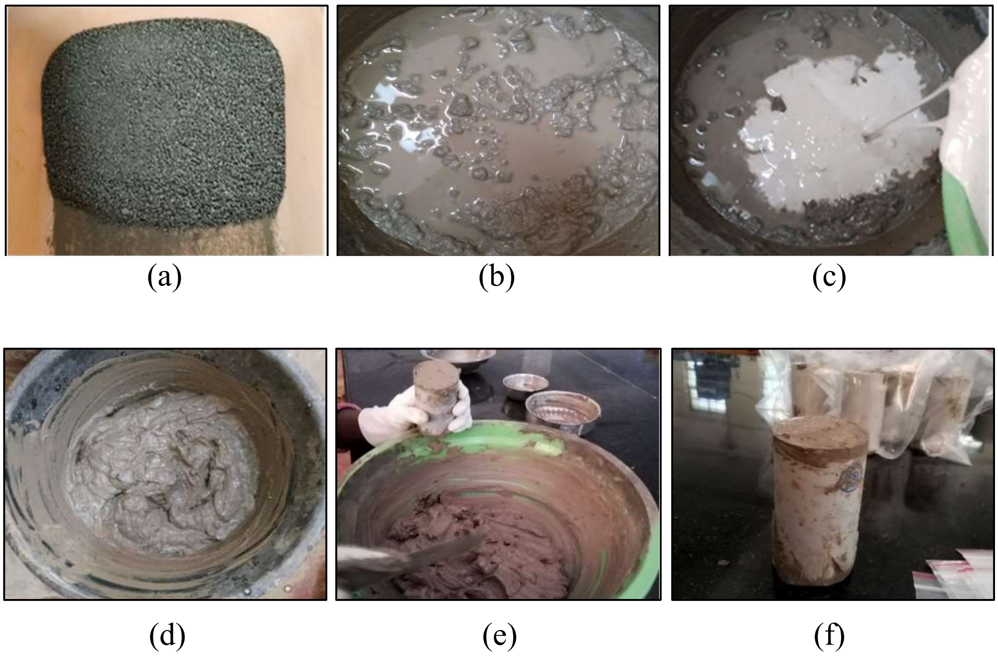


Fig. 3.5. Procedure for preparation of test specimens (a) dry soil (b) wet soil at 1.25 $w_L$ (c) pouring geopolymer slurry into wet soil (d) wet soil blended with geopolymer (e) filling PVC moulds with soil-geopolymer (f) prepared specimen in mould

## 3.5. Testing methodology

In the current research work, strength, durability and microstructural characteristics of soil stabilized with GGBS geopolymer and cement are investigated. Apart from these tests, the use of synthesized GGBS geopolymer in DSM applications is assessed by conducting model tests in test tank. This section deals with the testing methodology adopted in the present study.

### 3.5.1. Unconfined Compressive Strength (UCS) test

The unconfined compressive strength (UCS) test is an easy and quick test to estimate the effect of several variables such as stabilizer type and quantity on the strength improvement of treated soil. The UCS test specimens were prepared with dimensions of 50 mm diameter and 100 mm length with length to diameter ratio of 2.0 as per IS: 2720 (Part 10): 1991. The UCS of the hardened specimens was tested as shown in Fig. 3.6 at a displacement rate of 1 mm/min after curing period. For each combination of mixes, a minimum of three specimens were made and tested. The UCS result reported is the average of the three test specimens. Any specimen with a coefficient of variation (COV) value beyond 10% was discarded.

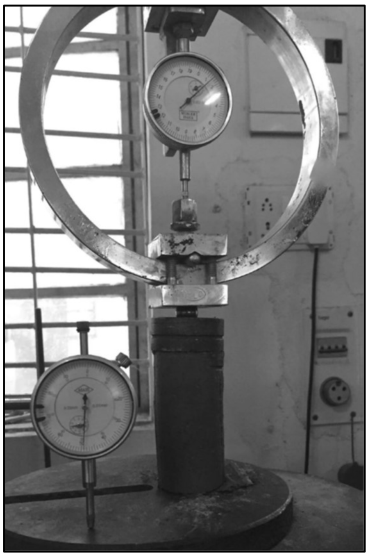


**Fig. 3.6. UCS testing of stabilized soil specimens**

### 3.5.2. Flexural strength (FS) test or Four-Point Flexural Beam Test

For flexural strength test, treated specimens of dimensions 50 × 50 × 200 mm were prepared in rectangular steel moulds and then extracted and secured in desiccators for 28 days curing period as mentioned in the previous section. The cured beam specimens were subjected to loading using flexural testing apparatus (Fig. 3.7) at a displacement rate of 1.2 mm/minute as per IS: 4332 (Part VI):1972. The span between the supports is 150 mm and that between the loading points on the beam is 50 mm.

From the test data, the modulus of rupture (R) or flexural strength (FS) values of the stabilized beam specimens were calculated. The two loading points divide the beam span between the supports into three equal parts. If the failure plane occurred within the central third part of the span length, then the following formula is used to calculate the flexural strength of the beam specimens.

$$R = \frac{PL}{bd^2} \quad (3.1)$$

The following formula is used to calculate the flexural strength of the beam specimens if the failure plane was not more than 5% of the span length outside the center third part of the span length.

$$R = \frac{3Pa}{bd^2} \quad (3.2)$$

'P' is the failure load in N, 'L' is the span length between the two lower supports as shown in the apparatus, '$b$' is the specimen width in mm, '$d$' is the specimen depth in mm and '$a$' is the distance between failure plane and the nearest support in mm, measured at the beam's bottom surface near the centre line.

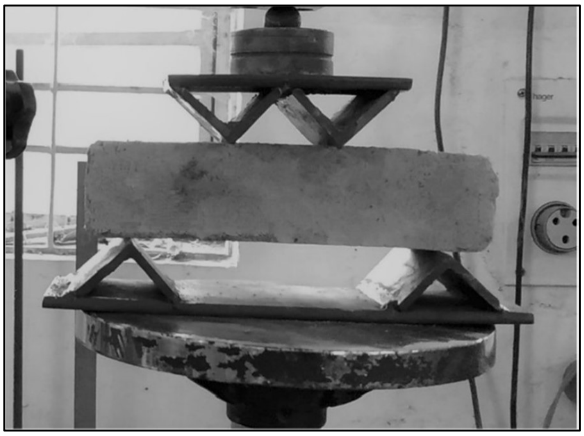


**Fig. 3.7. Flexural strength testing of stabilized soil specimens**

### 3.5.3. Durability tests

The durability tests were carried out for 12 cycles of wetting and drying (w-d) at the end of 28 days curing period. Firstly, specimens were immersed in water for wetting (Fig. 3.6a) for a duration of 5 hours and then taken out, surface dried and weighed. Then, the specimens were oven dried at 80°C for 42 hours, followed by cooling them to room temperature for one hour before weighing them for mass loss thus completing one cycle of wetting-drying (w-d) for 48 hours as per IS 4332 (Part IV): 1968. The stabilized soil specimens were subjected to 12 such w-d cycles, resulting in a total curing time of 52 days. The specimens were immersed in water for two hours at the end of 1, 3, 6, 9 and 12 w-d cycles before testing for residual UCS after the drying and cooling down period. The mass loss, volume change and residual UCS of the stabilized soil specimens were evaluated for all w-d cycles using separate sets of specimens.

(a) (b)

**Fig. 3.8. Treated specimens subjected to wetting-drying cycles (a) during wetting (b) before oven drying**

### 3.5.4. Microstructural analysis

#### 3.5.4.1. Scanning Electron Microscopy (SEM)

Due to the fact that geopolymer stabilization makes use of geopolymer for the purpose of stabilization, the modification of the material chemical composition is what causes the improvement in the engineering properties of the material. To investigate the alterations in morphology, chemical and mineralogical composition, and the chemical structure of the material, a microstructure analysis of the selected samples is performed. This examination is carried out by a Scanning Electron Microscope.

In the current study, SEM has been used to understand the structure of selected geopolymer and cement stabilized soil specimens. The soil samples collected from the failed UCS specimens were oven dried, crushed and sieved through 75 microns sieve. The dry sample passing through 75 microns sieve is gold-coated and placed in the SEM device for imaging. The SEM technique gives the image of stabilized samples at different resolutions. In the present work, the SEM images of the samples were collected at a resolution of 2k and 10 micrometers. The image thus obtained shows the morphology of the stabilized soil sample including the presence of voids or closely packed structures which helps in analyzing the strength attributing structure. The soft clay particles presented almost clustered irregular shapes (Fig. 3.1). The GGBS and cement particles resembled having semi-polygonal shapes with smooth surfaces.

#### 3.5.4.2. Energy Dispersive X-ray Spectroscopy (EDS)

It helps to identify the alteration of clay minerals or the formation of new mineral phases resulting from the geopolymerization process. This information is crucial in understanding the mechanisms of geopolymer stabilization and how it affects the properties of treated soils. By combining SEM and EDS data, a comprehensive understanding of the changes in microstructure and composition occurring within the soil-geopolymer system can be obtained. EDS is employed to determine the composition of the materials used and the stabilized soil. The EDS tests were conducted on the same samples prepared for SEM analysis using EDS device attached to the SEM device. The EDS results indicated that soft clay is rich in silicon (44.15 %), aluminium (22.2%) and Iron (14.9%); whereas GGBS was rich in calcium (45.8%), silicon (30.1%) and aluminium (13.4%); while cement is rich in calcium (64.1%) as shown in Tables 3.2 and 3.3.

### 3.5.5. Model tests in test tank

Evaluating the performance of real-life structures is challenging due to the significant investment of time and financial resources it requires. However, conducting extensive field tests incurs significant costs, resulting in a limited number of such field tests reported in the literature. Considering this, scaled-down laboratory physical model tests are the preferable option. They are essential for advancing our understanding of geotechnical phenomena. The results from model tests yield vital insights into the behaviour of the prototype ground and serve to validate analytical and numerical findings.

In the current study, laboratory scale model tests are conducted on soft soil bed which is improved with single and groups of end-bearing and floating soil-cement and soil-geopolymer columns with and without fiber reinforcement. The model tests examine the effect of parameters such as binder type, binder content, number of columns, column end condition, and fiber inclusion, on the load capacity of the composite ground under axial loading. The test programme, model test set-up, soft clay bed preparation are described in this section.

#### 3.5.5.1. Test program and model test set-up

For preparing soil-cement columns, *w*/C ratio is taken as 0.4; and A/B ratio is taken as 0.75 for soil-geopolymer columns. The columns for model study were made with binder contents of 20% and 30% and fiber content of 1% by dry weight of soil. A total of 37 model tests were conducted in which one control model test was conducted on soft virgin clay bed; 18 model tests each were performed on composite bed improved with end-bearing and floating columns conditions.

The diameter of the columns in the model tests was selected as 25 mm. The experiments were conducted on columns with lengths of 250 mm and 150 mm, under end bearing and floating conditions, respectively. The diameter of soil-geopolymer columns in practice typically ranges from 0.5 to 1.75 m, with a center-to-center spacing of 1 to 1.5 m and the length varies between 10 to 30 m for end bearing columns (Larsson et al., 2005). The ratio of column length to diameter, denoted as l/d, was chosen as 10 and 6 for end bearing columns and floating columns respectively. The scaling factor for length and diameter of the soil-geopolymer columns was taken as 1/40 according to the scaling laws mentioned in Wood, D.M. (2017). Thus, the model soil-geopolymer columns with length and diameter of 250 mm and 25 mm respectively in this study represent prototype soil-geopolymer columns with length and diameter of 10 m and 1 m respectively in field. The depth of the soft clay bed was kept constant at 250 mm during all the tests.

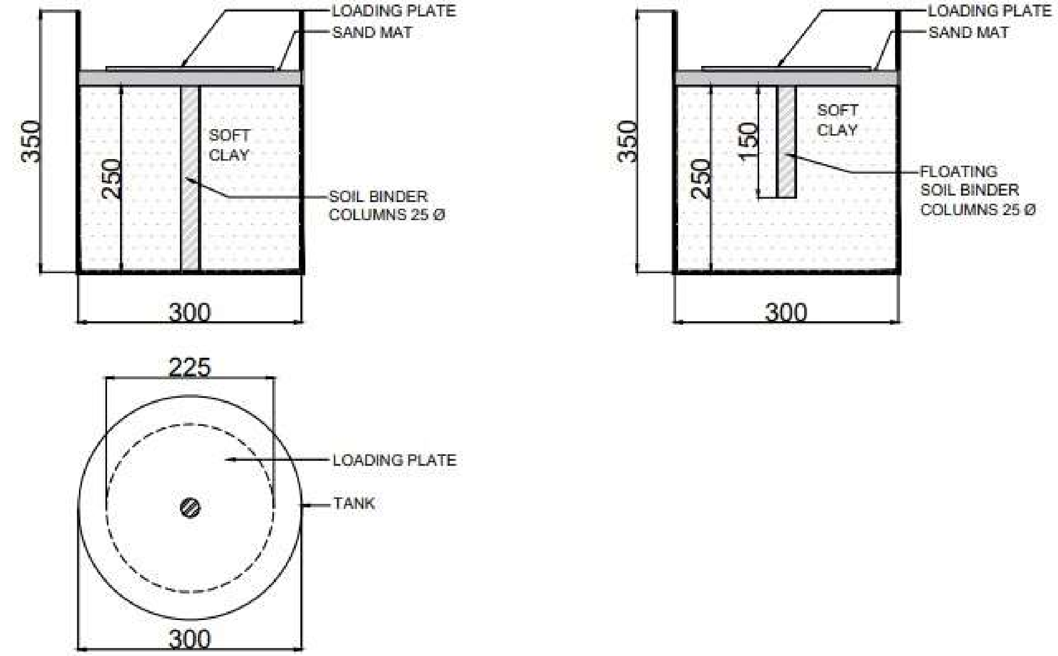



**Fig. 3.9. Test details on the improved soft clay bed with single column (a) Section – end bearing column (b) Section – floating column (c) Elevation – All dimensions in mm**

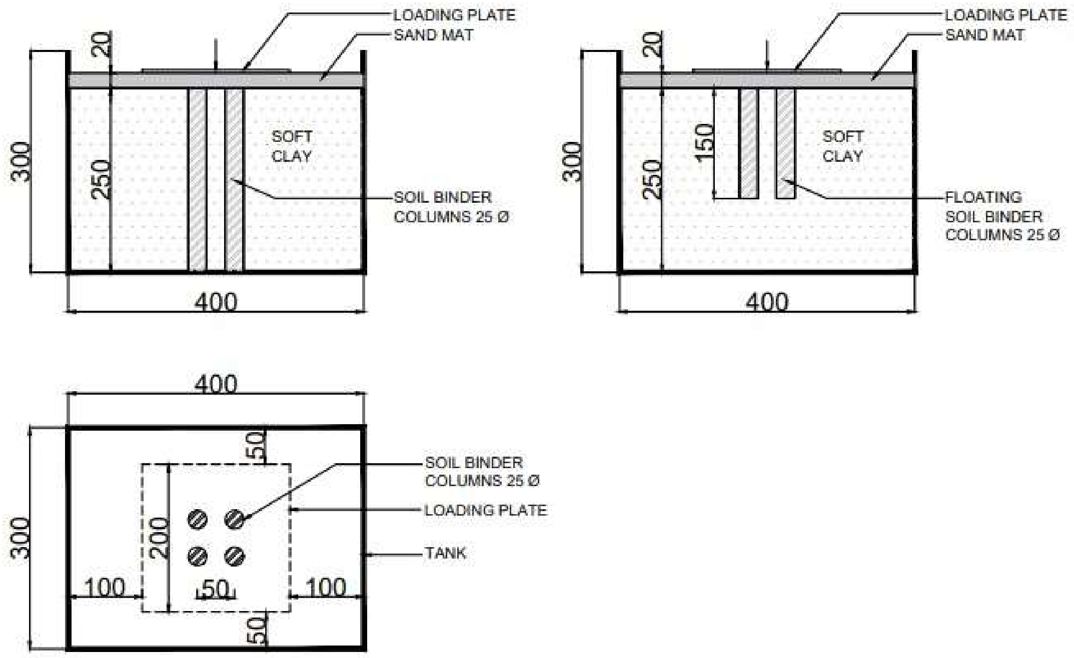



**Fig. 3.10. Test details on the improved soft clay bed with 2x2 group columns (a) Section – end bearing column (b) Section – floating column (c) Elevation – All dimensions in mm**

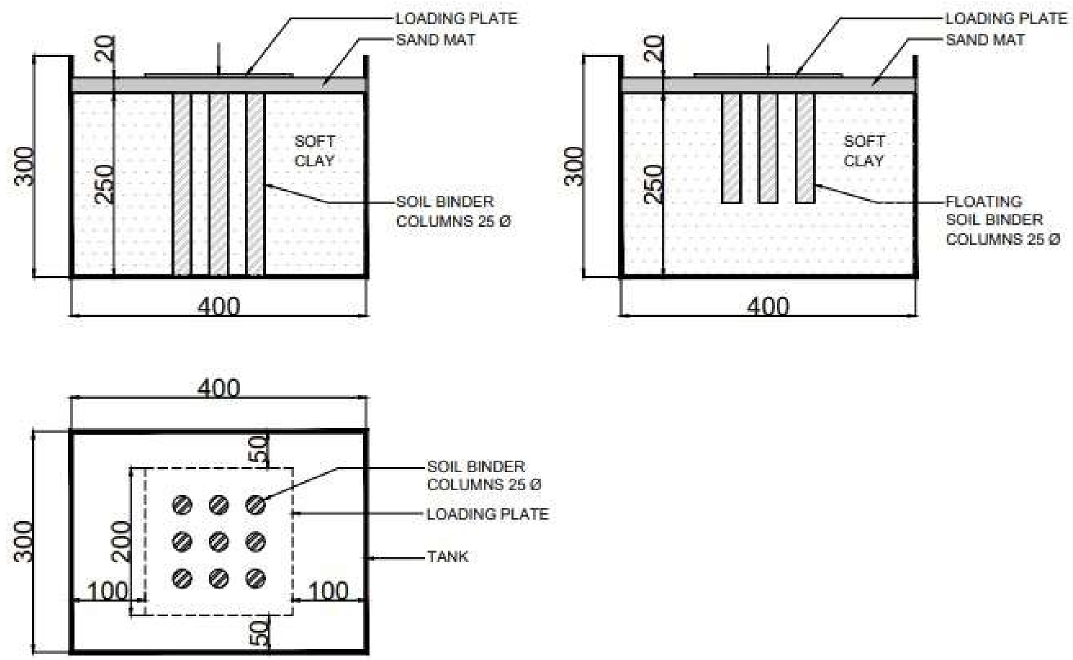



**Fig. 3.11. Test details on the improved soft clay bed with 3x3 group columns (a) Section – end bearing column (b) Section – floating column (c) Elevation – All dimensions in mm**

Tests on group of columns were conducted in a rectangular 450 mm x 300 mm x 300 mm high tank; and that on single columns were conducted in a cylindrical 300 mm diameter 350 mm high tank. Thin open-ended PVC pipes of 25 mm outer diameter and 1 mm thickness were used for casting of soil-cement and soil-geopolymer columns. The test details of the end bearing and floating column reinforced model soft clay bed are shown in Figures 3.9 to 3.11.

### 3.5.5.2. Preparation of model soft clay bed

To prepare a model soft clay bed, the quantity of water that was required to be mixed with the dry soil was taken as $0.75w_L$, i.e., 58%. Dry soil passing through 2 mm sieve was thoroughly mixed with this water content to prepare a uniform mixture and left undisturbed for 24 hours for proper water absorption. Prior to filling in the test tank, a thin layer of grease was applied to the inside surface of the tank walls to reduce friction on the side walls. The prepared soil was later carefully filled in the test tank in layers manually (Fig. 3.12a), to prevent the formation of any air voids. After the tank was filled to the desired level, it was kept undisturbed for 48 hours to restore its thixotropic strength (Fig. 3.12b). To prevent loss of moisture, the top of the tank was wrapped with moist jute bag until the initiation of column installation.

#### 3.5.5.3. Column installation in the soft clay bed and loading arrangement

Soil-cement and soil-geopolymer columns (with and without PP fiber inclusion) were installed inside the model soft clay bed in the test tanks by adopting replacement technique. The column installation procedure was the same throughout the study. A detailed procedure for installation of stabilized soil columns in the soft ground is described below.

Initially, the surface of the model soil bed was marked with the positions of the columns. Then, thin PVC pipes with a wall thickness of 1 mm and an external diameter matching the desired diameter (25 mm) of the stabilized soil columns were gradually driven into the model soft clay bed at these specific locations reaching the desired depth (250 mm for end bearing columns and 150 mm for floating columns) one at a time. A guiding arrangement, with openings corresponding to the number of columns to be installed, was employed to ensure the vertical alignment of the pipes during their insertion into the model clay bed, as illustrated in Figure 3.12(c). Prior to this, a thin coating of grease was applied to both the inner and outer surfaces of the PVC pipes to reduce friction between the pipes and the clay.

The soil within the pipes was then extracted using a ladle-like instrument, while the pipes themselves remained in position, as depicted in Figure 3.12(d). Then the mass of the extracted clay was measured, and the quantity of dry soil was computed. The extracted clay was then blended with a binder slurry containing the required quantity of binder. The paste was manually blended for approximately 3 minutes to get a consistent mixture and to prevent any soil or binder loss. Subsequently, the pipe was carefully extracted from the tank to form cylindrical apertures in increments. A small quantity of soil-cement or soil geopolymer paste was subsequently deposited into the cylindrical aperture in discrete clumps, ensuring that it could descend smoothly into the aperture without displacing the surrounding clay. The clumps within the cavity were then carefully tamped with a rod to eliminate any air voids. This process continued until the entire column was formed. Subsequently, the remaining pipes were gradually removed, and the columns were successively installed in the model soft clay bed. The model composite ground thus formed with soil-binder columns (Fig. 3.12e) was then left for moist curing for a period of 28 days. The top surface of the tank was wrapped with a moist jute cover to prevent any loss of moisture during this curing period.

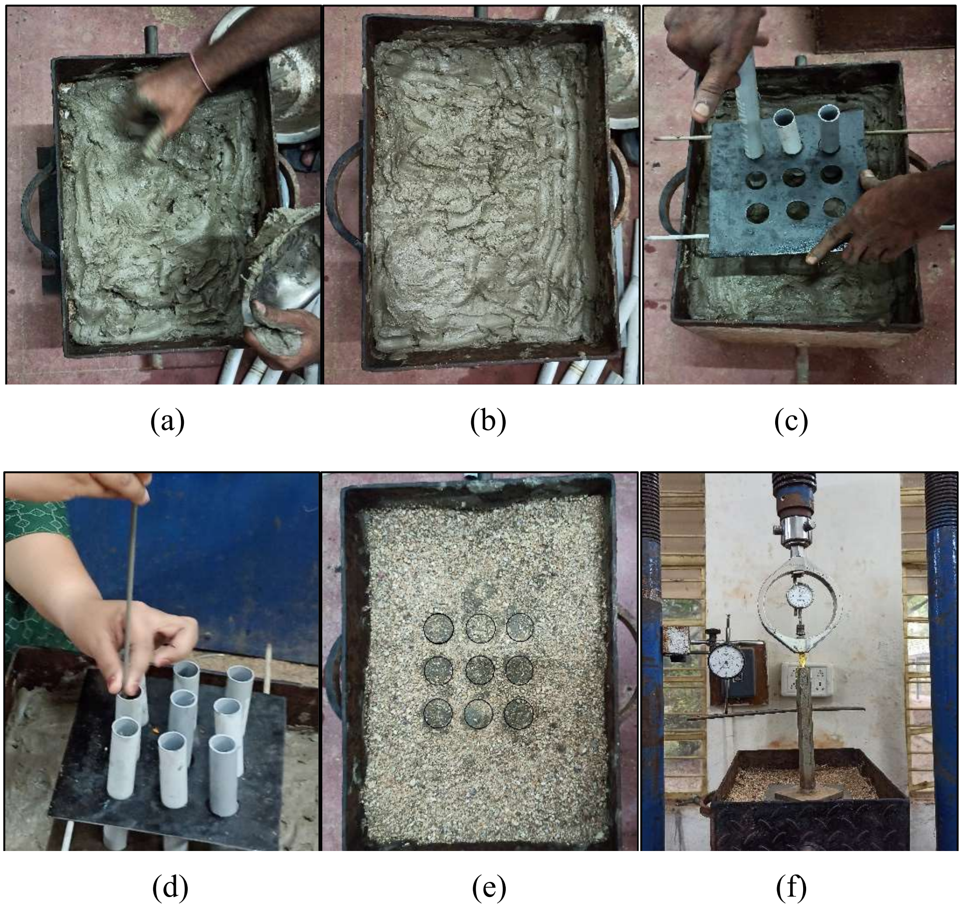



**Fig. 3.12. Preparation of model soft clay bed, installation of columns and loading arrangement**

A sand layer of 20 mm thickness was placed uniformly over the surface of the composite bed before testing. A square iron plate of dimensions 200 mm x 200 mm, resembling a model footing, was placed on the column group. For cylindrical tank containing model composite ground with single column, a circular iron plate of diameter 225 mm (having approximately same area as that of square plate) was used for this purpose. The model composite bed was subjected to a strain-controlled axial compression loading by a compression testing machine consisting of a proving ring of 20 kN capacity. The footing settlements on the model composite ground were measured using a dial gauge. All the instruments were calibrated before testing. The loading arrangement for the model composite ground is shown in Fig. 3.12f.

# Chapter – 4
# Results and Discussion

## 4.1. Introduction

The efficacy of GGBS geopolymer in soil stabilization is influenced by various factors like binder content, molarity of the alkali (NaOH), alkali to binder ratio, etc. For convenient reading, GGBS based geopolymer is mentioned as geopolymer (GP), GGBS geopolymer stabilized soil is written as soil–geopolymer, OPC treated soil is written as soil–cement and GGBS content is mentioned as binder content in this chapter. As mentioned in Chapter-2, limited literature exists on the use of GGBS as binder to prepare geopolymer for soil stabilization and deep soil mixing applications. Therefore, this chapter's primary aim is to synthesize a suitable geopolymer mix through detailed laboratory testing. Locally available black cotton soil (rich in montmorillonite mineral) was selected to prepare the soft clay for stabilization in this investigation. The experimental investigation is divided into four phases, the results of which are presented and discussed in the following sections.

## 4.2. Strength and stress–strain behaviour of soil–geopolymer

In Phase-I of the experimental methodology described in Chapter-3, the Unconfined Compressive Strength (UCS) test is conducted as an indicator for studying the effect of different variables on the strength of soil–geopolymer specimens at curing periods of 3, 7, 14 and 28 days. Using UCS test results, the role of GGBS and NaOH in enhancing the strength of the soil–geopolymer mix was examined. The strength of soil–geopolymer mixes was then compared with that of the soil–cement mixes. The flexural strength of the rectangular beam specimens prepared at the optimized mixes from UCS results was also assessed. Also, the microstructure of selected mixes was studied using Scanning Electron Microscopy (SEM) with Energy Dispersive X-ray Spectroscopy (EDS) to study the contribution of GGBS and NaOH based geopolymer in the strength enhancement of the stabilized soil.

### 4.2.1. Variables of Phase-I study and mixes used

The UCS test specimens were prepared with soil–geopolymer mixes as described in Section 3.3 of Chapter-3 for soil moisture contents of $0.75w_L$, $w_L$ and $1.25w_L$ ($w_L$ being the liquid limit of soil) indicating soft (S), very soft (V) and liquid (L) consistencies of the soil.

Based on the range of binder content considered for DSM (Bruce et al., 2013; Horpibulsuk et al., 2011a), for each soil moisture content, the binder content is varied from 10 to 30%. Also, the alkali to binder ratio (A/B ratio) is varied from 0.5 to 1.0 and the NaOH molarity is varied from 8M to 12M which is considered to be the most efficient range (Nematollahi and Sanjayan, 2014 and Rios et al., 2017) as mentioned in Table 4.1. A total of 81 soil-geopolymer mixes and 9 soil-cement mixes were prepared for UCS testing. Mix designations for all the soil–cement and soil–geopolymer mixes used in this Phase-I study are presented in Tables 4.2 and 4.3 respectively.

**Tabe 4.1. Variables of the Phase-I study**

| Parameters | Geopolymer | Cement |
|---|---|---|
| Materials | GGBS | OPC |
| Alkali or activator | NaOH | Water |
| Molarity of Alkali | 8M, 10M &12M | - |
| Binder content (%) | 10, 20 & 30 | 10, 20 & 30 |
| A/B ratio or *w*/C ratio | 0.5, 0.75 & 1.0 | 0.4 |
| Soil water content (%) | $0.75w_L$, $w_L$ &$1.25w_L$ | $0.75w_L$, $w_L$ &$1.25w_L$ |
| Strength tests | UCS and Flexural strength | UCS and Flexural strength |

**Tabe 4.2. Mix designations for Phase-I soil–cement mixes**

| Mix Designation | Soil water content | Binder Content (%) |
|---|---|---|
| C-S-B10 | $0.75\ w_L$ | 10 |
| C-S-B20 | | 20 |
| C-S-B30 | | 30 |
| C-V-B10 | $w_L$ | 10 |
| C-V-B20 | | 20 |
| C-V-B30 | | 30 |
| C-L-B10 | $1.25\ w_L$ | 10 |
| C-L-B20 | | 20 |
| C-L-B30 | | 30 |

**Tabe 4.3. Mix designations for Phase-I soil–geopolymer mixes**

| Mix Designation | Initial soil water content | NaOH Molarity | A/B ratio | Binder Content (%) |
|---|---|---|---|---|
| GP-S-8M-A0.5-B10 | 0.75 $w_L$ | 8M | 0.5 | 10 |
| GP-S-8M-A0.5-B20 | | | | 20 |
| GP-S-8M-A0.5-B30 | | | | 30 |
| GP-S-8M-A0.75-B10 | | | 0.75 | 10 |
| GP-S-8M-A0.75-B20 | | | | 20 |
| GP-S-8M-A0.75-B30 | | | | 30 |
| GP-S-8M-A1-B10 | | | 1.0 | 10 |
| GP-S-8M-A1-B20 | | | | 20 |
| GP-S-8M-A1-B30 | | | | 30 |
| GP-S-10M-A0.5-B10 | | 10M | 0.5 | 10 |
| GP-S-10M-A0.5-B20 | | | | 20 |
| GP-S-10M-A0.5-B30 | | | | 30 |
| GP-S-10M-A0.75-B10 | | | 0.75 | 10 |
| GP-S-10M-A0.75-B20 | | | | 20 |
| GP-S-10M-A0.75-B30 | | | | 30 |
| GP-S-10M-A1-B10 | | | 1.0 | 10 |
| GP-S-10M-A1-B20 | | | | 20 |
| GP-S-10M-A1-B30 | | | | 30 |
| GP-S-12M-A0.5-B10 | | 12M | 0.5 | 10 |
| GP-S-12M-A0.5-B20 | | | | 20 |
| GP-S-12M-A0.5-B30 | | | | 30 |
| GP-S-12M-A0.75-B10 | | | 0.75 | 10 |
| GP-S-12M-A0.75-B20 | | | | 20 |
| GP-S-12M-A0.75-B30 | | | | 30 |
| GP-S-12M-A1-B10 | | | 1.0 | 10 |
| GP-S-12M-A1-B20 | | | | 20 |
| GP-S-12M-A1-B30 | | | | 30 |
| GP-V-8M-A0.5-B10 | $w_L$ | 8M | 0.5 | 10 |
| GP-V-8M-A0.5-B20 | | | | 20 |
| GP-V-8M-A0.5-B30 | | | | 30 |
| GP-V-8M-A0.75-B10 | | | 0.75 | 10 |
| GP-V-8M-A0.75-B20 | | | | 20 |
| GP-V-8M-A0.75-B30 | | | | 30 |
| GP-V-8M-A1-B10 | | | 1.0 | 10 |
| GP-V-8M-A1-B20 | | | | 20 |
| GP-V-8M-A1-B30 | | | | 30 |
| GP-V-10M-A0.5-B10 | | 10M | 0.5 | 10 |
| GP-V-10M-A0.5-B20 | | | | 20 |
| GP-V-10M-A0.5-B30 | | | | 30 |

| | | | | |
|---|---|---|---|---|
| GP-V-10M-A0.75-B10 | | | 0.75 | 10 |
| GP-V-10M-A0.75-B20 | | | | 20 |
| GP-V-10M-A0.75-B30 | | | | 30 |
| GP-V-10M-A1-B10 | | | 1.0 | 10 |
| GP-V-10M-A1-B20 | | | | 20 |
| GP-V-10M-A1-B30 | | | | 30 |
| GP-V-12M-A0.5-B10 | | 12M | 0.5 | 10 |
| GP-V-12M-A0.5-B20 | | | | 20 |
| GP-V-12M-A0.5-B30 | | | | 30 |
| GP-V-12M-A0.75-B10 | | | 0.75 | 10 |
| GP-V-12M-A0.75-B20 | | | | 20 |
| GP-V-12M-A0.75-B30 | | | | 30 |
| GP-V-12M-A1-B10 | | | 1.0 | 10 |
| GP-V-12M-A1-B20 | | | | 20 |
| GP-V-12M-A1-B30 | | | | 30 |
| GP-L-8M-A0.5-B10 | 1.25 $w_L$ | 8M | 0.5 | 10 |
| GP-L-8M-A0.5-B20 | | | | 20 |
| GP-L-8M-A0.5-B30 | | | | 30 |
| GP-L-8M-A0.75-B10 | | | 0.75 | 10 |
| GP-L-8M-A0.75-B20 | | | | 20 |
| GP-L-8M-A0.75-B30 | | | | 30 |
| GP-L-8M-A1-B10 | | | 1.0 | 10 |
| GP-L-8M-A1-B20 | | | | 20 |
| GP-L-8M-A1-B30 | | | | 30 |
| GP-L-10M-A0.5-B10 | | 10M | 0.5 | 10 |
| GP-L-10M-A0.5-B20 | | | | 20 |
| GP-L-10M-A0.5-B30 | | | | 30 |
| GP-L-10M-A0.75-B10 | | | 0.75 | 10 |
| GP-L-10M-A0.75-B20 | | | | 20 |
| GP-L-10M-A0.75-B30 | | | | 30 |
| GP-L-10M-A1-B10 | | | 1.0 | 10 |
| GP-L-10M-A1-B20 | | | | 20 |
| GP-L-10M-A1-B30 | | | | 30 |
| GP-L-12M-A0.5-B10 | | 12M | 0.5 | 10 |
| GP-L-12M-A0.5-B20 | | | | 20 |
| GP-L-12M-A0.5-B30 | | | | 30 |
| GP-L-12M-A0.75-B10 | | | 0.75 | 10 |
| GP-L-12M-A0.75-B20 | | | | 20 |
| GP-L-12M-A0.75-B30 | | | | 30 |
| GP-L-12M-A1-B10 | | | 1.0 | 10 |
| GP-L-12M-A1-B20 | | | | 20 |
| GP-L-12M-A1-B30 | | | | 30 |

### 4.2.2. Unconfined compressive strength of soil–geopolymer

The influence of binder content, A/B ratio, NaOH molarity, soil water content, and curing period on the strength enhancement of soil–geopolymer specimens was investigated using UCS testing, and the results thus obtained are shown in Figures 4.1 to 4.3 for soil water contents of $0.75w_L$, $w_L$ and $1.25w_L$ respectively and discussed in this sub-section. For stabilized soil specimens, the target UCS in the range of 1.034 to 6 MPa is considered in this study for different DSM applications and the same is presented in the figures of this section for comparison.

#### 4.2.2.1. Effect of binder content

In geopolymerisation process, in the presence of sodium (Na) alkali cations, sodium aluminosilicate hydrate (NASH) geopolymeric gels are formed. As GGBS is rich in calcium (Ca), calcium silicate hydrate (CSH) gels are also formed along with NASH gels. Also, when Ca is substituted with Na in NASH gels, calcium aluminosilicate hydrate (CASH) gels are also formed in the system. This coexistence of NASH and CSH gels causes combined effect on the strength enhancement leading to denser soil–geopolymer mix with stronger bonds and thus results in significant increase in the UCS when higher contents of GGBS are used for preparing geopolymer (Xu and Van Deventer, 2000; Yang et al., 2012; Phetchuay et al., 2016). The clayey soils may also participate in geopolymeric reactions due to the existence of amorphous phases in them, leading to enhanced strength development (Hardjito and Rangan, 2005, Heah et al., 2012; Latifi et al., 2016). However, more studies are necessary to assess the extent of its impact on the process of geopolymerisation.

Figure 4.4 illustrates the variation of UCS for soil–geopolymer mixes at binder contents ranging from 10 to 30% after a curing period of 28 days for soil water content equal to liquid limit ($w_L$). The UCS values enhanced with an increase in binder content from 10% to 30% which can be attributed to the increased availability of silicon and calcium in the mix (Xu and Van Deventer, 2002). It is observed from Figures 4.1 to 4.4 that the average increase in UCS is significant (473.9%) with the increase in binder content from 10 to 20%, whereas with further increase in binder content from 20% to 30%, lower rate (45.6%) of average increase in the UCS of soil–geopolymer specimens is observed. This reduction in the rate of strength enhancement after 20% binder content may be attributed to the unreacted GGBS particles, thus not contributing to the products of geopolymerisation and causing damage to the uniform network in the internal structure (Horpibulsuk et al., 2013; Jiang et al., 2016).

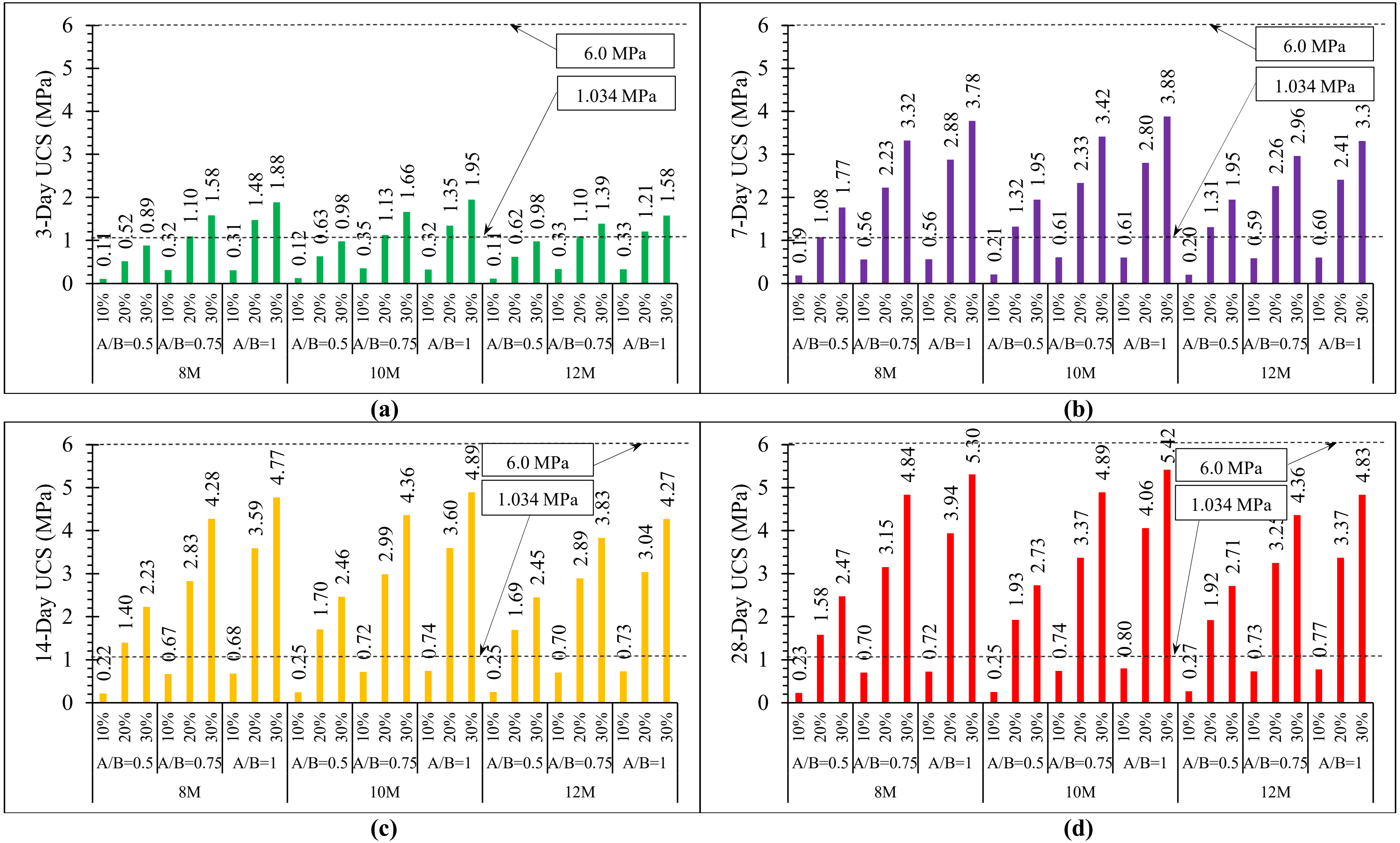



**Fig. 4.1. UCS of soil–geopolymer specimens at varying binder contents, A/B ratios and NaOH molarities at curing periods of (a) 3 days (b) 7 days (c) 14 days (d) 28 days for soil water content of 0.75$w_L$**

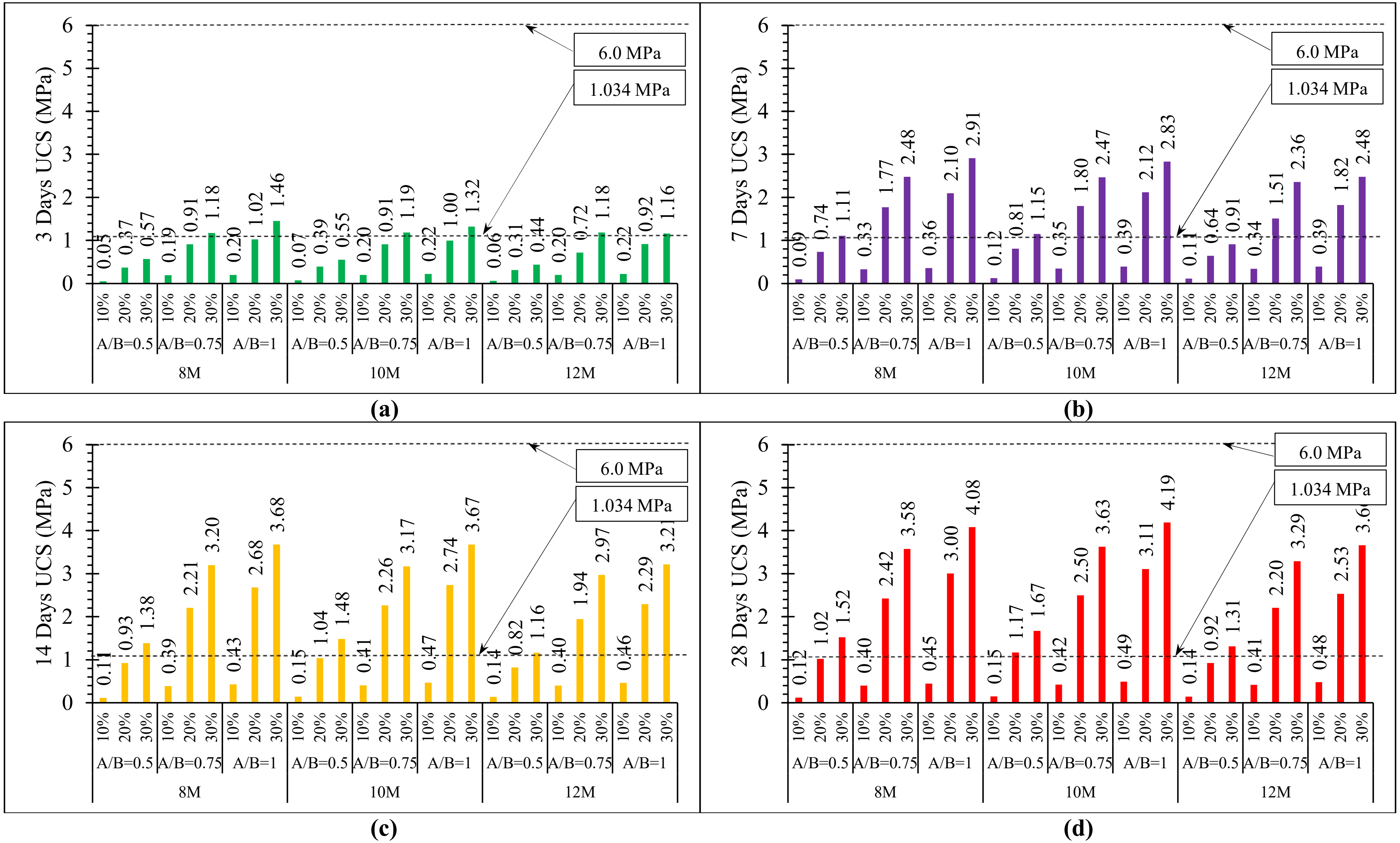
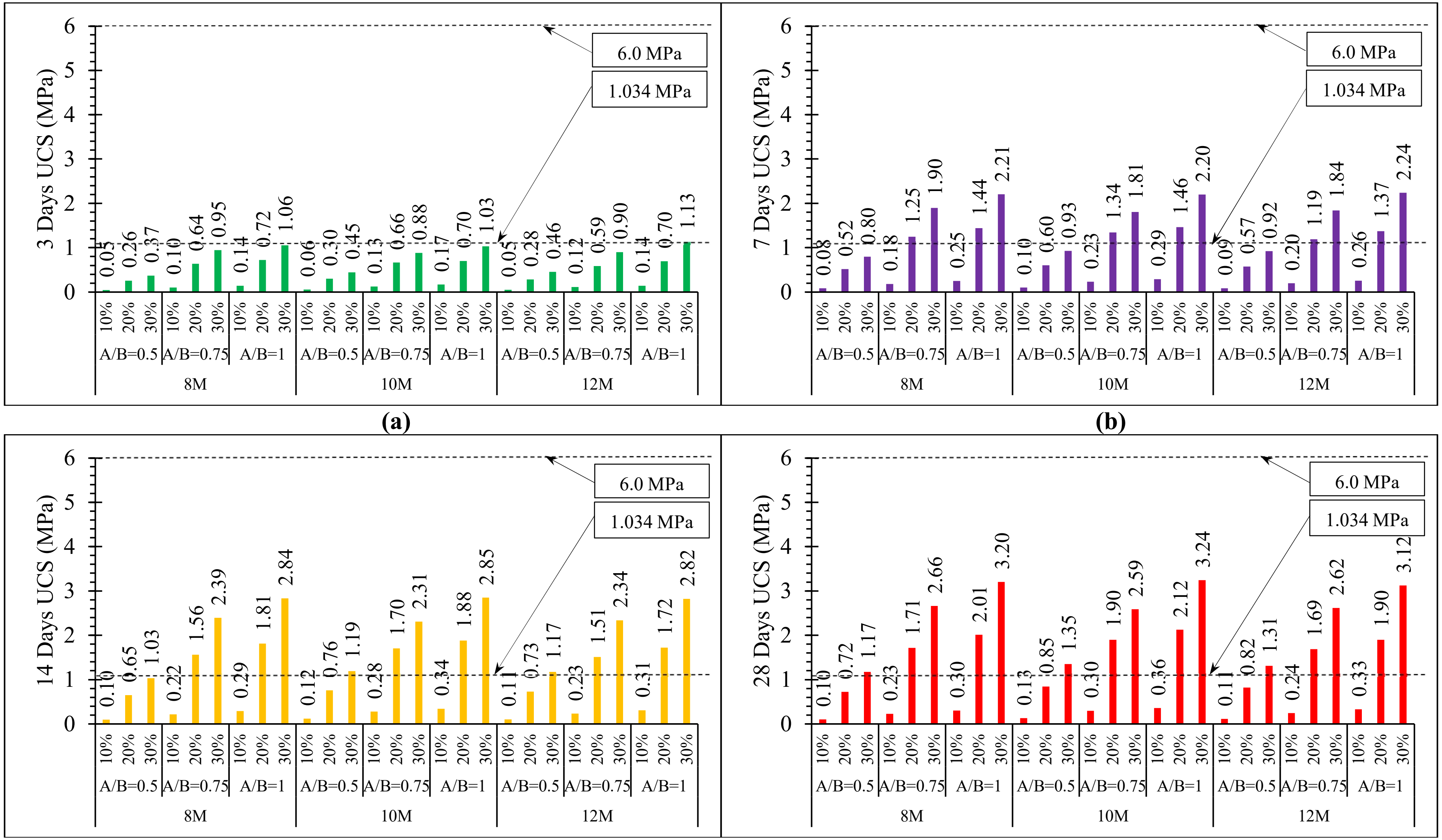



**Fig. 4.2. UCS of soil–geopolymer specimens at varying binder contents, A/B ratios and NaOH molarities at curing periods of (a) 3 days (b) 7 days (c) 14 days (d) 28 days for soil water content of $w_L$**

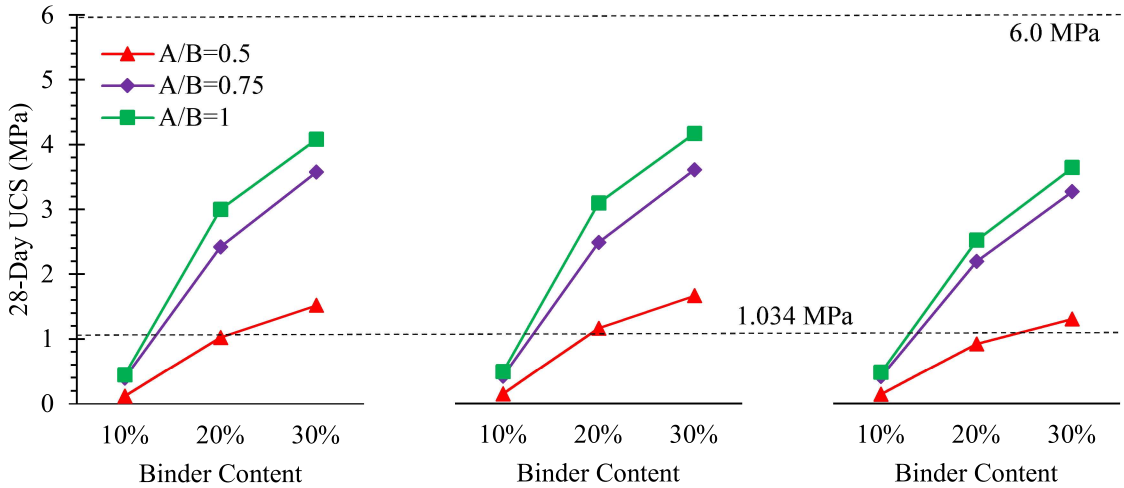



**(a)** **(b)** **(c)** **(d)**

**Fig. 4.3. UCS of soil–geopolymer specimens at varying binder contents, A/B ratios and NaOH molarities at curing periods of (a) 3 days (b) 7 days (c) 14 days (d) 28 days for soil water content of 1.25$w_L$**

However, the UCS still increases as these unreacted GGBS particles fill up the pore spaces between the clay particles and hardened geopolymerisation products.

Also, soil–geopolymer mixes made with binder content of 10% could not satisfy the target UCS criteria for DSM applications and hence is insufficient to develop geopolymeric structure with soft clay effectively. The mixes with 20% and 30% binder contents have satisfied the target UCS criteria at all A/B ratios except 0.5 where the UCS values have just reached the lower limit of the target UCS. Similar pattern of variation in UCS of the mix specimens is observed for mixes of all NaOH molarities and soil water contents with highest UCS at 30% binder content. Hence, the mixes with binder contents of 20% and 30% are considered more reliable in achieving higher strengths and achieving the target UCS requirements.




a) 8M b) 10M c) 12M

**Fig. 4.4. Variation of 28-day UCS of soil–geopolymer specimens at different binder contents and molarities**

#### 4.2.2.2. Effect of Alkali/Binder ratio

Al Bakri et al. (2012) reported an increasing trend in the UCS of flyash and slag based geopolymers by increasing the Alkali/Binder ratio. However, there is a limit for increasing this ratio as there would be decrease in UCS after that limit. Hence, as previously recommended by the researchers (Cristelo et al., 2011; Phetchuay et al., 2016), this limit for A/B ratio is taken as 1.0.

Thus, A/B ratio of 0.5, 0.75, and 1.0 is considered in this study and its effect on the UCS of soil-geopolymer specimens is investigated. Figure 4.5 shows the variation of UCS of soil-geopolymer specimens with A/B Ratio at 28 days curing period. From Figures 4.1 to 4.3

and Figure 4.5 it can be observed that with an increase in A/B ratio from 0.5 to 1.0, there is a significant increase in the UCS of the soil-geopolymer specimens. Sodium (Na) present in the alkali i.e., NaOH, combined with the dissolved silicon, aluminium, and calcium in GGBS to create monomers. The polymerization of these monomers subsequently led to the development of geopolymeric networks, thus stabilizing the soft clay and improving its UCS (Hardjito and Rangan, 2005). The presence of silicon and aluminium in the soft clay is likely to dissolve by the alkali (Yaghoubi et al., 2018) and provide subsequent contribution to the development of strength. However, their impact was not substantial due to the limited reactivity of soils (Cristelo et al., 2011; Pourakbar et al., 2016).

It is also to be noted that with an increase in A/B ratio from 0.5 to 0.75, there is an average increase of about 125% in the UCS of soil-geopolymer specimens. However, with further increase in A/B ratio from 0.75 to 1.0, a lower rate of average increase of about 15% in the UCS of soil-geopolymer specimens is observed. Similar trend in the strength gain is observed for all the mixes with binder contents of 10 to 30%, NaOH molarities of 8M to 12M, and soil water contents of $0.75w_L$ to $1.25w_L$. As the soil mixes at different water contents were fully saturated, the increase in alkali content may result in increased porosity of the mix. Therefore, at higher A/B ratio, excessive alkali content in the system may affect proper contacts between the particles in the stabilized soil system. The continuity in the geopolymerisation network might have thus got hindered, which resulted in lower rate of UCS enhancement. In the laboratory, since A/B ratio of 0.5 has resulted in a poorly workable geopolymer paste which is of kneadable consistency at higher NaOH molarities, it resulted in improper mixing of geopolymer with the saturated soil which generally requires binder at slurry consistency for DSM applications. Further, though the UCS is increased with an increase in A/B ratio at 10% binder content, it could not achieve the target UCS criteria at any A/B ratio. Although the mixes with binder contents >10% have achieved the lower limit of the target UCS at 0.5 A/B ratio, they cannot be considered reliable as the laboratory results could be thrice that of the results achieved in field (Horpibulsuk et al., 2011b). Hence, in terms of the A/B ratio, 0.5 was insufficient for complete precipitation of GGBS, and 0.75 and 1.0 seem to be the preferred A/B ratios that result in higher strengths and workable binder slurry and hence are reliable in terms of achieving the target UCS criteria for DSM applications.

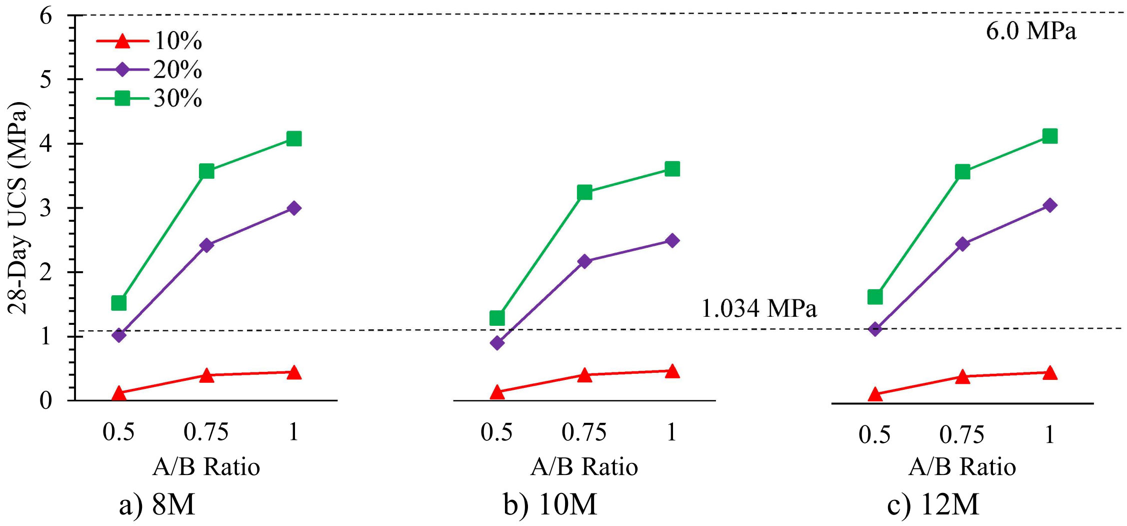



**Fig. 4.5. Variation of 28-day UCS of soil-geopolymer specimens at different A/B Ratios and molarities**

#### 4.2.2.3. Effect of NaOH molarity

The NaOH solutions of molarities 8M, 10M, and 12M are prepared as described in Chapter-3. The UCS of soil-geopolymer mix specimens prepared with this molarities is assessed and the variation of 28-day UCS of soil-geopolymer specimens with NaOH molarity is presented in Figure 4.6. From Figures 4.1 to 4.3 and Figure 4.6, it is observed that with an increase in molarity from 8M to 10M, there is a slight increase in the UCS of the specimens, whereas with further increase in the molarity from 10M to 12M, the UCS of the specimens slightly decreased. A similar trend in UCS variation is observed for curing periods of 3 to 28 days, binder contents of 10 to 30%, A/B ratios of 0.5 to 1.0, and soil water contents of $0.75w_L$ to $1.25w_L$. It is also noted that with an increase in molarity from 8M to 10M, there is an average increase of about 8.8% in the UCS of specimens. However, with further increase in molarity from 10M to 12M, an average 4.8% decrease in the UCS is observed. In general, an increase in the molarity of the alkali has the potential to reduce the UCS of the mix, as a higher molarity produces a more viscous and thicker geopolymer material. An increment in NaOH molarity results in an increase in the concentration of alkali ions within the matrix. This may result in an increased viscosity of the geopolymer paste, which could hinder the particles' movement and ability to properly pack together while mixing with the saturated soil. The formation of a compact and well-packed structure, which is essential for attaining high strength in the stabilized mixes, might be impeded by the enhanced viscosity. Furthermore, an increase in molarity may also facilitate the development of gel-like structures within the geopolymer, which might lack the strength and durability characteristics of a structure that has been fully crystallized. Although achieving the ideal molarity for maximum strength of the stabilized mix necessitates careful consideration of several parameters, including curing conditions, precursor type and content, and other additional mix design parameters, the optimal NaOH molarity is considered as 10M for the parameters considered in this study. As there is only a nominal difference in UCS of mixes with 8M and 10M NaOH, 8M can be considered as the optimal NaOH molarity, but the alkali concentration might get diluted at higher soil water contents (uncontrollable in field) resulting in an effective alkali concentration of < 8M. Thus, NaOH molarity of 10M is considered as optimal and the same is used for the mixes in the next phases of study.

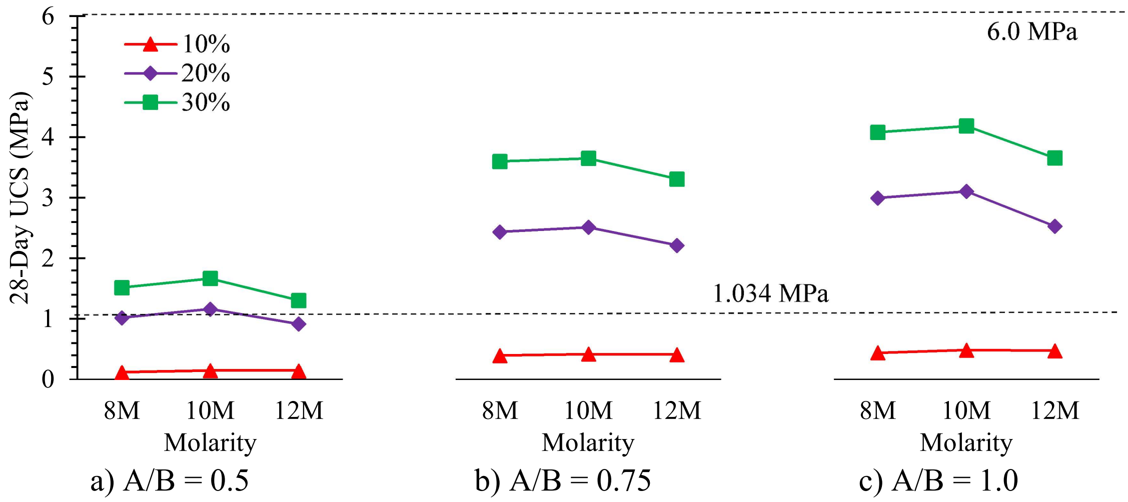



**Fig. 4.6. Variation of 28-day UCS of soil-geopolymer specimens with NaOH molarity for different A/B ratios**

#### 4.2.2.4. Effect of Soil water content

According to several previous studies (Cristelo et al., 2011; Horpibulsuk et al., 2011a; Lorenzo and Bergado, 2004; Phetchuay et al., 2016), the ideal water content for the improvement of higher water content clays (around liquid limit) is equivalent to the liquid limit of the soil. Thus, the initial soil water content prior to combining with the binders is chosen to be within the range of $0.75w_L$ to $1.25w_L$ considering the field conditions.

The effect of the initial soil water content on the UCS of soil-geopolymer mix specimens is investigated at $0.75w_L$, $w_L$, and $1.25w_L$. Fig. 4.7 shows the variation of 28-day UCS of soil-geopolymer specimens with soil water content. From Figures 4.1 to 4.3 and Figure 4.7, it is observed that with an increase in soil water content, there is a decrease in the UCS of the specimens. Similar trend in the variation of UCS of mixes is observed for curing periods of 3 to 28 days, binder contents of 10 to 30%, A/B ratios of 0.5 to 1.0, and NaOH molarities of 8M to 12M. It can also be noted that with an increase in initial soil water content from 0.75$w_L$ to $w_L$, there is an average decrease of about 32.5% in the UCS of specimens. With a further increase in initial soil water content from $w_L$ to 1.25$w_L$, there is an average decrease of about 27% in the UCS of soil-geopolymer specimens.

The decline in the UCS values resulting from increase in the soil water content through 0.75$w_L$ to 1.25$w_L$ can be attributed to two reasons. Primarily, the addition of more water may have decreased the concentration of alkali, leading to a decrease in the extent of geopolymerization. Additionally, the presence of excess water in the mix must have induced the formation of voids within the structure (Hardjito and Rangan, 2005; Phetchuay et al., 2016; Nath and Sarker, 2017). All the soil-geopolymer mixes with binder content > 10% and A/B ratio > 0.5 have achieved the target UCS requirements at all initial soil water contents under study.

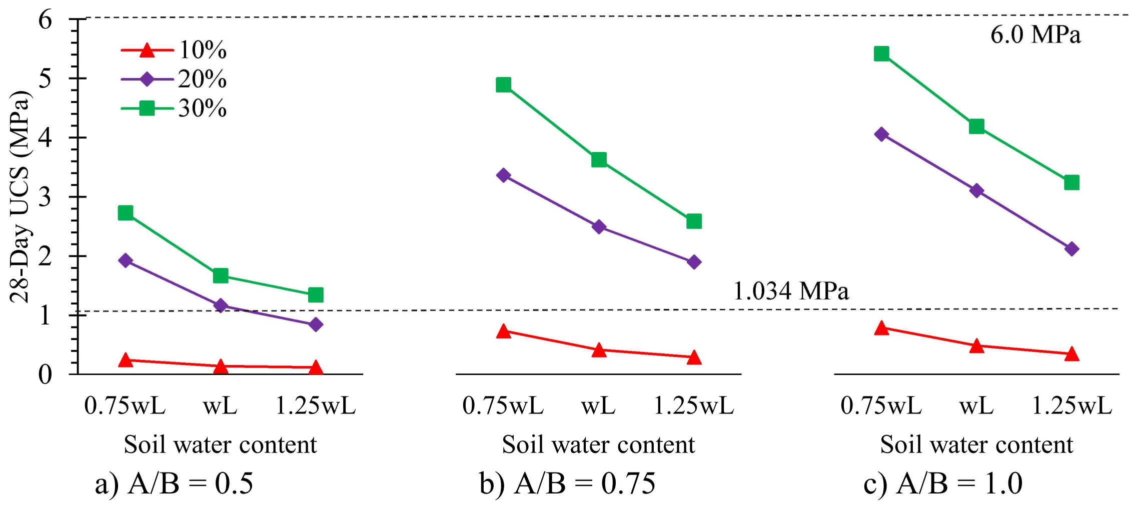



**Fig. 4.7. Variation of 28-day UCS of soil-geopolymer specimens with soil water content for different A/B ratios**

#### 4.2.2.5. Effect of curing period

The effect of curing period on the UCS of soil-geopolymer mix specimens is investigated at 3, 7, 14 and 28 days of curing. From Figs. 4.1 to 4.3, it can be observed that with an increase in curing duration, there is an increase in the UCS of the soil-geopolymer specimens. A similar trend in the UCS increase is observed for mixes with binder contents of 10 to 30%, A/B ratios of 0.5 to 1.0, molarities of 8M to 12M, and initial soil water contents of 0.75$w_L$ to 1.25$w_L$.

In addition, the mixes with binder content > 10% and A/B ratio > 0.5 have achieved the lower limit of the target UCS in just 3 days of curing at soil water content of $0.75w_L$. However, these mixes with other soil water contents reached this limit at 7 days of curing. Due to the faster reactivity of Ca compared to Si and Al in an alkaline medium, the required strength development can be achieved in a shorter curing period (Yip et al., 2005; Phetchuay et al., 2016). The preparation of geopolymer mixes with only GGBS as precursor led to elevated UCS values, even during the initial stages, owing to the abundant availability of Ca in GGBS.

***Correlation between UCS and curing time***

The development of UCS with curing time can be better understood by establishing a relationship between UCS of the specimens at 28 days curing ($UCS_{28}$) and at any other specific curing period ($UCS_D$) up to 28 days. The effect of variation in the mix combinations is eliminated by normalizing all the UCS values with respect to $UCS_{28}$ values (Horpibulsuk et al., 2011a; Phetchuay et al., 2016; Rios et al., 2017). The $UCS_D/UCS_{28}$ values thus obtained were plotted against curing time for all the mixes as shown in Fig. 4.8.

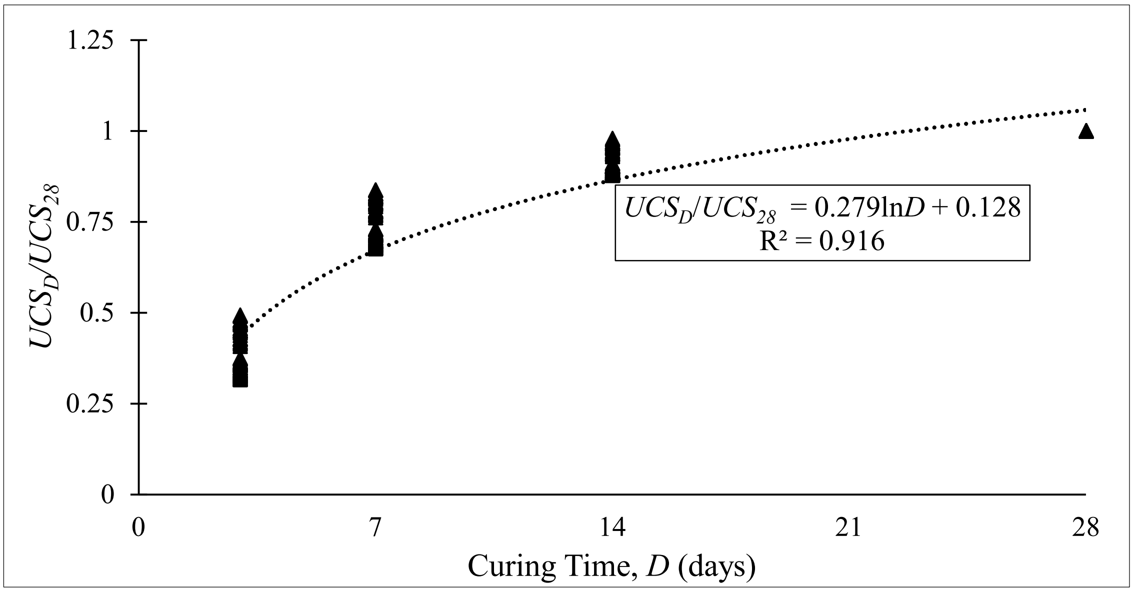



**Fig. 4.8. Variation of $UCS_D/UCS_{28}$ with curing time for soil-geopolymer mixes and its correlation**

The UCS of soil-geopolymer specimens at a particular curing time, D (within the range of 3 to 28 days) can be predicted with a high coefficient of determination, using Equation 4.1. The equation presented here bears resemblance to the equations proposed by Rios et al. (2017), Phetchuay et al. (2016), Horpibulsuk et al. (2011a), and Yaghoubi et al.

(2018) regarding soils that are stabilized using flyash-based, flyash and slag-based geopolymers and cement. For comparison, the equations from these investigations are also given below.

$$UCS_D/UCS_{28} = 0.279 \ln D + 0.128 \qquad \text{------ Eq. (4.1)}$$

$UCS_D/UCS_{28} = 0.293 \ln D + 0.026$ (Horpibulsuk et al., 2011)

$UCS_D/UCS_{28} = 0.269 \ln D$ (Phetchuay et al., 2016)

$UCS_D/UCS_{28} = 0.4441 \ln D - 0.4612$ (Rios et al., 2017)

$UCS_D/UCS_{28} = 0.334 \ln D - 0.1$ (Yaghoubi et al., 2018)

## 4.2.3. Unconfined compressive strength of soil-cement

The influence of cement content, initial soil water content and curing period on the strength of soil– cement specimens was investigated using UCS testing, and the results thus obtained are shown in Figures 4.1 to 4.3 for soil water contents of $0.75w_L$, $w_L$ and $1.25w_L$ respectively and discussed in this sub-section. For stabilized soil specimens, the target UCS in the range of 1.034 to 6 MPa is considered in this study for use in DSM applications and the same is presented in the figures of this section for comparison.

### 4.2.3.1. Effect of cement content and curing period

The soil-cement specimens were prepared with cement contents ranging from 10 – 30% and water/cement ($w$/C) ratio of 0.5 (for cement slurry) as described in Chapter-3 and tested for their unconfined compressive strength. Fig. 4.9 shows the effect of cement content and curing period on the UCS of soil-cement specimens for different initial soil water contents. Fig. 4.10 shows the variation of 28-day UCS of soil-cement specimens with cement content. It is observed that with an increase in cement content from 10% to 20%, there is an average increase of about 157% in the UCS, whereas with further increase in cement content from 20% to 30%, there is an average increase of about 23% in the UCS of soil-cement specimens. A similar trend in the UCS enhancement is observed for all curing periods and initial soil water contents. Also, almost all the mixes with 20% and 30% cement content reached the lower limit of target UCS (1.034 MPa) after 28 days of curing. From Figure 4.9, it is understood that only the soil-cement mixes with cement content 30% could achieve the lower limit of the target UCS in early stages of curing for all soil water contents except

0.75$w_L$ at which soil-cement mixes with even 20% cement content reached that criterion at 7 days of curing. However, even at higher cement contents, UCS of soil-cement specimens beyond 2 MPa could not be obtained. Thus, from the results of UCS tests on soil-geopolymer and soil-cement mix specimens, it can be inferred that soil-geopolymer mix specimens exhibited higher strengths than the soil-cement mix specimens. This may be attributed to the combined geopolymeric and pozzolanic reactions that occur in the soil-geopolymer mixes (Yip et al., 2005; Al Bakri et al., 2013; Nath and Sarker, 2017).

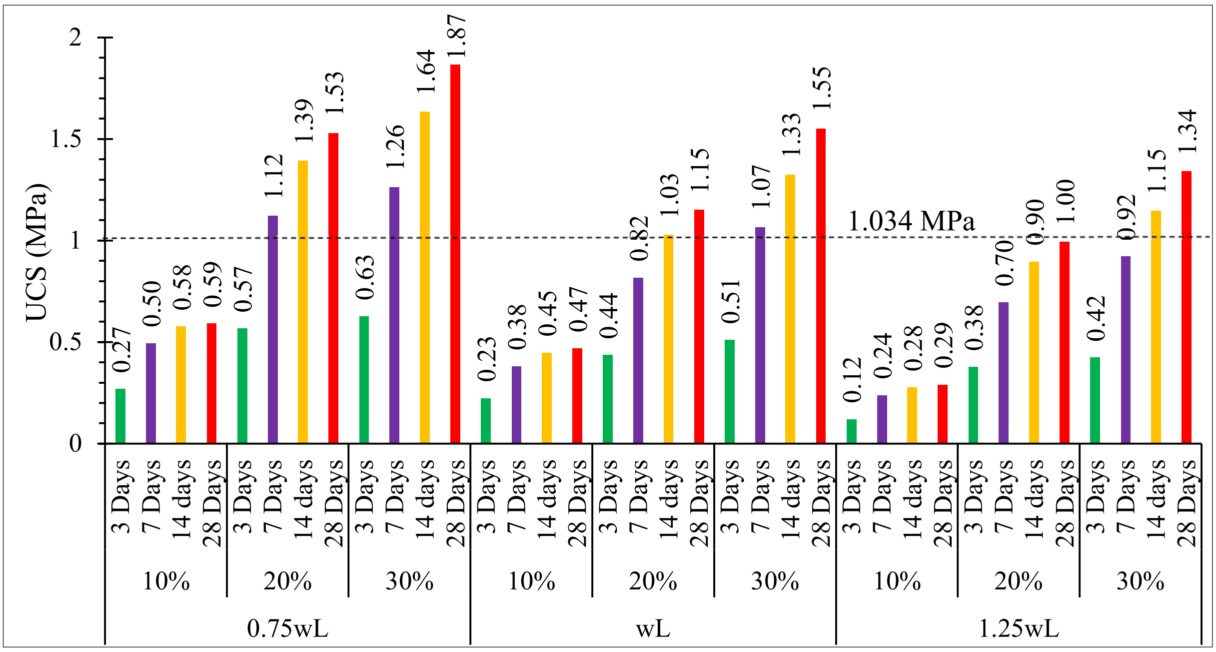



**Fig. 4.9 UCS of soil–cement specimens at varying cement contents and curing periods for different soil water contents**

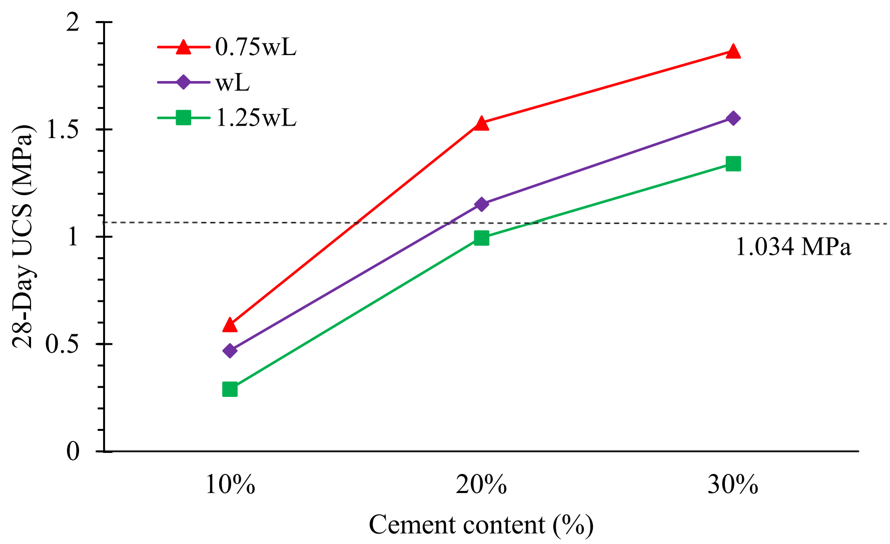



**Fig. 4.10. Variation of 28-day UCS of soil-cement specimens with cement content**

#### 4.2.3.2. Effect of soil water content

The effect of the initial soil water content on the UCS of soil-cement specimens is investigated at $0.75w_L$, $w_L$ and $1.25w_L$. Figure 4.11 shows the variation of 28-day UCS of soil-cement specimens with initial soil water content. From Figures 4.9 and 4.11, it can be noted that with an increase in initial soil water content from $0.75w_L$ to $w_L$, there is an average decrease of about 21% in the UCS of soil-cement specimens. With a further increase in initial soil water content from $w_L$ to $1.25w_L$, there is an average decrease of about 22% in the UCS of the specimens. A similar trend in the reduction of UCS is observed for all curing periods (3 to 28 days) and cement contents (10 to 30%) under study. This reduction in UCS might be because of the excessive availability of water than that required for the cement content, and the excess water content may have filled the pore spaces in the soil structure thus significantly increasing the total water/cement ($w$/C) ratio in the stabilized mix leading to strength reduction. This also prevents the efficient bonding of soil particles and thus caused porosity in the soil structure, which resulted in reduced UCS (Phetchuay et al., 2016; Nath and Sarker, 2017). Furthermore, it is observed from UCS results of soil-geopolymer and soil-cement mixes that at 10% binder content, none of the mixes achieved the target UCS requirement for all the variables in the study. In field, as soil water contents could be much higher, 10% binder content could not be considered as reliable for stabilization of high water content soft clays by deep mixing.

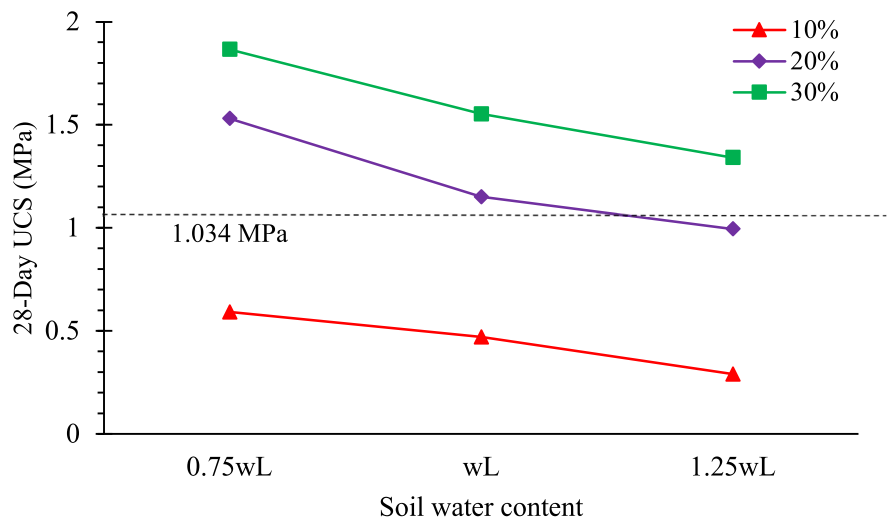



**Fig. 4.11. Variation of 28-day UCS of soil-cement specimens with soil water content**

### 4.2.4. Correlation between UCS and stiffness ($E_{50}$)

For all the soil-geopolymer and soil-cement mixes, the modulus of elasticity or secant modulus ($E_{50}$), defined as the ratio of stress and strain at 50% of the peak stress, is computed to study the variation in the stiffness of the stabilized mixes. The $E_{50}$ of the soft clay mixes is substantially enhanced by stabilizing them with geopolymer and cement, with trends similar to those observed for their respective UCS values. As illustrated in Figure 4.12, an analysis of the UCS and $E_{50}$ values revealed noteworthy correlations between them, as $E_{50}$ = 164UCS and $E_{50}$ = 187UCS for soil-geopolymer and soil-cement mixes respectively. As per the findings from previous studies on cement stabilized soils, $E_{50}$/UCS values range between 30 – 150 (Jamsawang et al., 2011; Shen et al., 2013a; Shen et al., 2013b; Bushra and Robinson, 2013; Du et al., 2014). Though, the correlations from Figure 4.12 indicate that the soil-geopolymer mixes exhibited less brittleness in comparison to the soil-cement mixes, $E_{50}$/UCS values from both the correlations are found to be beyond the range mentioned in findings from previous studies. This increase in stiffness might be due to the higher binder contents and higher concentrations of alkali used in the study. However, in preliminary design or numerical analysis of stabilized clays for DSM, these correlations help to assess stiffness of the material from its respective UCS and can be advantageous to both scientists and engineers.

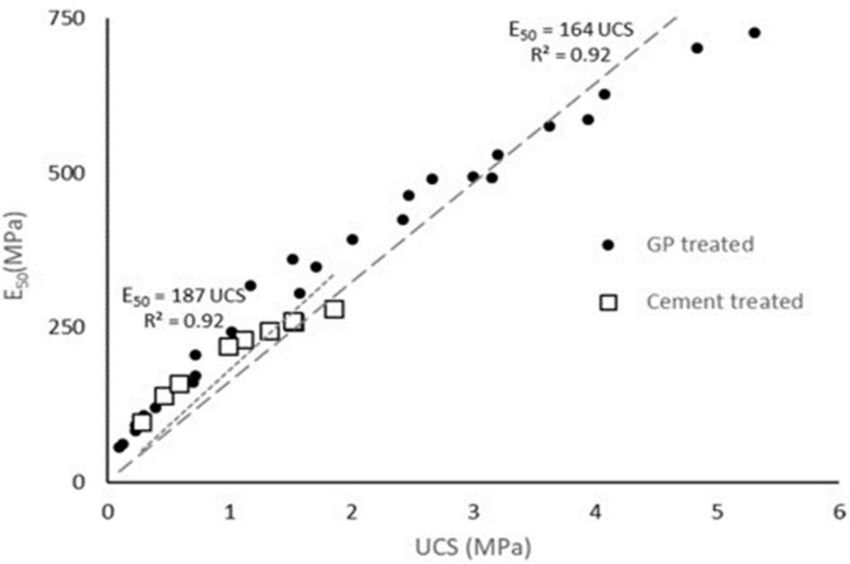



**Fig. 4.12. Correlation between stiffness ($E_{50}$) and UCS of soil-geopolymer and soil-cement mixes**

## 4.2.5. Flexural strength of soil–geopolymer

Four-point flexural strength tests are conducted on soil-geopolymer rectangular beam specimens made with 20% and 30% binder content, 0.75 and 1.0 A/B ratios, 10M NaOH molarity at $0.75w_L$–$1.25w_L$ soil water contents to study their flexural strength after 28 days of curing. Figure 4.13 shows the failure of the flexural strength beam specimen under loading. The two loading points divide the beam span between the supports into three equal parts. As the failure plane occurred within the central third part of the span length, the following formula (Eq. 3.1 from Chapter-3) is used to calculate the flexural strength or modulus of rupture (R) of the beam specimens.

$$R = \frac{PL}{bd^2}$$

**Fig. 4.13. Failure of flexural strength beam specimen**

The flexural strength values of the soil-geopolymer mix specimens followed a trend similar to that of the UCS values of their respective mixes. The highest flexural strength is reported for the mix with binder content of 30%, A/B ratio of 1.0 and NaOH molarity of 10M at any soil water content in the study. During flexural testing, both compression and tension have contributed to the failure of the beams. However, tensile stresses have a more significant impact on the flexural failure compared to compressive stresses, which were experienced by the lower and upper halves of the beams, respectively (Nath and Sarker, 2017). The variation in flexural strength is further plotted and discussed in the next sections along with fiber reinforced soil-geopolymer mixes.

*Correlation between flexural strength and compressive strength of soil-geopolymer*

The UCS test is a commonly employed and advantageous indicator for establishing correlations that facilitate the estimation of flexural strength values. Figure 4.15 illustrates the relationship that exists between the UCS and flexural strength values of soil-geopolymer mixes. Irrespective of the variables of mixes used, soil-geopolymer mix specimens exhibited exceptional linear correlation with high coefficient of determination as shown in Figure 4.15 and Equation 4.2. The correlations given in the previous studies on cement stabilized soils are also presented below for comparison.

$$FS = 0.175 * UCS + 0.135 \qquad \text{------ Eq. (4.2)}$$

$FS = 0.3261 + 0.1131 * UCS$ (Alaitz model)

$FS = e^{1.05} * UCS^{0.58}$ (Heriberto model)

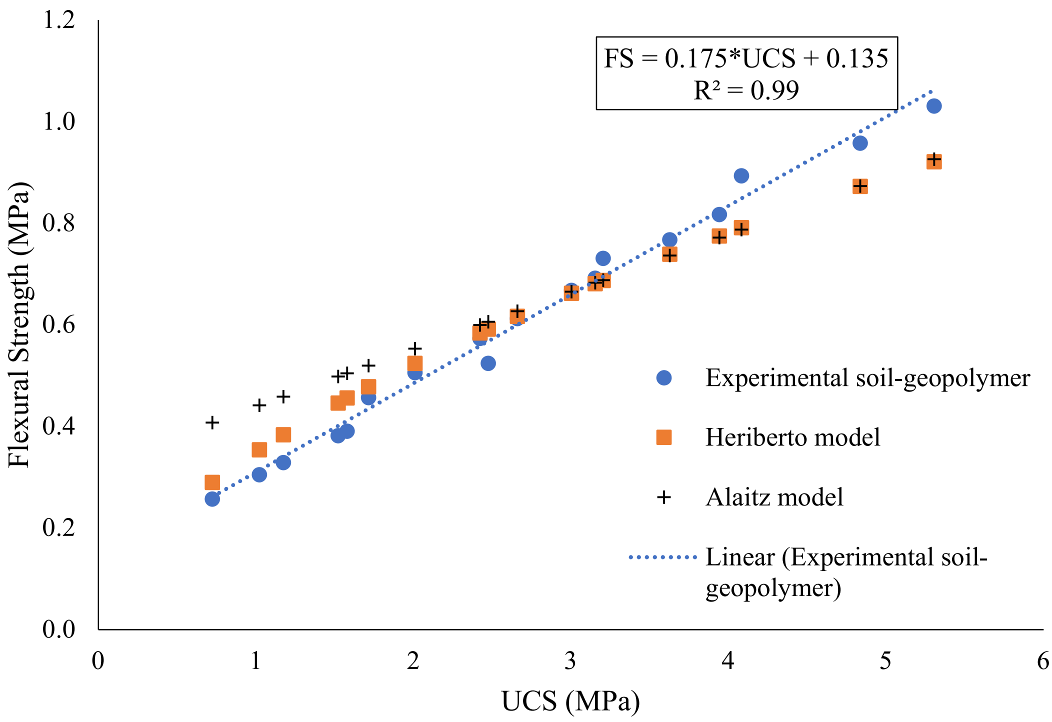



**Fig. 4.14. Correlation between Flexural strength (FS) and UCS of soil-geopolymer mixes**

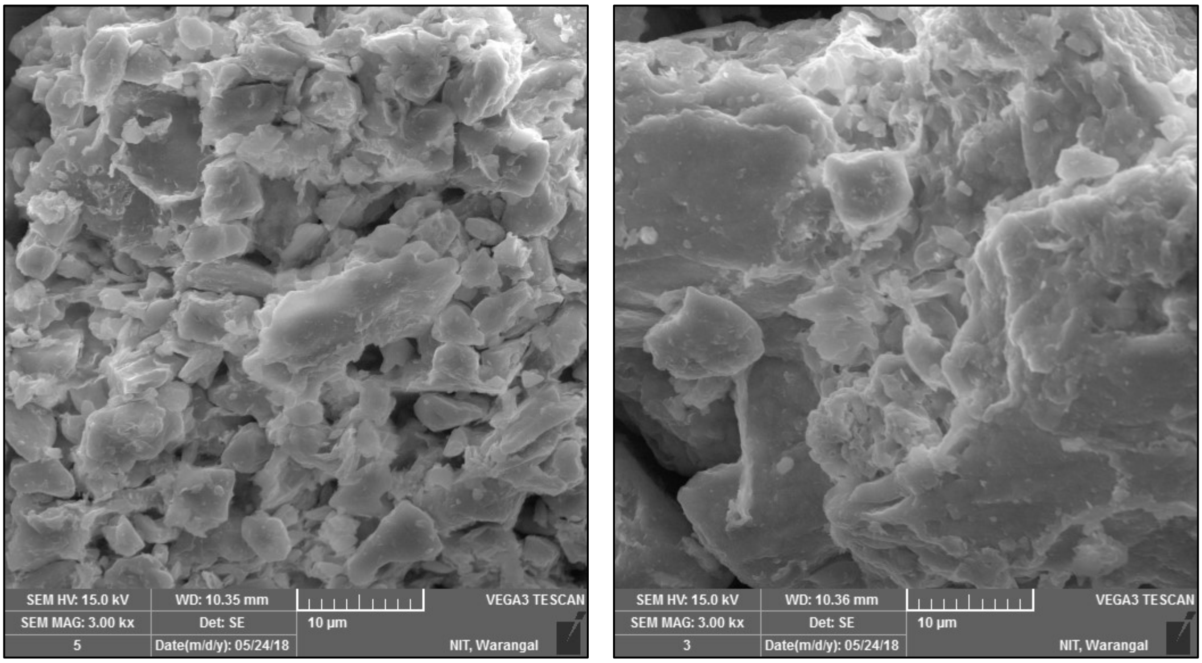
The limitation with Equations 4.2 is that it does not satisfy the fundamental assumption that in the absence of UCS, a mix specimen will also lack flexural strength. Therefore, the minimum UCS value from which Equation 4.2 is valid must be determined. In order to account for all potential cases of UCS, this value must not fall below the lower limit of the target UCS range after a curing period of 28 days. Hence, flexural strength of the soil-geopolymer mixes can be determined using Equation 4.2 if their UCS is greater than 1.034 MPa.

## 4.2.6. Microstructure of soil-geopolymer and soil-cement

Scanning Electron Microscopy (SEM) and Energy Dispersive X-ray Spectroscopy (EDS) tests are performed on selected soil-geopolymer and soil-cement mixes to understand the effect of binder type on the structure of the hardened stabilized mixes and their chemical compositions. Though EDS analysis for the elements in the mixes is less accurate, it is still a useful aid to quantify amorphous elements abundant in geopolymers.

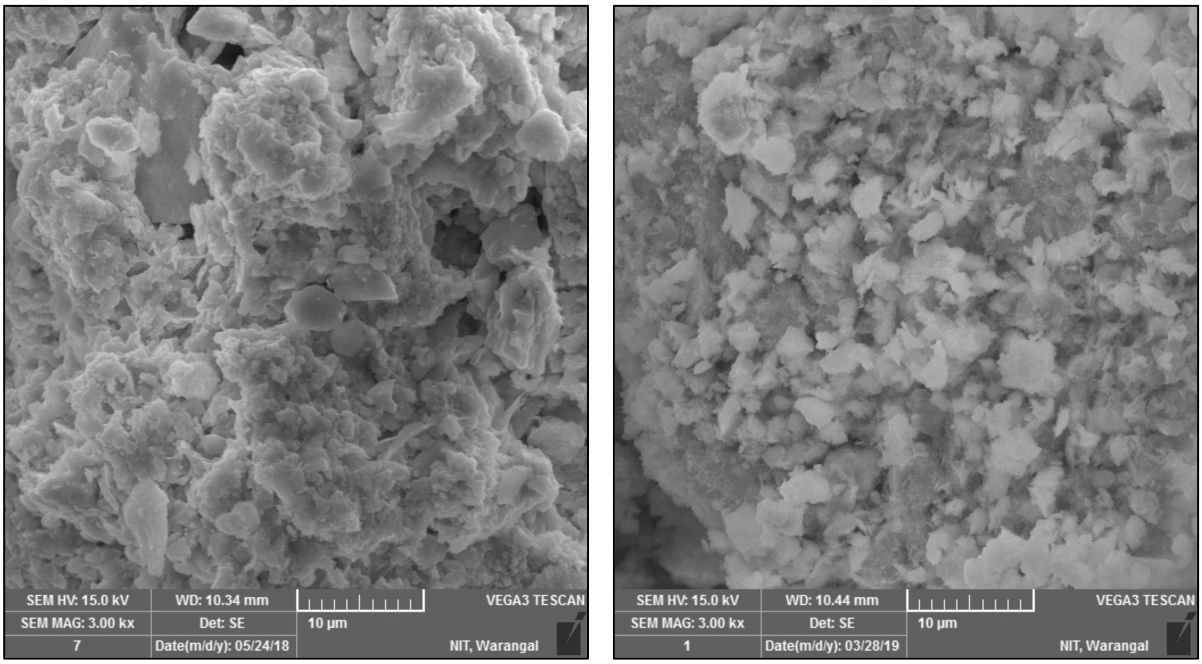
### 4.2.6.1. Scanning Electron Microscopy (SEM)

Figures 4.15 and 4.16 show SEM images of soil-cement (10 and 20% cement content) and soil-geopolymer (20 and 30% binder content with 0.75 A/B ratio) mixes respectively for initial soil water content of $w_L$. In soil-cement mixes, a porous structure of the stabilized matrix (Fig. 4.15a) is observed at 10% cement content thus indicating the inadequacy of cementitious material to bind the soil particles and to form a denser matrix at high water contents. However, at 20% cement content, CSH gels are sufficiently formed to bind all the soil particles together thus forming a denser matrix (Fig. 4.15b) as compared to the one at 10% cement content but is not as dense as soil-geopolymer mix. Figure 4.16a shows that strong bonds are formed between GGBS particles and clay particles in the form of geopolymeric gels thus generating a uniform denser matrix at 20% binder content. This is due to the formation of CSH and NASH gels from GGBS in the presence of NaOH. The combined effect of pozzolanic and geopolymeric mechanisms has an effective impact on the strength enhancement of GGBS based geopolymer stabilized clays. The soil-geopolymer mix at 30% binder content also exhibited denser structure resulting from strong geopolymeric bonds, however, the unreacted or partially reacted GGBS particles (as seen in Fig. 4.16b) present in the soil-geopolymer mix at 30% binder content may cause discontinuity in the gel formation and non-uniformity in the bond formation throughout the mix. Also, it might cause weaker gels and flocculation of particles resulting in internal forces. This might be the reason for the reduction in strength enhancement at 30% binder content for soil-geopolymer mixes. However, it requires further detailed study to affirm this point.




(a) (b)

**Fig. 4.15. SEM images of soil-cement mixes at (a) 10% cement (b) 20% cement for soil water content of $w_L$**

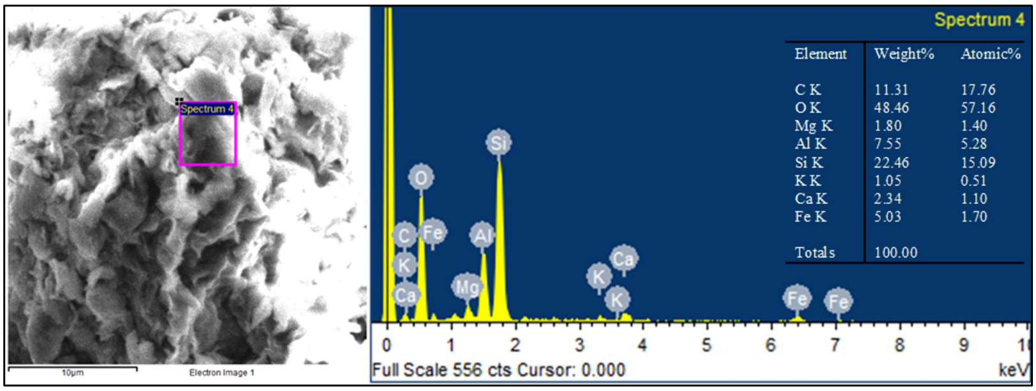



(a) (b)

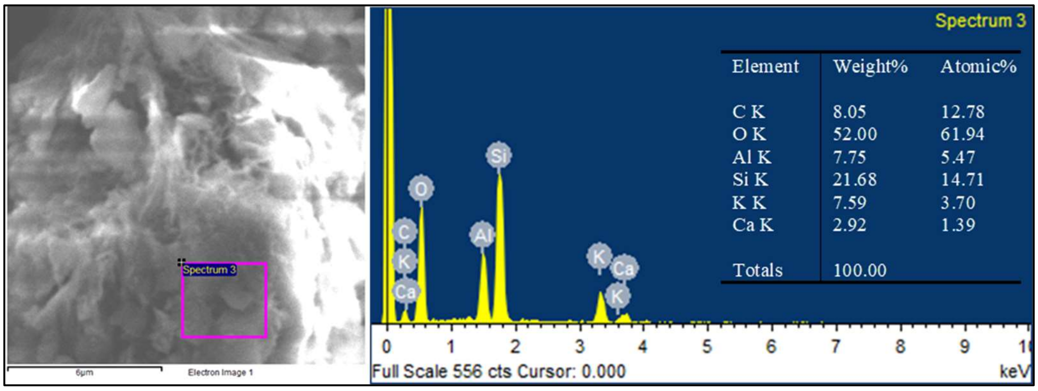
**Fig. 4.16. SEM images of soil-geopolymer mixes at (a) 20% GGBS (b) 30% GGBS for A/B ratio of 0.75 and soil water content of $w_L$**

#### 4.2.6.2. Energy Dispersive X-ray Spectroscopy (EDS)

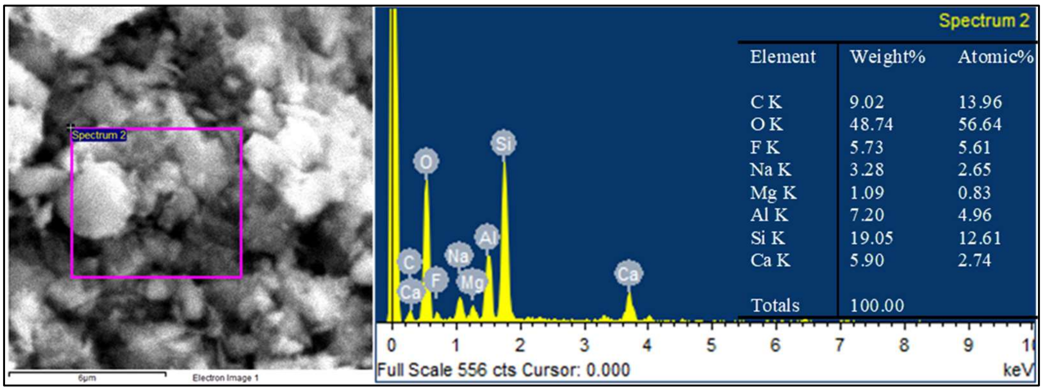
The EDS test results for soil-cement (10% and 20% cement) and soil-geopolymer mixes (20% and 30% GGBS for soil water content of $w_L$ are presented in Figure 4.17. It is observed that the Ca/Si ratios are 0.104 and 0.135, and Si/Al ratios are 2.975 and 2.797 for soil-cement mixes with 10% and 20% cement content respectively. It is also observed that the Ca/Si ratios are 0.310 and 0.400, Si/Al ratios are 2.646 and 2.263, and Na/Al ratios are 0.456 and 0.468 for soil-geopolymer mixes with 20% and 30% binder content respectively at A/B ratio of 0.75. The higher Ca/Si ratios are developed in soil-geopolymer mixes than in soil-cement mixes. This explains the significantly higher strength of geopolymer mixes with GGBS. Further, with increase in the binder content from 20% to 30% in soil-geopolymer mixes, although the Si/Al ratio slightly decreases, due to which the strength is expected to decrease, the combined impact of Ca/Si, Si/Al and Na/Al ratios has caused increase in the strength of the mixes (Cristelo et al., 2013; Singhi et al., 2016).



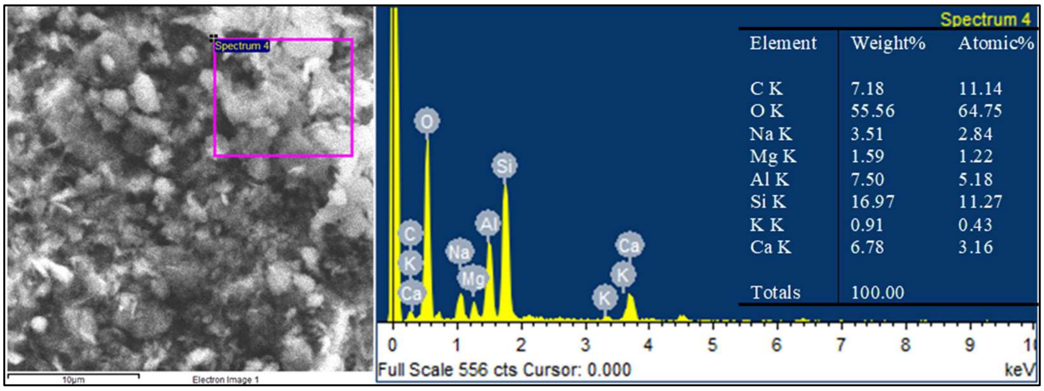

(a)




(b)




(c)




(d)

**Fig. 4.17. SEM images of soil-cement and soil-geopolymer mixes at (a) 10% cement (b) 20% cement (c) 20% GGBS (d) 30% GGBS for soil water content of $w_L$**

## 4.2.7. Cost comparison of soil-geopolymer and soil-cement

The cost incurred to stabilize 1 $m^3$ of soft clay with geopolymer and cement in various mix proportions is calculated according to the prevailing market rates by considering the quantities of materials required for the respective mixes. During calculation, Rs. 52/- for NaOH pellets, Rs. 2.50/- for GGBS and Rs. 9/- for Cement are considered as per market rates per kg. For example, to form a stabilized DSM column, the calculated cost for GP-V-10M-A0.75-B20 mix and C-V-B30 mix is Rs. 3,547/- and Rs. 3,510/- respectively per $m^3$ of the column. However, at almost same cost, the above soil-geopolymer mix gives better UCS (UCS = 2.5 MPa) than the soil-cement mix (UCS = 1.55 MPa). Figure 4.18 represents the calculated Cost (Rs.) and UCS of the soil-geopolymer and soil-cement mixes which helps in selecting a suitable mix that could impart the required UCS at effective cost. To understand this further and to select the binder which is cost effective while imparting the required target UCS for DSM, Cost / UCS ratio is calculated for all the soil-geopolymer and soil-cement mixes at soil water content of $w_L$ and the values are presented in Figure 4.19. In this figure, it is observed that all the soil-geopolymer mixes with binder content of 20 – 30% and A/B ratio of 0.75 – 1.0 have presented lower Cost / UCS values than the soil-cement mixes. It indicates that, to form a DSM column of a particular UCS, GGBS+NaOH geopolymer with aforementioned alkali and binder contents is cost effective than Cement as binder.

From Figure 4.19, it is also observed that all the mixes with binder content of 10% and A/B ratio of 0.5 have presented higher Cost / UCS values because of their lower UCS. This means that these mixes incur considerable costs for stabilization imparting lower UCS to the DSM column (Figure 4.18). Considering the findings from this sub-section and the previous sub-sections, the soil-geopolymer mixes with NaOH molarity up to 10M, binder content >10% and A/B ratio > 0.5 are efficient and economical, and thus considered for further phases of study.

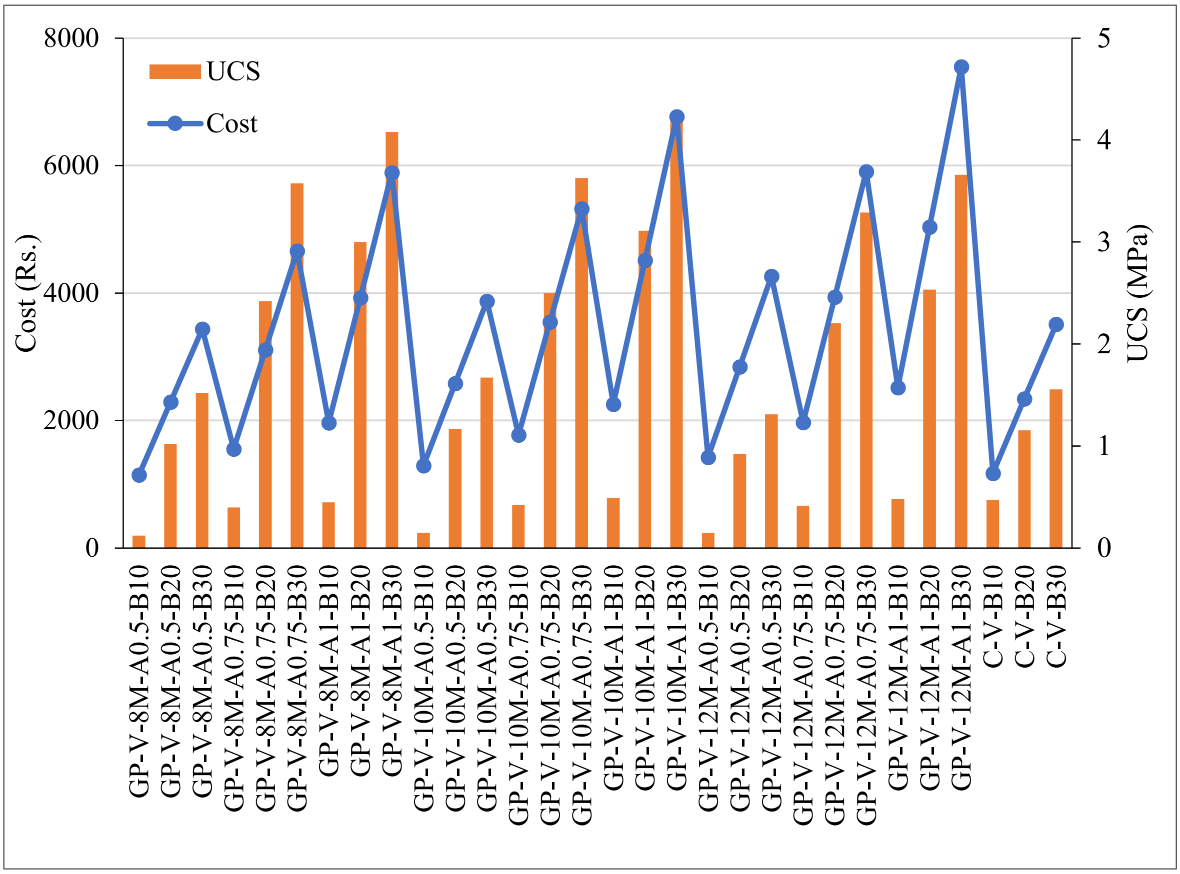



**Fig. 4.18. Cost and UCS of soil-geopolymer and soil-cement mixes at $w_L$**

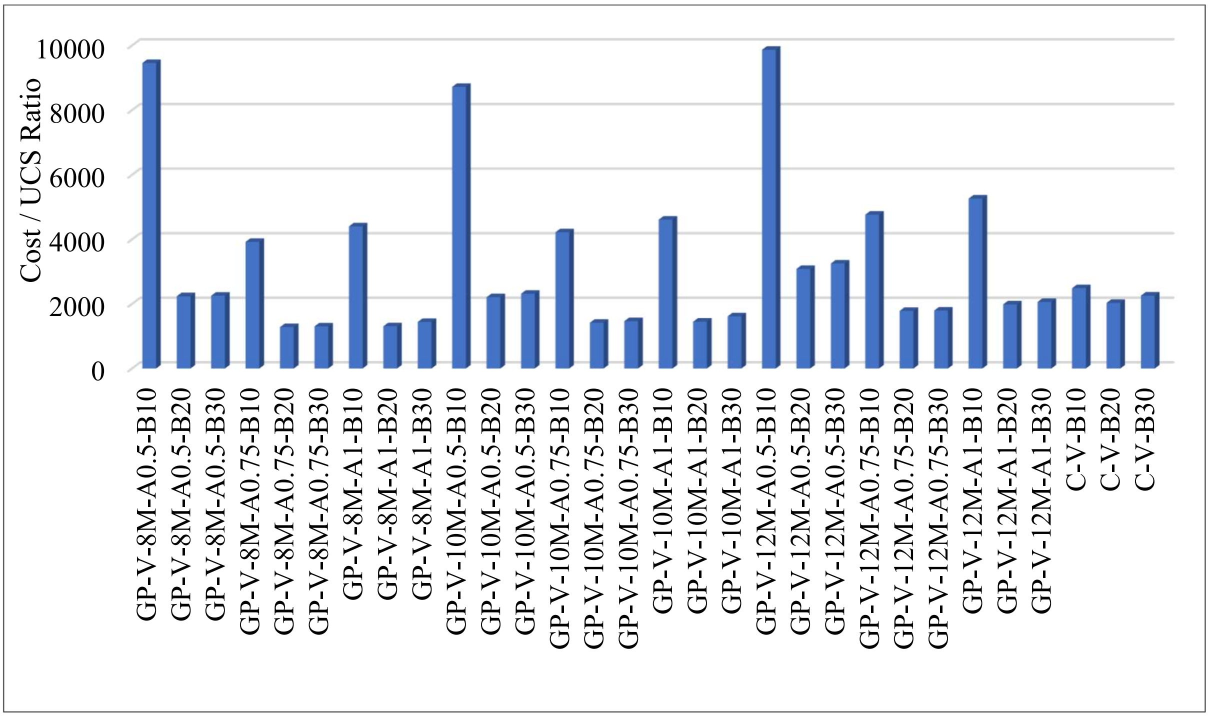



**Fig. 4.19. Cost/UCS ratio of soil-geopolymer and soil-cement mixes at $w_L$**

## 4.3. Strength and stress–strain behaviour of fiber reinforced geopolymer stabilized soft clay

Based on the results obtained from the previous section, it can be noted that soil-geopolymer mixes with A/B ratio of 0.75 and 1.0, and a binder content of 20% and 30% satisfied the target UCS requirement for DSM applications. However, it is observed that the soil-geopolymer specimens displayed brittle failure. Thus, to improve their ductility, Polypropylene (PP) fibers of 12 mm length are used as reinforcement in 0.25 to 1.0% proportions to understand the effect of fiber inclusion on the strength and stiffness of soil-geopolymer mix specimens in Phase-II study. For convenient reading, the soil-geopolymer specimens with and without PP fiber reinforcement are mentioned as FGP and GP specimens respectively in this section.

### 4.3.1. Variables of Phase-II study and mixes used

The effect of PP fiber inclusion on the strength and stiffness of fiber reinforced soil-geopolymer (FGP) specimens is assessed by performing UCS and flexural strength tests. The preparation of samples for these tests and the testing methodology adopted is as discussed in Chapter-3 for soil moisture contents of $0.75w_L$, $w_L$ and $1.25w_L$ ($w_L$ is liquid limit of the soil) indicating soft (S), very soft (V) and liquid (L) consistencies of the soil. As per the observations from the previous section, only selected mixes were used to evaluate the effect of PP fiber inclusion on the strength and stiffness of soil-geopolymer specimens. For each soil water content, the mixes with binder contents of 20% and 30%, A/B ratio of 0.75 and 1.0, NaOH molarity of 10M and PP fiber content or dosage of 0, 0.25, 0.5, 0.75, and 1.0 % by dry weight of soil are considered as given in Table 4.4.

**Table 4.4. Variables of the Phase-II study**

| Parameters | Geopolymer |
|---|---:|
| Materials | GGBS |
| Alkali or activator | NaOH |
| Molarity of Alkali | 10M |
| Binder content (%) | 20, 30 |
| A/B ratio | 0.75, 1.0 |
| Soil water content (%) | $0.75w_L$, $w_L$, $1.25w_L$ |
| Fiber Dosage (%) | 0, 0.25, 0.5, 0.75, 1.0 |
| Tests conducted | UCS, Flexural strength |

Thus, a total of 60 soil-geopolymer mixes with and without PP fiber reinforcement (12 GP mixes and 48 FGP mixes) are prepared. The mix designations for all the GP and FGP mixes used in this phase of study are presented in Table 4.5. The results of the Phase-II experimental investigation are discussed in the subsequent sub-sections.

**Table 4.5. Mix designations for GP and FGP mixes of Phase-II study**

| Mix Designation | Initial soil water content | A/B ratio | Binder Content (%) | Fiber Content (%) |
|---|---|---|---|---|
| GP-S-10M-A0.75-B20 | 0.75 $w_L$ | 0.75 | 20 | 0 |
| FGP-S-10M-A0.75-B20-F0.25 | | | | 0.25 |
| FGP-S-10M-A0.75-B20-F0.5 | | | | 0.50 |
| FGP-S-10M-A0.75-B20-F0.75 | | | | 0.75 |
| FGP-S-10M-A0.75-B20-F1 | | | | 1 |
| GP-S-10M-A0.75-B30 | | | 30 | 0 |
| FGP-S-10M-A0.75-B30-F0.25 | | | | 0.25 |
| FGP-S-10M-A0.75-B30-F0.5 | | | | 0.50 |
| FGP-S-10M-A0.75-B30-F0.75 | | | | 0.75 |
| FGP-S-10M-A0.75-B30-F1 | | | | 1 |
| GP-S-10M-A1-B20 | | 1.0 | 20 | 0 |
| FGP-S-10M-A1-B20-F0.25 | | | | 0.25 |
| FGP-S-10M-A1-B20-F0.5 | | | | 0.50 |
| FGP-S-10M-A1-B20-F0.75 | | | | 0.75 |
| FGP-S-10M-A1-B20-F1 | | | | 1 |
| GP-S-10M-A1-B30 | | | 30 | 0 |
| FGP-S-10M-A1-B30-F0.25 | | | | 0.25 |
| FGP-S-10M-A1-B30-F0.5 | | | | 0.50 |
| FGP-S-10M-A1-B30-F0.75 | | | | 0.75 |
| FGP-S-10M-A1-B30-F1 | | | | 1 |
| GP-V-10M-A0.75-B20 | $w_L$ | 0.75 | 20 | 0 |
| FGP-V-10M-A0.75-B20-F0.25 | | | | 0.25 |
| FGP-V-10M-A0.75-B20-F0.5 | | | | 0.50 |
| FGP-V-10M-A0.75-B20-F0.75 | | | | 0.75 |
| FGP-V-10M-A0.75-B20-F1 | | | | 1 |
| GP-V-10M-A0.75-B30 | | | 30 | 0 |
| FGP-V-10M-A0.75-B30-F0.25 | | | | 0.25 |
| FGP-V-10M-A0.75-B30-F0.5 | | | | 0.50 |
| FGP-V-10M-A0.75-B30-F0.75 | | | | 0.75 |
| FGP-V-10M-A0.75-B30-F1 | | | | 1 |
| GP-V-10M-A1-B20 | | 1.0 | 20 | 0 |

| | | | | |
|---|---|---|---|---|
| FGP-V-10M-A1-B20-F0.25 | | | | 0.25 |
| FGP-V-10M-A1-B20-F0.5 | | | | 0.50 |
| FGP-V-10M-A1-B20-F0.75 | | | | 0.75 |
| FGP-V-10M-A1-B20-F1 | | | | 1 |
| GP-V-10M-A1-B30 | | | | 0 |
| FGP-V-10M-A1-B30-F0.25 | | | | 0.25 |
| FGP-V-10M-A1-B30-F0.5 | | | 30 | 0.50 |
| FGP-V-10M-A1-B30-F0.75 | | | | 0.75 |
| FGP-V-10M-A1-B30-F1 | | | | 1 |
| GP-L-10M-A0.75-B20 | | | | 0 |
| FGP-L-10M-A0.75-B20-F0.25 | | | | 0.25 |
| FGP-L-10M-A0.75-B20-F0.5 | | | 20 | 0.50 |
| FGP-L-10M-A0.75-B20-F0.75 | | | | 0.75 |
| FGP-L-10M-A0.75-B20-F1 | | 0.75 | | 1 |
| GP-L-10M-A0.75-B30 | | | | 0 |
| FGP-L-10M-A0.75-B30-F0.25 | | | | 0.25 |
| FGP-L-10M-A0.75-B30-F0.5 | | | 30 | 0.50 |
| FGP-L-10M-A0.75-B30-F0.75 | | | | 0.75 |
| FGP-L-10M-A0.75-B30-F1 | 1.25 $w_L$ | | | 1 |
| GP-L-10M-A1-B20 | | | | 0 |
| FGP-L-10M-A1-B20-F0.25 | | | | 0.25 |
| FGP-L-10M-A1-B20-F0.5 | | | 20 | 0.50 |
| FGP-L-10M-A1-B20-F0.75 | | | | 0.75 |
| FGP-L-10M-A1-B20-F1 | | 1.0 | | 1 |
| GP-L-10M-A1-B30 | | | | 0 |
| FGP-L-10M-A1-B30-F0.25 | | | | 0.25 |
| FGP-L-10M-A1-B30-F0.5 | | | 30 | 0.50 |
| FGP-L-10M-A1-B30-F0.75 | | | | 0.75 |
| FGP-L-10M-A1-B30-F1 | | | | 1 |

### 4.3.2. Unconfined compressive behavior of fiber reinforced soil–geopolymer

These samples are tested for their unconfined compressive strength (UCS) and their stiffness behavior is also presented based on the respective stress-strain plots.

#### 4.3.2.1. Compressive stress-strain behavior of fiber reinforced soil-geopolymer

The effect of fiber addition on the stress-strain behavior of stabilized soft clay specimens under unconfined compression is examined and presented as shown in Figures 4.20 to 4.22 for soil water contents ranging from $0.75w_L$ to $1.25w_L$. It can be observed that GP specimens exhibited a brittle failure, while the FGP specimens exhibited an evident improvement in the peak strains with increase in the fiber content. For all soil water contents, the peak stresses have increased significantly with an increase in binder content of FGP specimens from 20% to 30%. Also, though the FGP specimens displayed higher peak stresses at 0.5% fiber content, their peak strains and failure strains are higher at 1% fiber content.

***Energy absorption and Toughness Index***

To quantify the effect of fiber addition on the ductility of stabilized soft clay specimens, the energy absorption in compression ($EA_C$) and toughness index ($I_T$) are evaluated. The energy absorption in compression ($EA_C$) is the area under the compressive stress-strain curve, while the toughness index ($I_T$) is the ratio of $EA_C$ of fiber reinforced soil-geopolymer specimen and the $EA_C$ of corresponding unreinforced soil-geopolymer specimen. When compared to unreinforced soil specimens, FGP specimens have exhibited higher Energy absorption values, and this may be attributed to the improved mechanical properties and structural behavior imparted by the presence of fibers. In addition, the fibers prevent the development and propagation of cracks (crack bridging mechanism), allowing the soil to absorb more energy before failure by which the FGP specimens exhibited higher values. The variation of $EA_C$ with PP fiber dosage is presented in Figures 4.23 to 4.25 for soil water contents from $0.75w_L$ to $1.25w_L$. It can be noted that the increment in energy absorption in compression with increase in fiber dosage follows a linear pattern and the maximum increment of EAc values were exhibited by mixes with 1% fiber dosage. At higher fiber dosage, more fibers are distributed within the soil matrix which results in a greater reinforcement effect, enhancing the overall post peak behaviour of the FGP mixes. The variation in toughness index ($I_T$) is presented in Figures 4.26 to 4.28. From these figures it can be deduced that FGP specimens exhibited higher $I_T$ values than unreinforced mixes and this may be due to the energy absorption exhibited by the fibers thereby reducing the severity of deformations and enhancing the soil's ability to absorb stress without undergoing significant damage. This increased energy absorption capacity allows the soil to withstand higher levels of stress without undergoing significant damage, contributing to enhanced toughness. Further it can be noted that there is an increase in toughness index with increase in fiber dosage and this can be due to better bonding between the fibers and soil particles. This improved bonding enhances the transfer of stress between the soil matrix and the fibers, contributing to increased toughness index values.

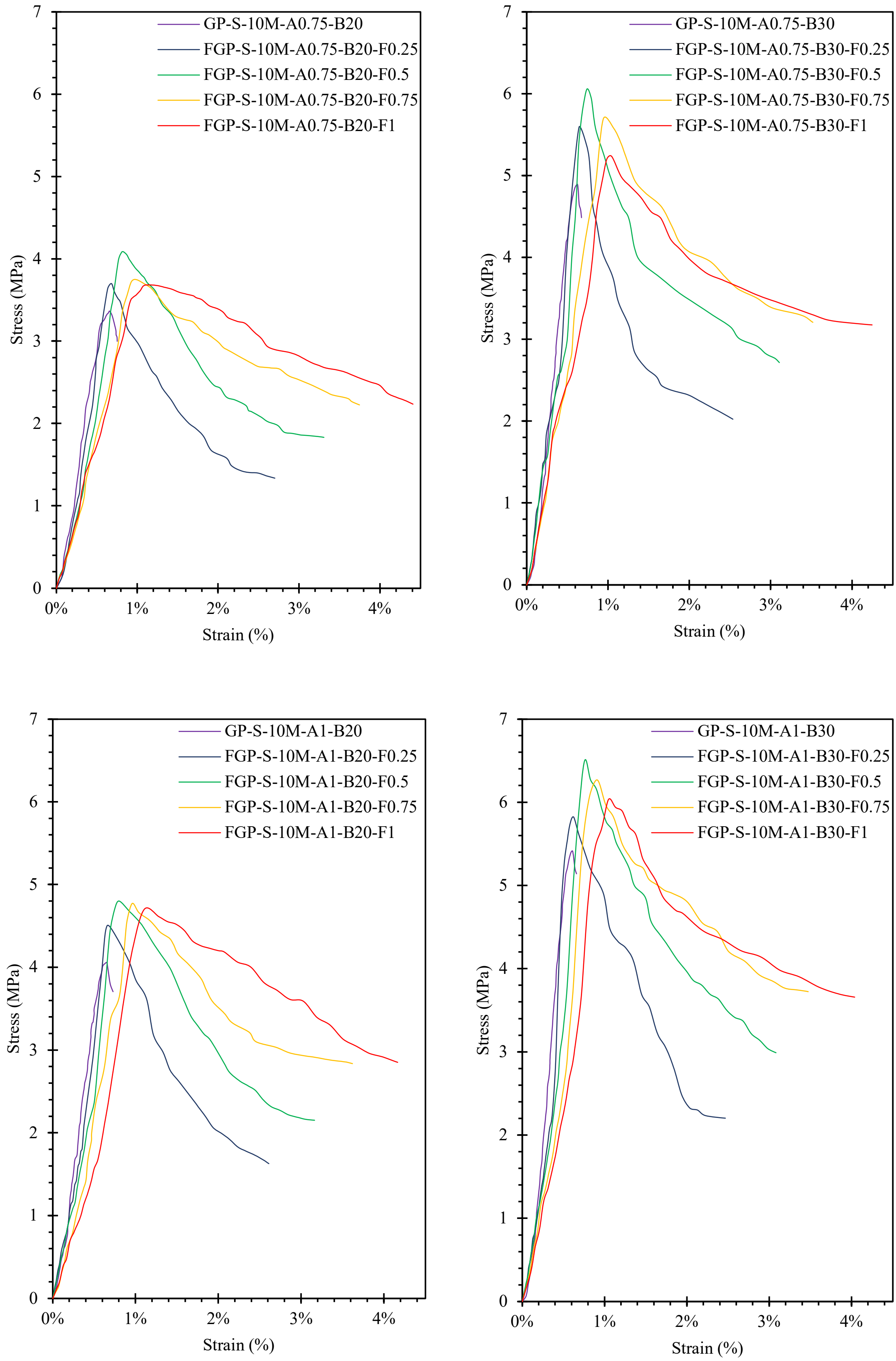



**Fig. 4.20. Compressive stress-strain behavior of GP and FGP specimens for soil water content of 0.75$w_L$**

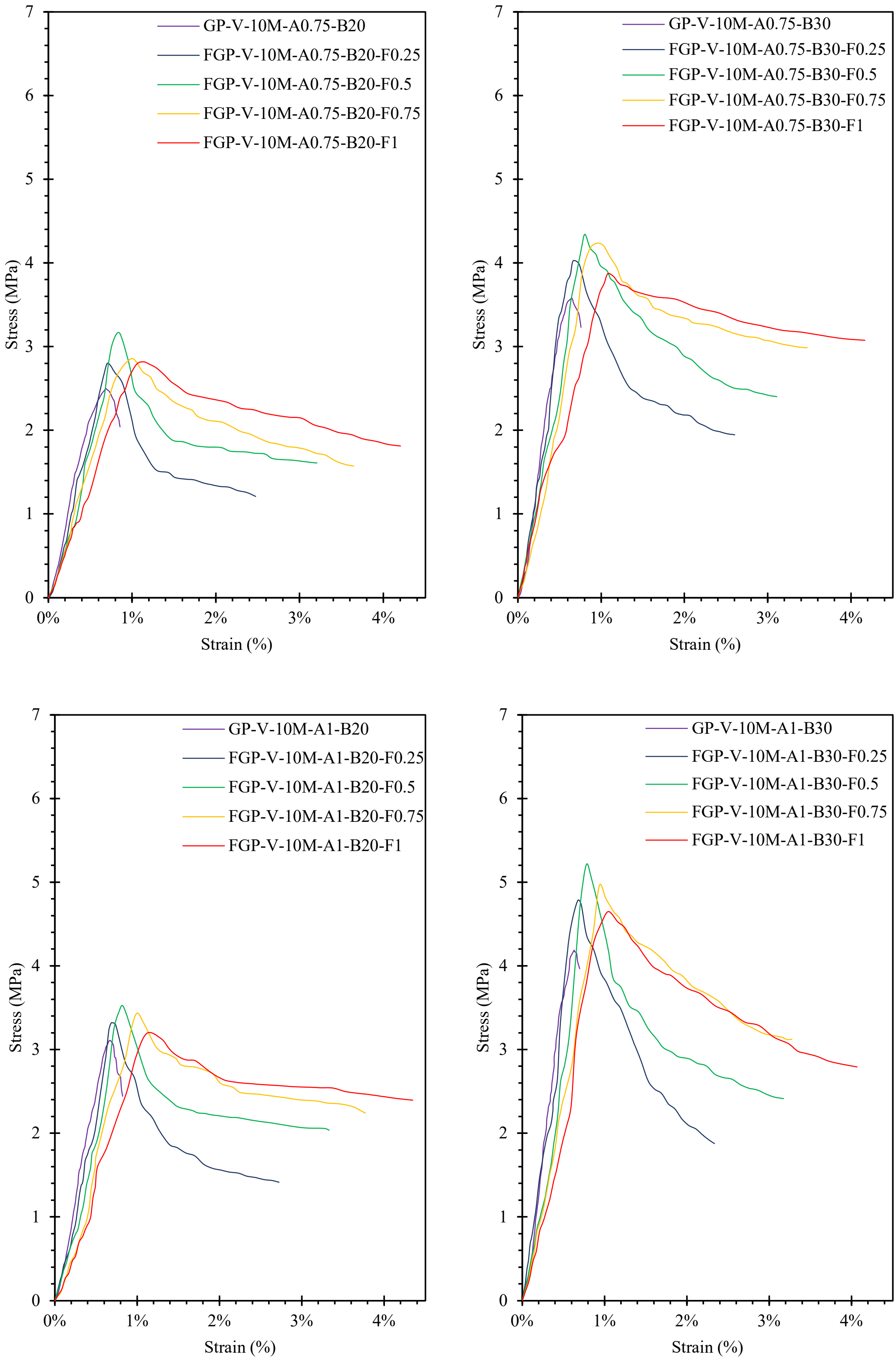



**Fig. 4.21. Compressive stress-strain behavior of GP and FGP specimens for soil water content of $w_L$**

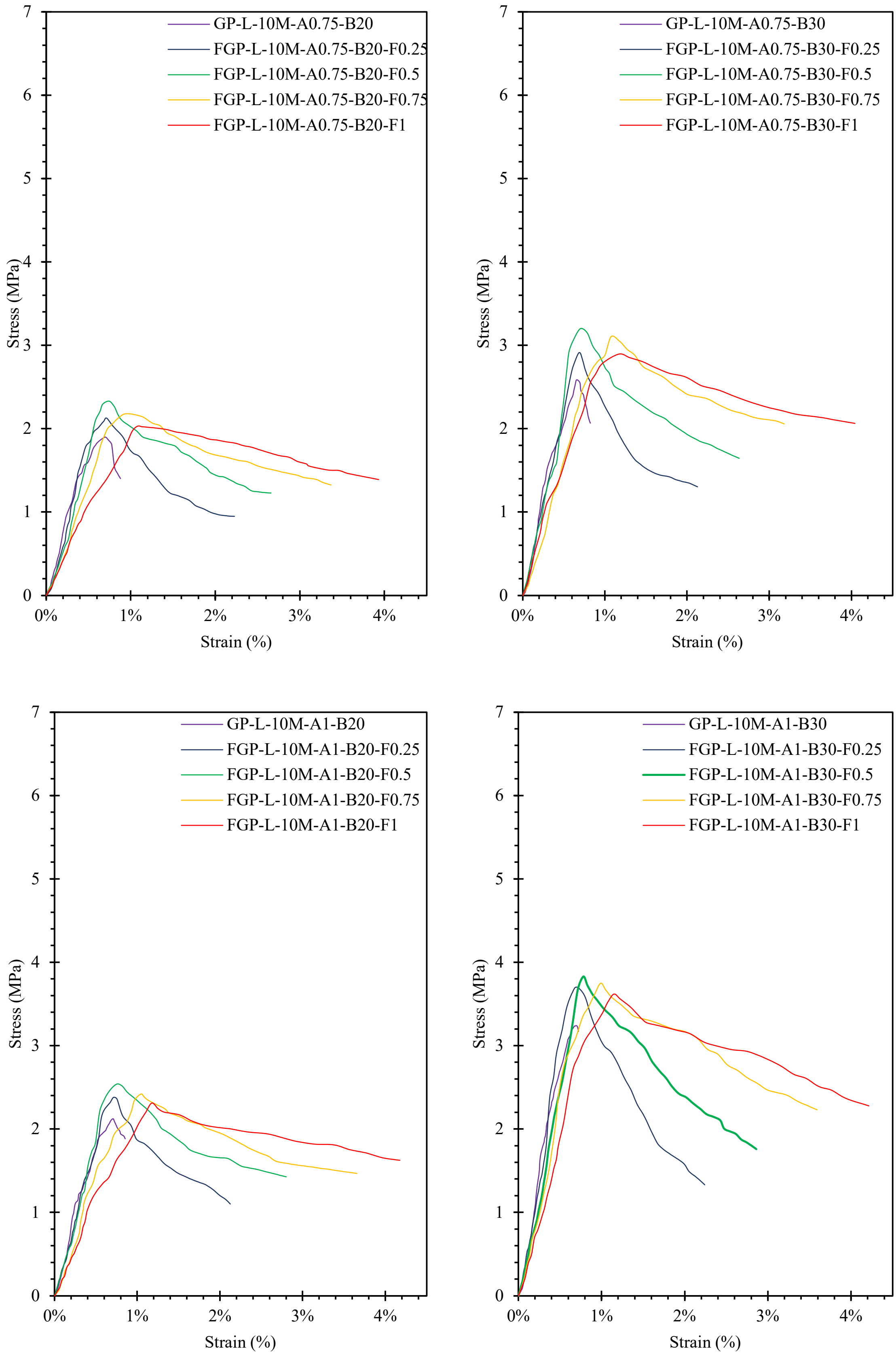



**Fig. 4.22. Compressive stress-strain behavior of GP and FGP specimens for soil water content of 1.25$w_L$**

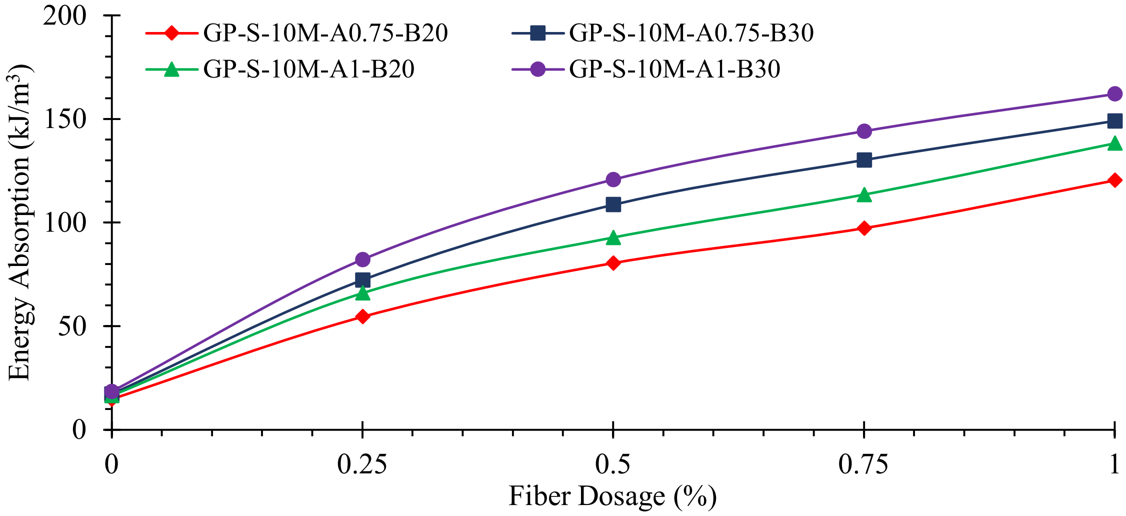



**Fig. 4.23. Energy absorption in compression ($EA_C$) of fiber reinforced soil-geopolymer mixes for soil water content of $0.75w_L$**

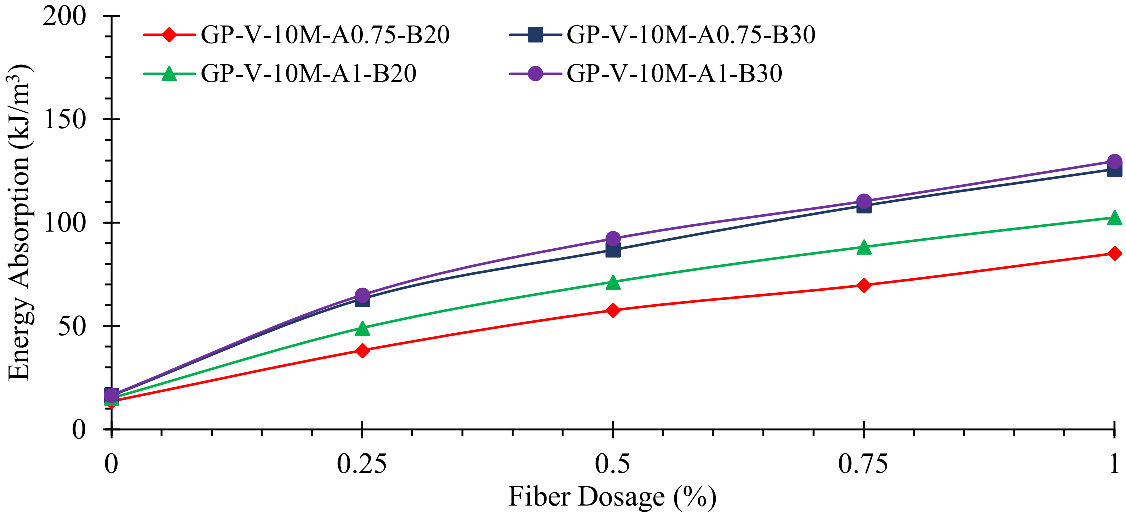



**Fig. 4.24. Energy absorption in compression ($EA_C$) of fiber reinforced soil-geopolymer mixes for soil water content of $w_L$**

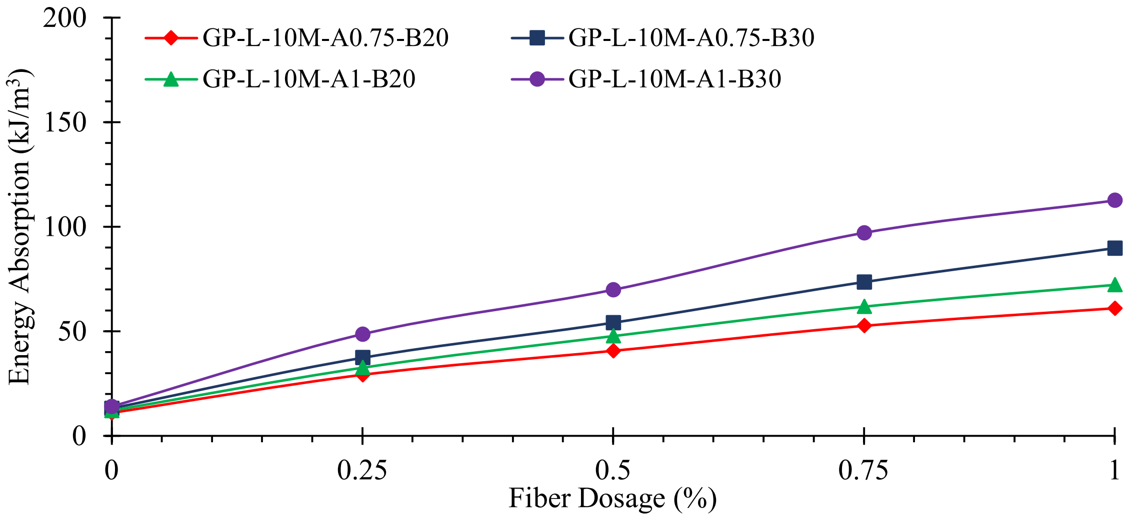



**Fig. 4.25. Energy absorption in compression ($EA_C$) of fiber reinforced soil-geopolymer mixes for soil water content of $1.25w_L$**

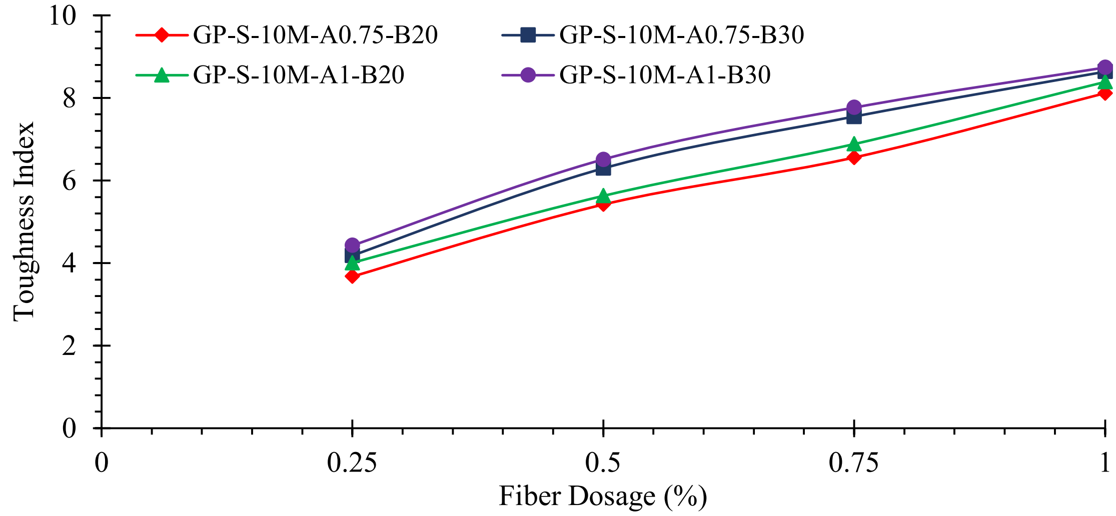



**Fig. 4.26. Toughness index ($I_T$) of fiber reinforced soil-geopolymer mixes for soil water content of 0.75$w_L$**

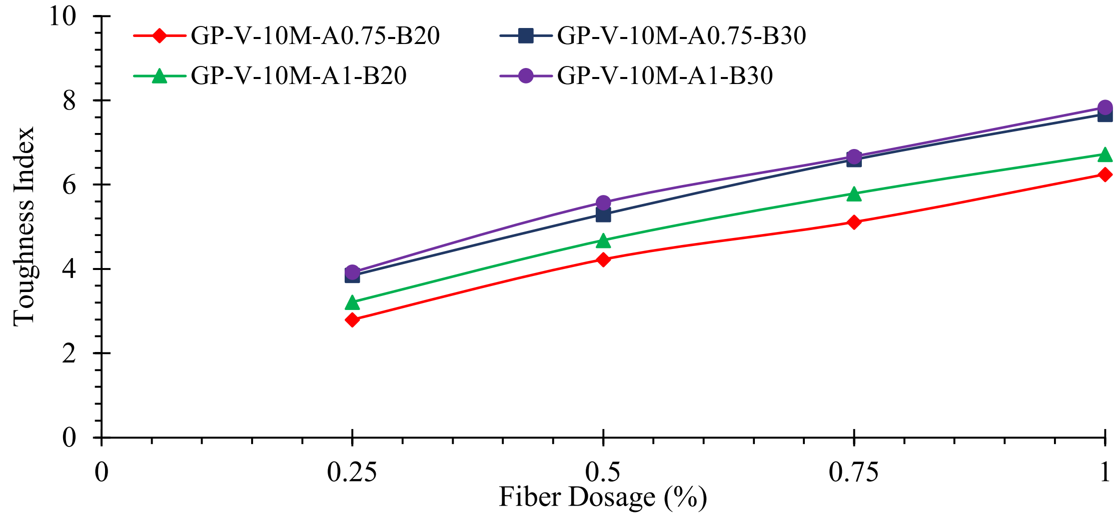



**Fig. 4.27. Toughness index ($I_T$) of fiber reinforced soil-geopolymer mixes for soil water content of $w_L$**

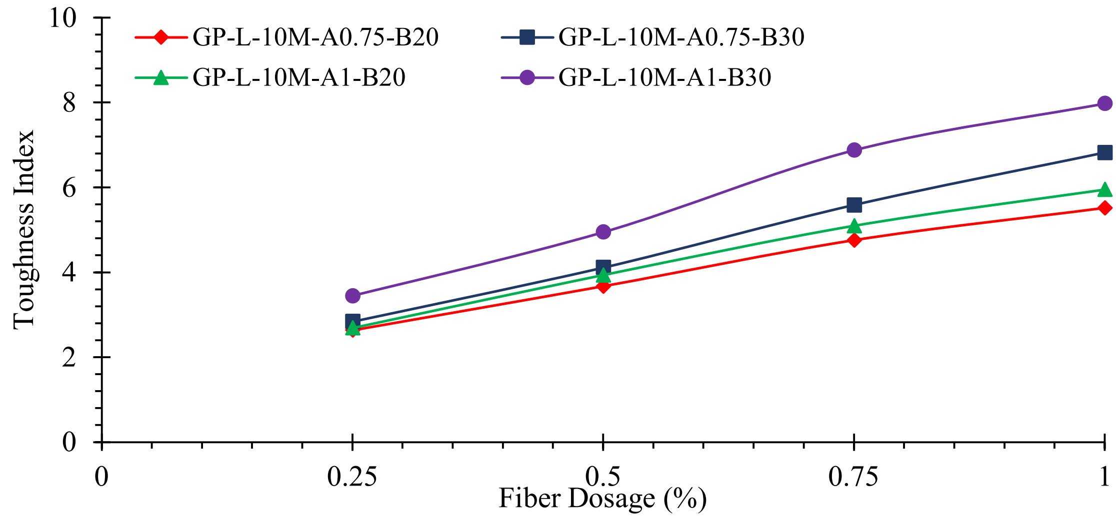



**Fig. 4.28. Toughness index ($I_T$) of fiber reinforced soil-geopolymer mixes for soil water content of 1.25$w_L$**

#### 4.3.2.2. Unconfined compressive strength of fiber reinforced soil-geopolymer

The variation of UCS of all fiber reinforced soil-geopolymer (FGP) mixes at four different curing periods (3, 7, 14, 28 days) with fiber dosage (or fiber content) is plotted and compared with unreinforced soil-geopolymer (GP) mixes as shown in Figures 4.29 to 4.31. With the increase in binder content and A/B ratio, there is an increment in UCS values for FGP mixes when compared with the GP mixes. From Figures 4.29 to 4.31, it is observed that there is an increment in the UCS values of the specimens with an increase in fiber content up to 0.5%; thereafter, there is a slight decrement in UCS values, irrespective of binder content and A/B ratio. This improvement in the UCS up to 0.5% fiber content may be attributed to the homogeneous distribution of fibers in the stabilized soil matrix that arrested crack formation by bridging cracks and enhancing the strength of the composite material. At the micro-level, fibers interact with the soil-geopolymer matrix through mechanical interlocking and chemical bonding that distribute stress more evenly, inhibit crack initiation and propagation, and improve the ductility of the material making the mix specimens more resistant to cracking under various loading conditions. With further increase in fiber dosage beyond 0.5%, all the mixes have shown a slight decrement in the UCS, and this could be attributed to the difficulty in uniform mixing of fibers. This may also be due to clumping of fibers during mixing and casting of specimens thereby losing contact with the nearby stabilized soil matrix which may have caused slippage at those locations under load. The reduction in strength, on the other hand, can be deemed as nominal. In order to understand the general trend in the variation of UCS with fiber dosage, UCS ratio or compressive strength ratio of all the specimens is determined. Compressive strength ratio is the ratio of UCS of FGP mix specimens to UCS of GP mix specimens, and can be written as:

$$Compressive\ Strength\ Ratio = \frac{UCS_f}{UCS_0}$$

Where, $UCS_f$ is the UCS of the specimens at any fiber dosage from 0.25 – 1.0 % and $UCS_0$ is the UCS of GP specimens which do not contain fibers.

The compressive strength ratio values are normalized with $UCS_0$ values and are plotted in Figure 4.32 and a unique relationship is obtained by relating the UCS ratio with fiber dosage, which is given by equation 4.3.

$$Compressive\ Strength\ Ratio \left(\frac{UCS_f}{UCS_0}\right) = -0.56f^2 + 0.66f + 1.00 \qquad \text{----- Eq. 4.3}$$

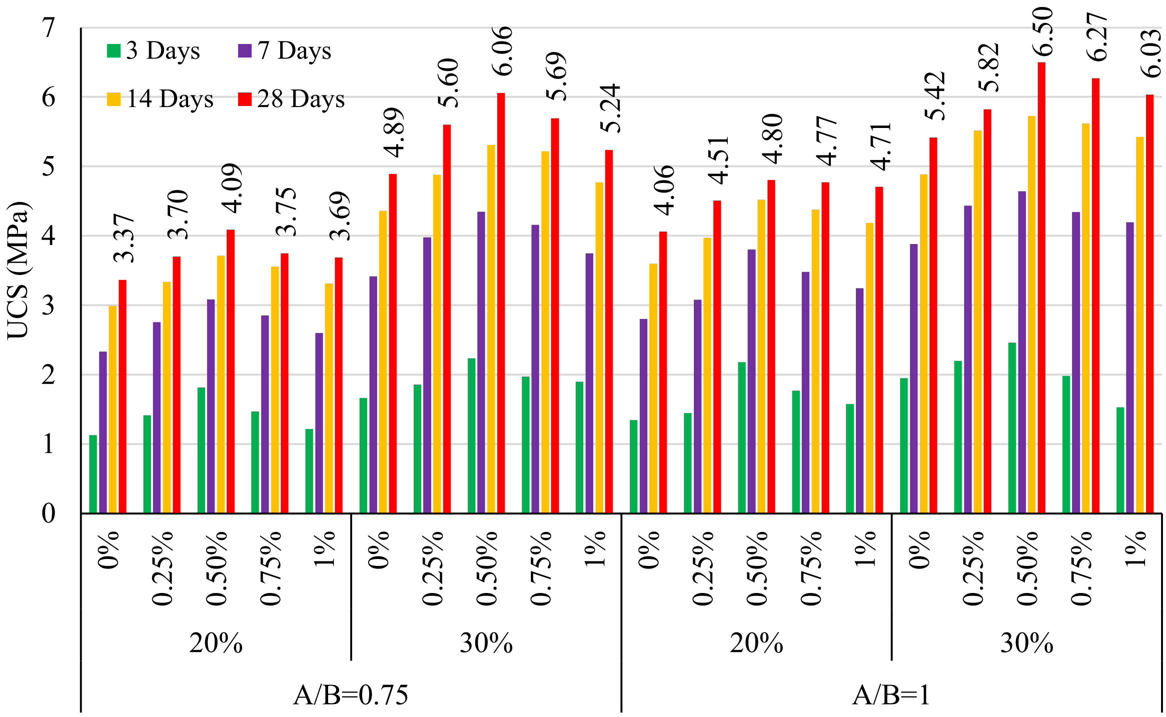



**Fig. 4.29. Variation of UCS with fiber dosage at initial soil water content of 0.75$w_L$**

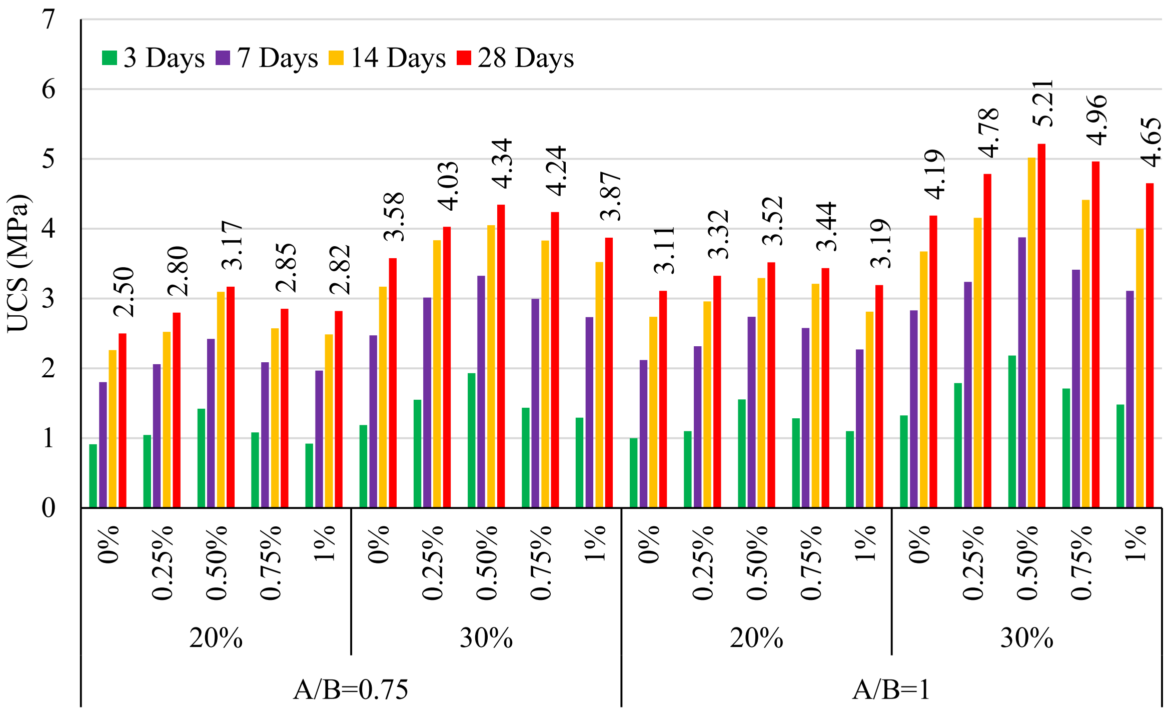



**Fig. 4.30. Variation of UCS with fiber dosage at initial soil water content of $w_L$**

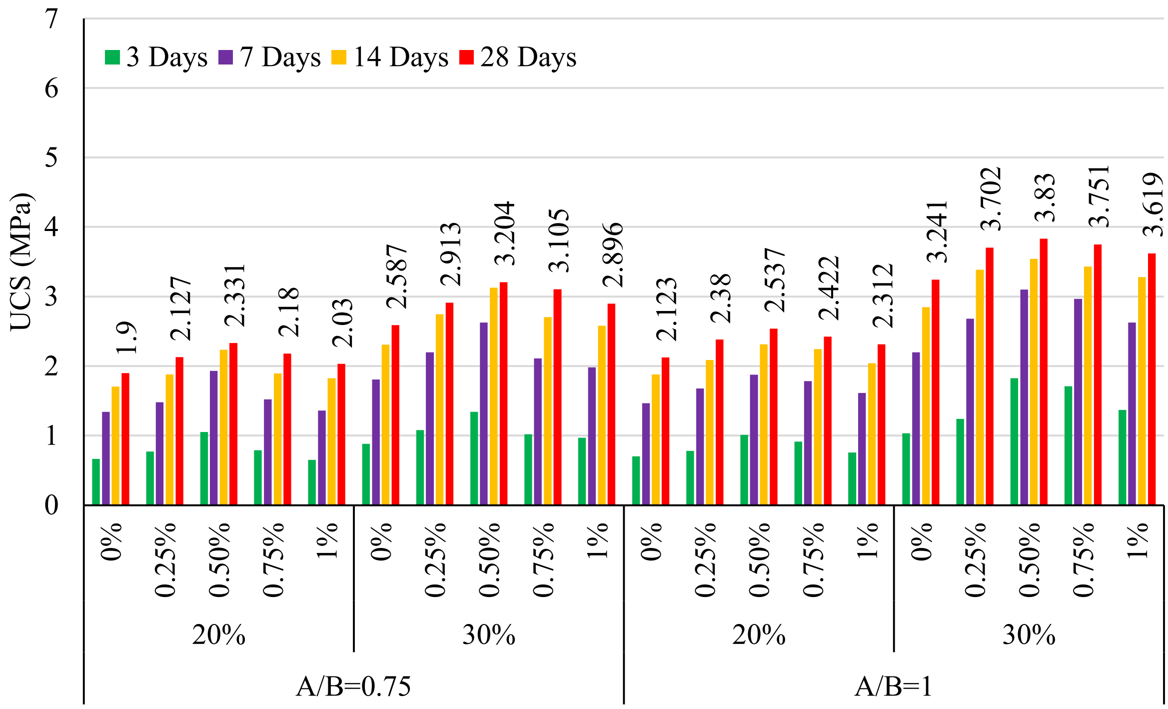



**Fig. 4.31. Variation of UCS with fiber dosage at initial soil water content of 1.25$w_L$**

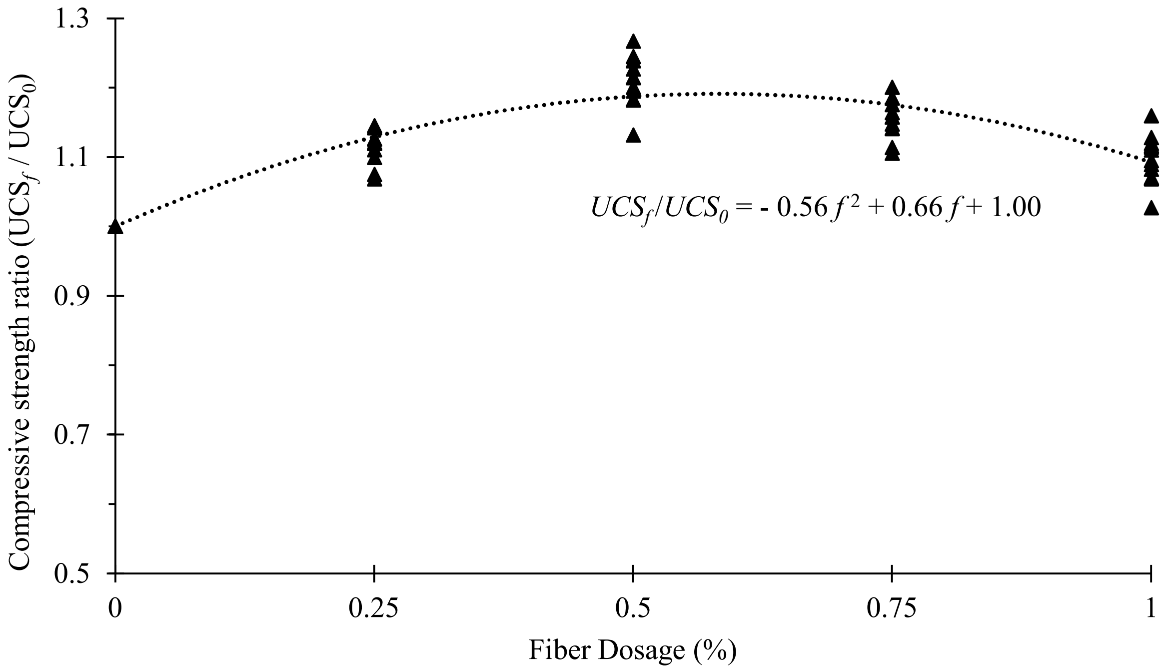



**Fig. 4.32. Compressive strength ratio ($UCS_f/UCS_0$) vs fiber dosage**

Equation 4.3 can be used to approximately assess the UCS of FGP mixes at any fiber content up to 1% if the UCS of the respective unreinforced GP mix is known, thus it can be helpful for engineers and researchers in designs and numerical modelling.

#### 4.3.2.3. Stiffness of fiber reinforced soil-geopolymer

The stiffness of the soil-geopolymer mix specimens with and without PP fibers is evaluated using Secant Modulus ($E_{50}$) or modulus of elasticity which is defined as the ratio of stress to strain at 50% of peak stress and is plotted as shown in Figures 4.33 to 4.35 for GP and FGP mix specimens for soil water contents from $0.75w_L$ to $1.25w_L$. As discussed in the previous section, the $E_{50}$ of the soft clay mixes is substantially enhanced by stabilizing them with geopolymer at increased A/B ratios and binder contents. This increase in the stiffness of the stabilized mix specimens indicates their brittle and sudden failure under compressive load. This can prove to be problematic when these mixes with higher stiffness are used for DSM columns in the field. Thus, in order to reduce the stiffness or brittleness of the mixes thus improving their ductility, not compromising their higher strength under compressive load, PP fibers of lower modulus of elasticity are used as reinforcement as mentioned in Chapter-2. From the stiffness plots of GP and FGP specimens (Figures 4.33 to 4.35), it can be noticed that the $E_{50}$ of the stabilized mixes is substantially reduced with increase in fiber content from 0% to 1% thus making them less brittle, with maximum reduction in $E_{50}$ observed at 1% fiber content (average reduction of 42.5%). Similar trends in variation of $E_{50}$ are observed for all the mixes with any A/B ratio and binder content for all soil water contents in the study. Figure 4.36 shows the failure of GP and FGP mix specimens under compression.

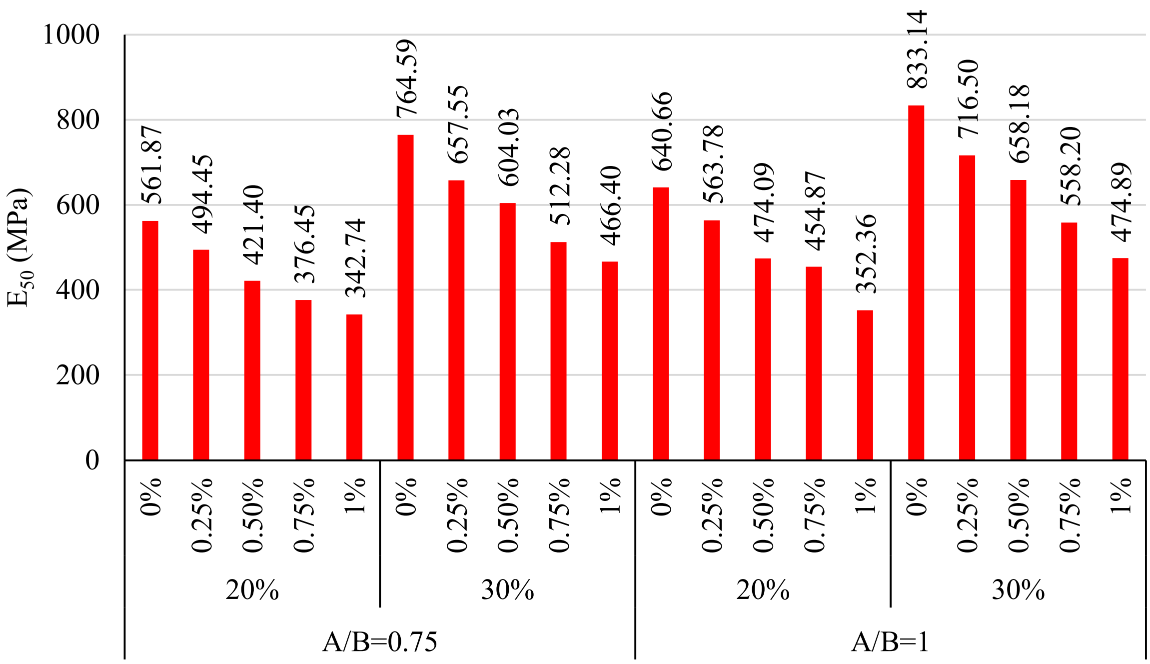



**Fig. 4.33. Variation of Stiffness ($E_{50}$) with fiber content for mixes at $0.75w_L$**

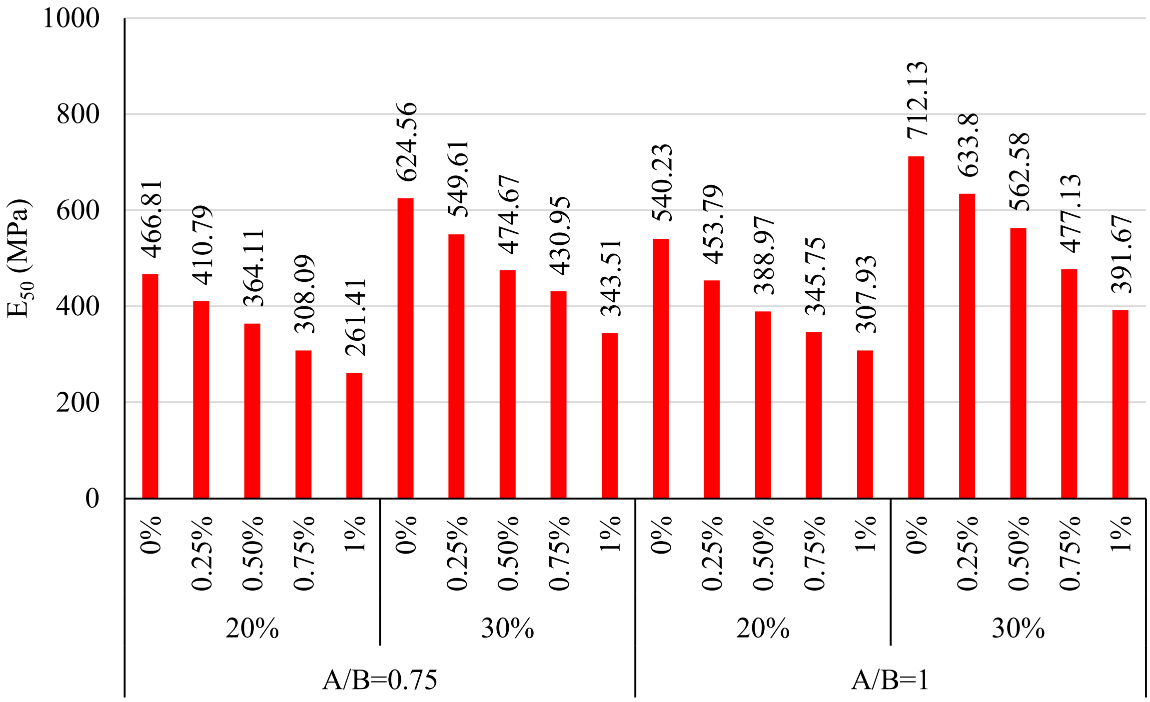



**Fig. 4.34. Variation of Stiffness ($E_{50}$) with fiber content for mixes at $w_L$**

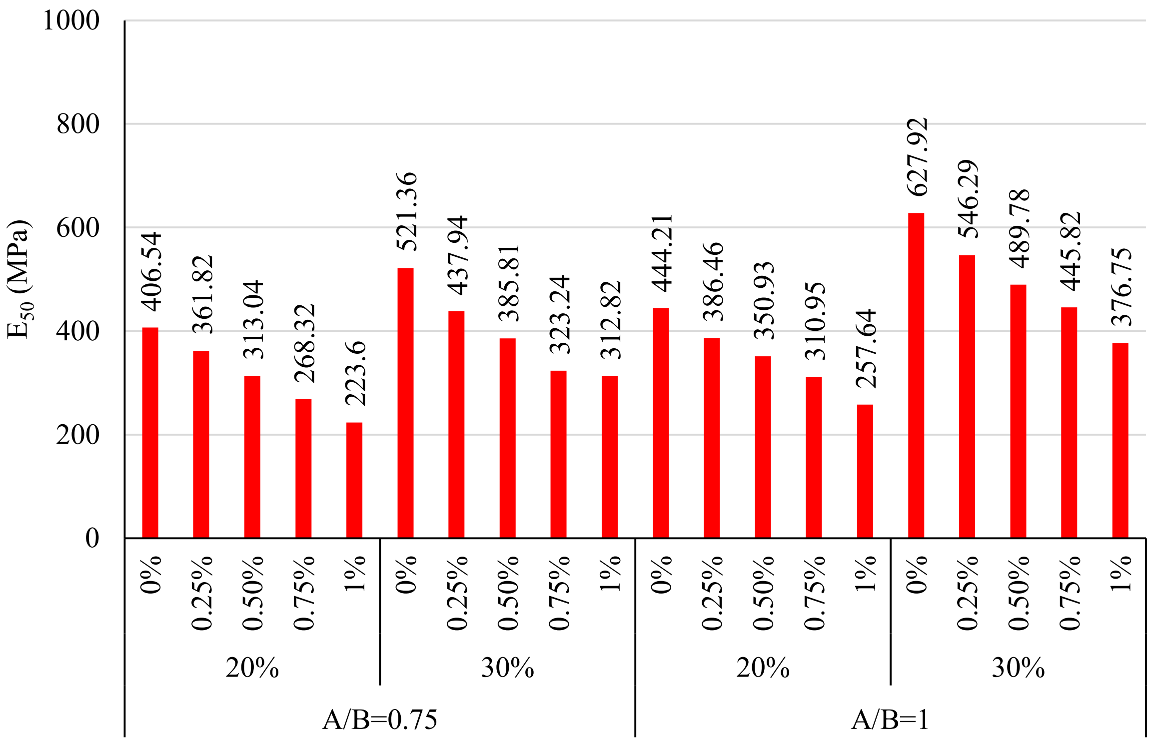



**Fig. 4.35. Variation of Stiffness ($E_{50}$) with fiber content for mixes at $1.25w_L$**

(a)

(b)

**Fig. 4.36. Failure of (a) GP and (b) FGP specimens under compression**

## 4.3.3. Flexural strength of fiber reinforced soil-geopolymer

The flexural strength of the fiber reinforced soil-geopolymer mix specimens is evaluated and plotted as shown in Figures 4.38 to 4.40 by conducting four-point flexural strength tests on FGP rectangular beam specimens made with 20% and 30% binder content, 0.75 and 1.0 A/B ratios, 10M NaOH molarity at $0.75w_L$–$1.25w_L$ soil water contents after 28 days of curing. Figure 4.37 shows the failure of the FGP beam specimen at 1% fiber content under flexural loading. The two loading points divide the beam span between the supports into three equal parts. As the failure plane occurred within the central third part of the span length, the following formula (Eq. 3.1 from Chapter-3) is used to calculate the flexural strength (F) or modulus of rupture (R) of the beam specimens.

$$R = \frac{PL}{bd^2}$$

**Fig. 4.37. Failure of fiber reinforced soil-geopolymer beam specimen under flexure**

The flexural strength values of the soil-geopolymer mix specimens followed a trend similar to that of the UCS values of their respective mixes. During flexural testing, both compression and tension have contributed to the failure of the beams. However, tensile stresses had a more significant impact on the flexural failure compared to compressive stresses, which were experienced by the lower and upper halves of the beams, respectively (Nath and Sarker, 2017). From the flexural strength plots of GP and FGP specimens (Figures 4.38 to 4.40), it can be observed that the flexural strength of the stabilized mixes is increased with increase in fiber content from 0% to 0.5% with average increase of about 136%, after which nominal reduction is observed till 1% fiber content because of the reason mentioned in sub-section 4.3.2.2. The highest flexural strength is noticed for the FGP mix with binder content of 30%, A/B ratio of 1.0, NaOH molarity of 10M and fiber content of 0.5% at any soil water content in the study.

The addition of fibers to the soil-geopolymer specimens has changed the failure mode from brittle to ductile. To understand the general trend in the variation of flexural strength with fiber content, flexural strength ratios are determined for all the mix specimens. Flexural strength ratio is the ratio of flexural strength of FGP mix specimens to flexural strength of GP mix specimens, and can be written as:

$$Flexural\ Strength\ Ratio = \frac{F_f}{F_0}$$

Where, $F_f$ is the flexural strength of the specimens at any fiber dosage from 0.25 – 1.0 % and $F_0$ is the flexural strength of GP specimens which do not contain fibers.

The flexural strength ratio values are normalized with $F_0$ values and are plotted in Figure 4.41 and a unique relationship is established by relating the flexural strength ratio with fiber dosage, which is given in Equation 4.4.

$$Flexural\ Strength\ Ratio\left(\frac{F_f}{F_0}\right) = -3.17f^2 + 4.12f + 1.00 \qquad \text{----- Eq. 4.4}$$

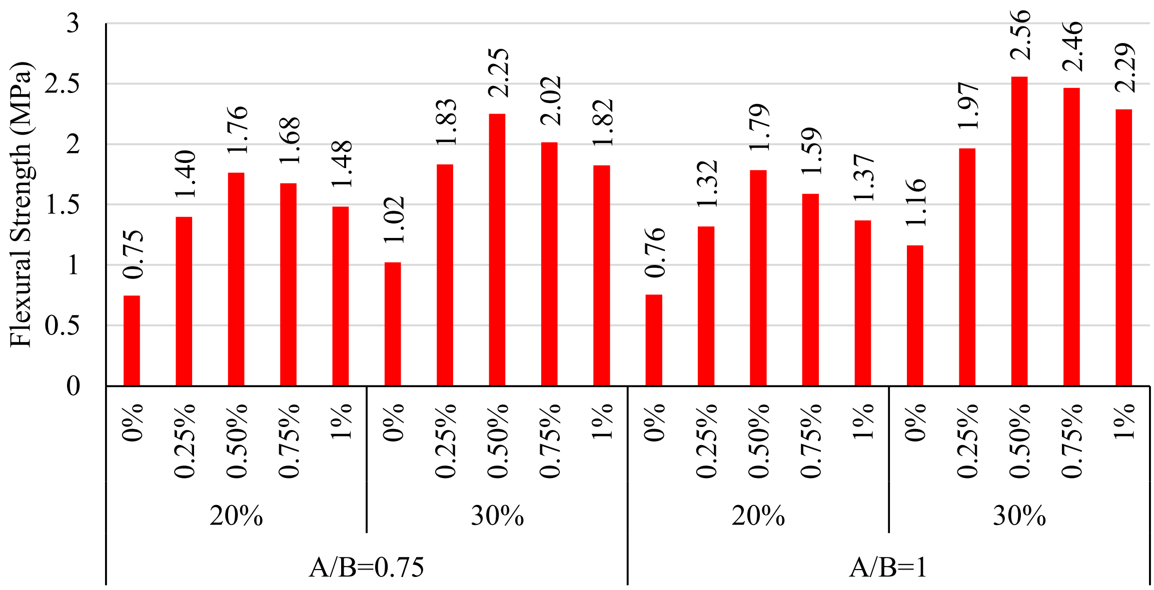



**Fig. 4.38. Variation of Flexural Strength with fiber dosage for mixes at 0.75$w_L$**

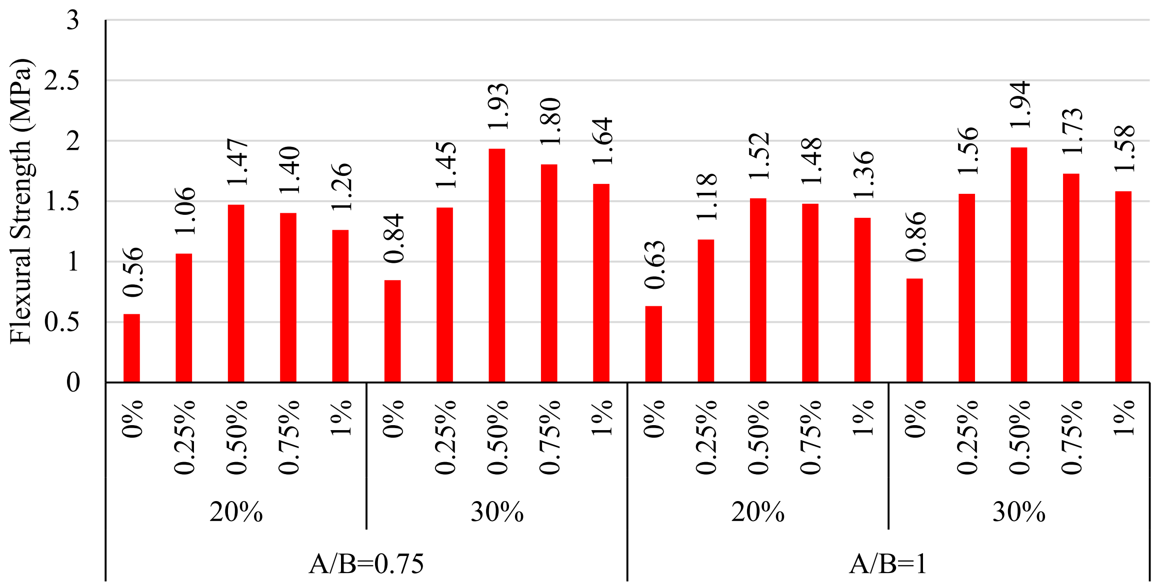



**Fig. 4.39. Variation of Flexural Strength with fiber dosage for mixes at $w_L$**

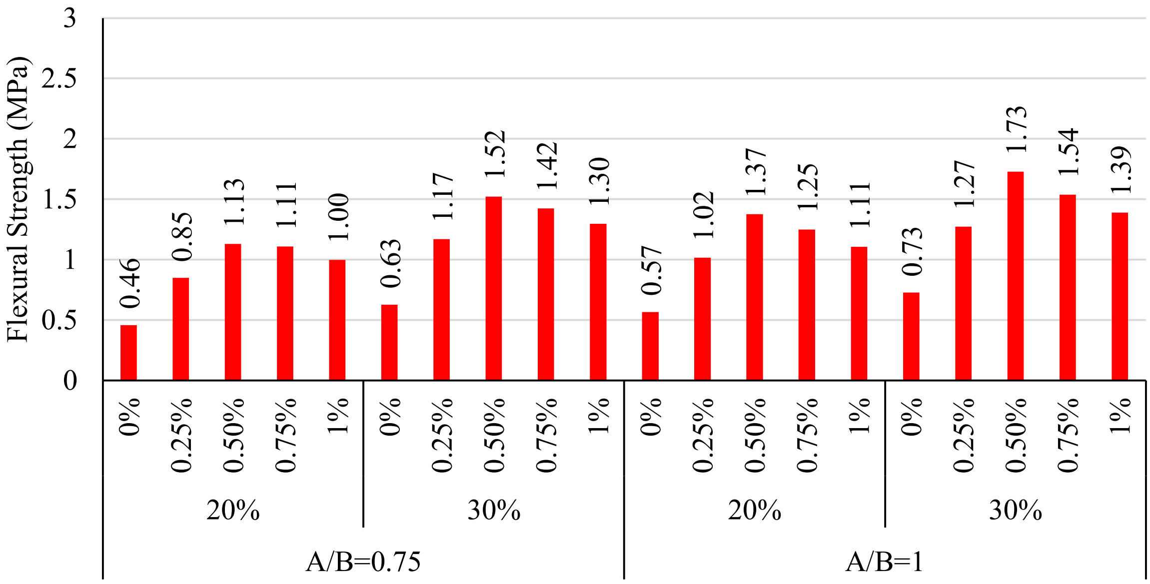



**Fig. 4.40. Variation of flexural strength with fiber dosage for mixes at 1.25$w_L$**

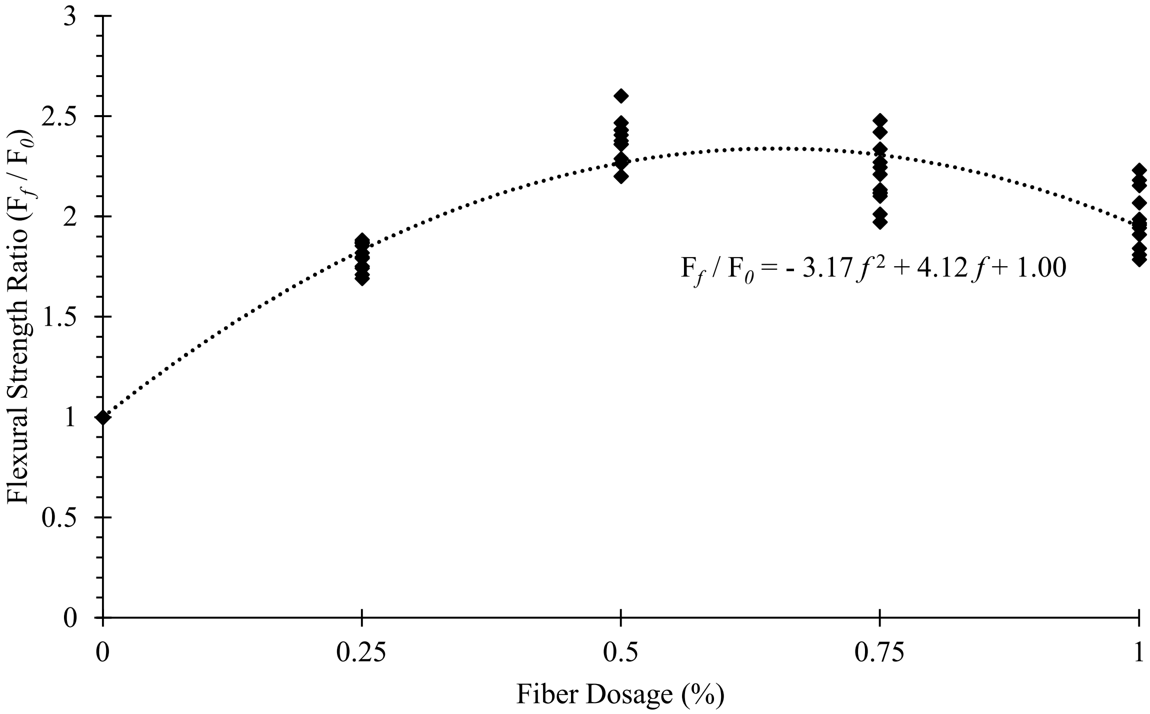



**Fig. 4.41. Flexural strength ratio ($F_f / F_0$) vs fiber dosage**

Equation 4.4 can be used to approximately assess the flexural strength of FGP mixes at any fiber content up to 1% if the flexural strength of the respective unreinforced GP mix is known, thus it can be helpful for engineers and researchers in designs and numerical modelling.

# 4.4. Durability of unreinforced and fiber reinforced geopolymer stabilized soft clay against wetting and drying

The temperature and moisture changes in field influence the performance of the stabilized soil with respect to durability against wetting and drying. As discussed in Chapter-2, limited literature is available on the performance of soft clay stabilized with geopolymer (reinforced or unreinforced with fibers) with respect to durability characteristics. Thus, this section discusses Phase-III part of the study, i.e., the durability of unreinforced and fiber reinforced soil-geopolymer specimens subjected to twelve wetting-drying (w-d) cycles after 28 days curing period. Three specimens for each mix at soil water content of $0.75w_L$ were cast as described in Chapter-3 to study the volume change, mass loss and residual UCS at the end of each w-d cycle. The mix designations of the mixes considered in this phase of study are similar to the ones in the previous section. The stabilized soil specimens that endured all the 12 w-d cycles with volume change < 10% (Pedarla et al. 2011) and residual UCS in the target UCS range is considered as the governing criteria to characterize the stabilized soil specimens as durable against wetting and drying.

## 4.4.1. Durability of unreinforced soil-geopolymer

The results of wetting-drying tests for the unreinforced soil-geopolymer (GP) specimens for mass loss, volume change and residual strength are presented and discussed in the following sub-sections. All the soil-geopolymer specimens survived throughout the 12 w-d cycles.

### 4.4.1.1. Mass Loss

The mass loss (%) of the unreinforced soil-geopolymer mix specimens subjected to w–d cycles is presented in Figure 4.42. It can be observed that there is a reduction in mass of the specimens for each w-d cycle. From Figure 4.42, it can be understood that a significant amount of mass loss is observed for GP-treated mixes up to the second cycle and thereafter a gradual increment was observed up to the third cycle and later the mass loss was constant up to the twelfth cycle. The increase of mass loss up to the third cycle maybe due to the leaching of unreacted and unbonded clay and GGBS particles. However, after three cycles, the geopolymeric network was strongly formed such that the mass loss of the specimens was not much affected by w-d cycles. With an increase in binder content from 20% to 30%, reduction in mass loss was observed for all the w-d cycles, from which it can be understood that increasing the geopolymer content leads to a stronger interparticle bonding and improved resistance to disintegration or particle detachment. This increased bonding has reduced the mass loss and enhances the durability of the soil-geopolymer mixes. Further with increase in A/B ratio from 0.75 to 1.0, the mass loss was increased for all the w-d cycles, and this can be attributed to the formation of a more porous geopolymer matrix at higher alkali content. The presence of increased voids or pores can facilitate the ingress of moisture, chemicals, and gases into the specimens, which can negatively impact durability. The mass loss is observed to be minimum (13.02%) for GP mixes with binder content of 30% and A/B ratio of 0.75 at the end of 12 w-d cycles.

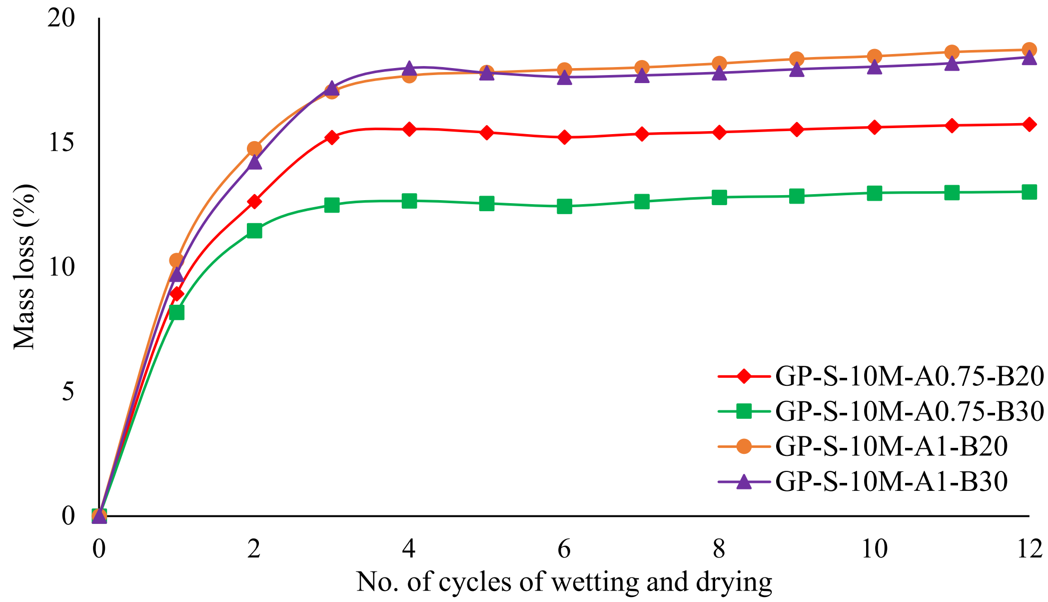



**Fig. 4.42. Mass loss of soil-geopolymer specimens subjected to w-d cycles**

#### 4.4.1.2. Volume change

The volume change of the GP binder stabilised specimens subjected to 12 wetting and drying (w–d) cycles is presented in Figure 4.43. The GP binder stabilised shows less variability in its volume change behaviour as the geopolymer binder fills the pore spaces in the soil matrix and forms a stable and dense matrix, reducing the ability of the soil to absorb and release moisture. This reduced moisture absorption and retention capability further decreases the volume change during wetting and drying cycles. The GP stabilised specimens with 20% and 30% binder content and A/B = 0.75 showed the least volume change amongst the other specimens as shown in Figure 4.43. The increase in volume change is observed for

A/B ratio of 1.0 and this may be due to higher availability of alkali content which can lead to slight increased shrinkage of geopolymer stabilized soils during drying cycles. This occurs because a higher concentration of alkaline activator solution results in the formation of partially formed gels and flocculated matrix at few places which cause internal forces and pore spaces.

The volume change is observed to be < 10% for all the mixes in the study and is found to be minimum (5.71%) for GP mixes with binder content of 30% and A/B ratio of 0.75 at the end of 12 w-d cycles. Thus, all the GP mixes satisfied the durability criterion for volume change.

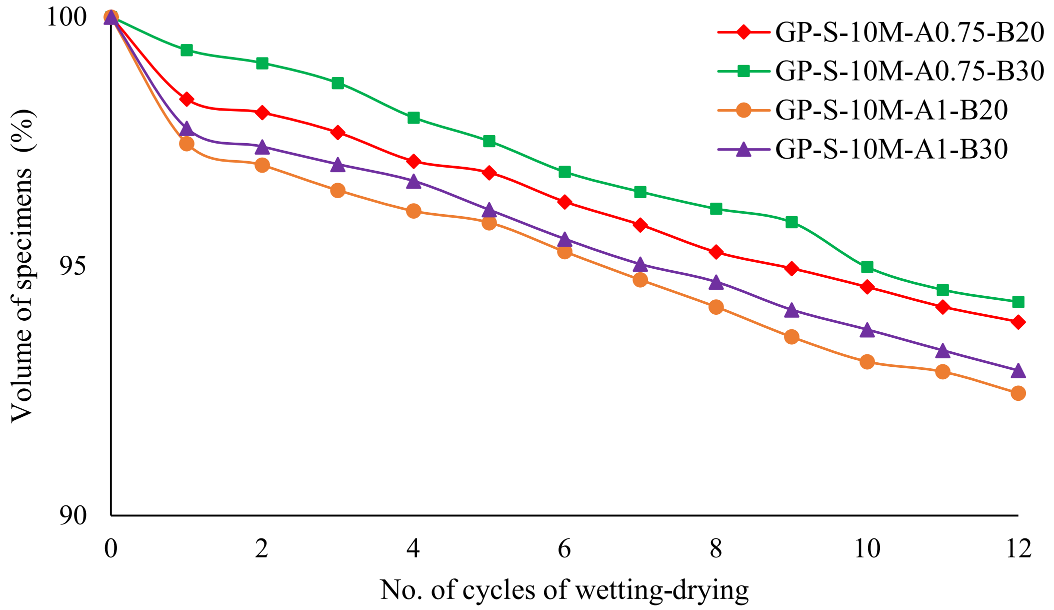



**Fig. 4.43. Volume change of soil-geopolymer specimens subjected to w-d cycles**

### 4.4.1.3. Residual strength

The UCS of the treated samples subjected to w–d cycles are tested after 1, 3, 6, 9, and 12 cycles and is termed as residual strength or residual UCS. A separate set of GP specimens is taken for all the mixes for determining the UCS of the specimens subjected to wetting and drying. Figure 4.44 shows the residual UCS of the GP specimens at different number of w–d cycles. The residual UCS of GP specimens decreased with the number of cycles. A sudden reduction in strength is recorded after the first cycle for specimens tested in wetting state. Considerable macro-cracks and surface deterioration are observed with an increase in the number of cycles up to the third cycle (Figure 4.44). After the third w-d cycle, the loss in strength was negligible and the residual UCS is observed to be almost constant from 6 to 12 cycles. This might be due to the formation of a strong geopolymeric network internally throughout the treated specimen which has shown resistance to wetting and drying. From Figure 4.44, all the GP specimens satisfied the target UCS of 1.034 to 6 MPa amongst which, the specimens with 30% binder content and A/B of 0.75 and 1.0 have shown higher residual UCS (≈ 2.60 MPa) at the end of 12 w-d cycles.

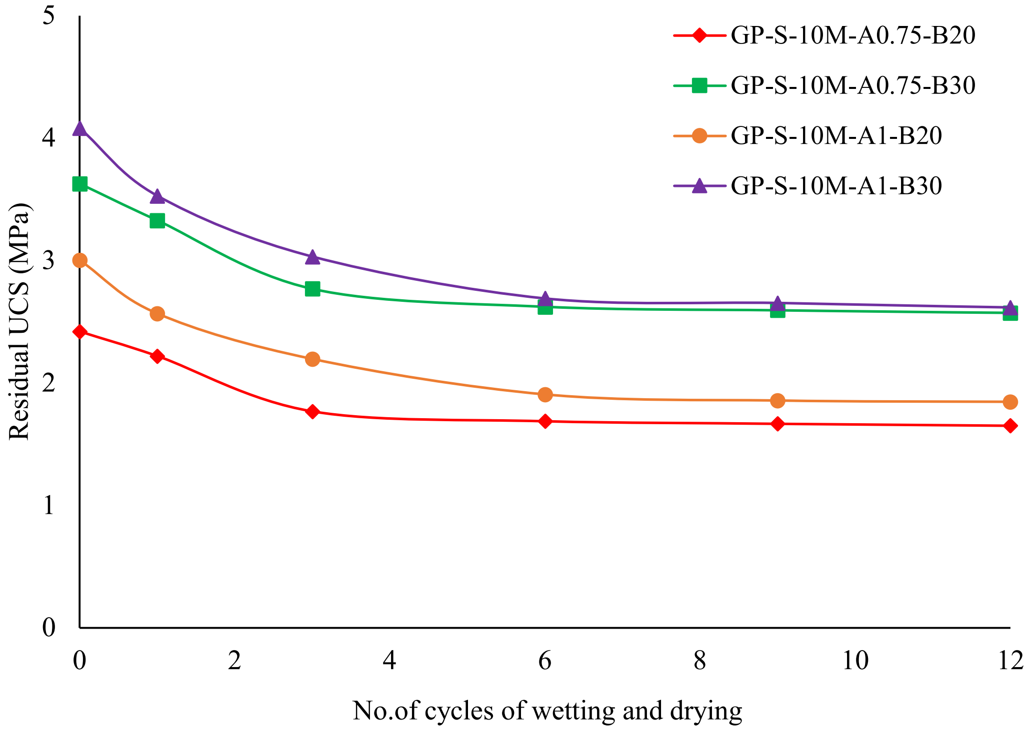



**Fig. 4.44. Residual UCS of soil-geopolymer specimens subjected to w-d cycles**

Thus, from the durability results of unreinforced soil-geopolymer (GP), it is observed that all the GP mixes have satisfied the durability requirements of lower mass loss, volume change < 10% and residual UCS in the range of the target UCS (1.034 to 6 MPa) and hence are durable against wetting and drying. The GP mix with 30% binder content and A/B = 0.75 is observed to be durable against wetting and drying in terms of volume change, mass loss and residual UCS.

## 4.4.2. Durability of polypropylene fiber reinforced soil-geopolymer

The results of wetting-drying tests for the polypropylene fiber reinforced soil-geopolymer (FGP) specimens for mass loss, volume change and residual strength are presented and discussed in the following sub-sections. All the FGP specimens survived throughout the 12 wetting-drying cycles.

#### 4.4.2.1. Mass Loss

The mass loss (%) of both unreinforced and fiber reinforced soil-geopolymer specimens subjected to 12 w–d cycles is presented in Figures 4.45 to 4.48. It can be observed that there is a reduction in mass of the specimens for each w-d cycle up to third cycle after which the mass is almost constant till the end of twelfth cycle for all the GP and FGP mix specimens. For all the FGP specimens, with increase in binder content from 20% to 30% and decrease in A/B ratio from 1.0 to 0.75, there is a reduction in mass loss for all the w-d cycles and the same is observed for GP specimens also. However, the FGP specimens exhibited lower mass loss (< 10%) as compared to the GP specimens (> 10%). This can be attributed to the bridging and crack arresting phenomenon of the fibers which minimized the formation and propagation of the desiccation cracks during oven drying of the specimens and due to their strong bond formation with the products of geopolymerisation and the clay particles surrounding them that prevented leaching of unreacted particles. Among all the FGP mixes with different A/B ratios, binder contents and fiber dosages, the mass loss is minimum for the FGP mixes with 1% fiber content having average mass loss of 3.92%.

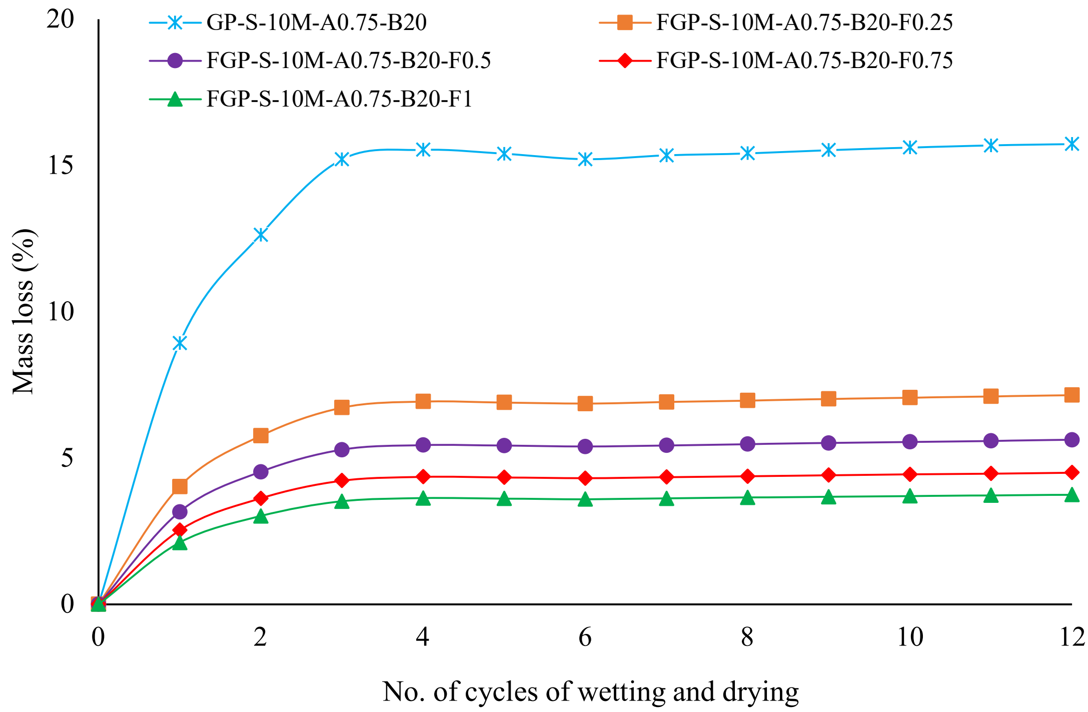



**Fig. 4.45. Mass loss of GP and FGP specimens with B = 20% and A/B = 0.75 subjected to w-d cycles**

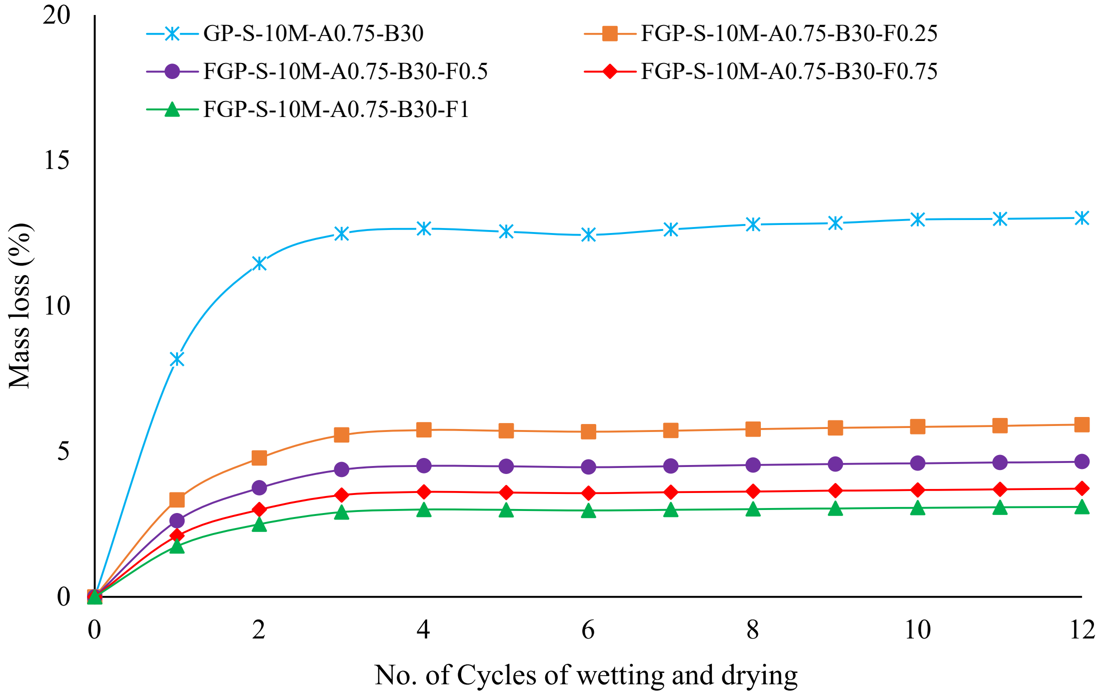



**Fig. 4.46. Mass loss of GP and FGP specimens with B = 30% and A/B = 0.75 subjected to w-d cycles**

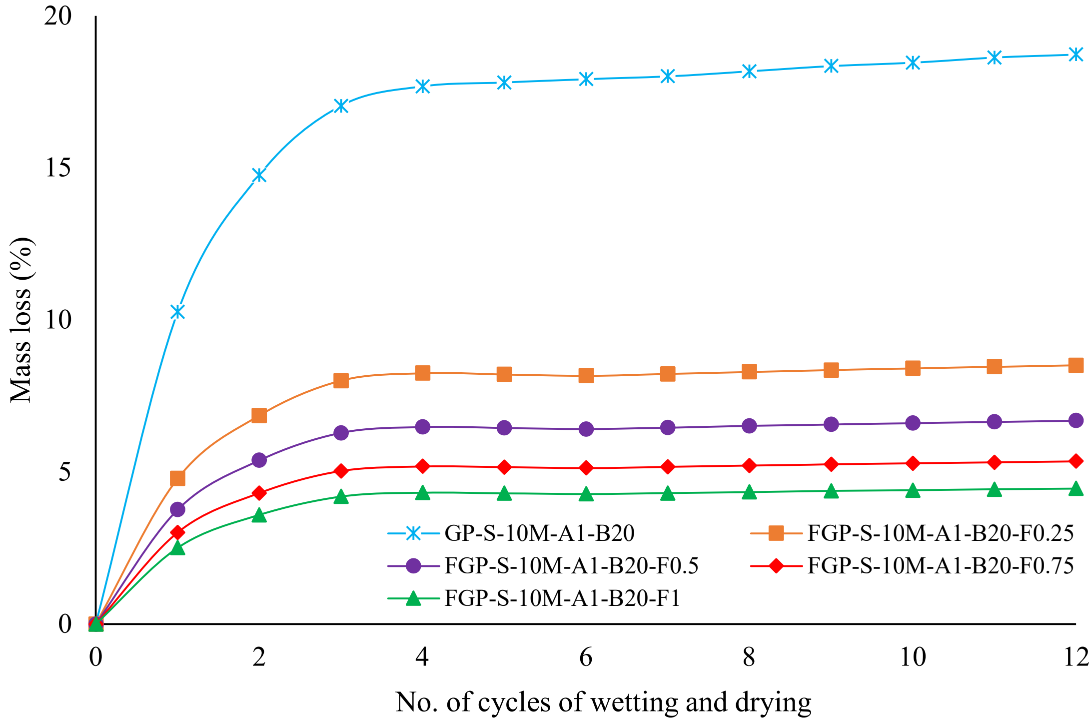



**Fig. 4.47. Mass loss of GP and FGP specimens with B = 20% and A/B = 1.0 subjected to w-d cycles**

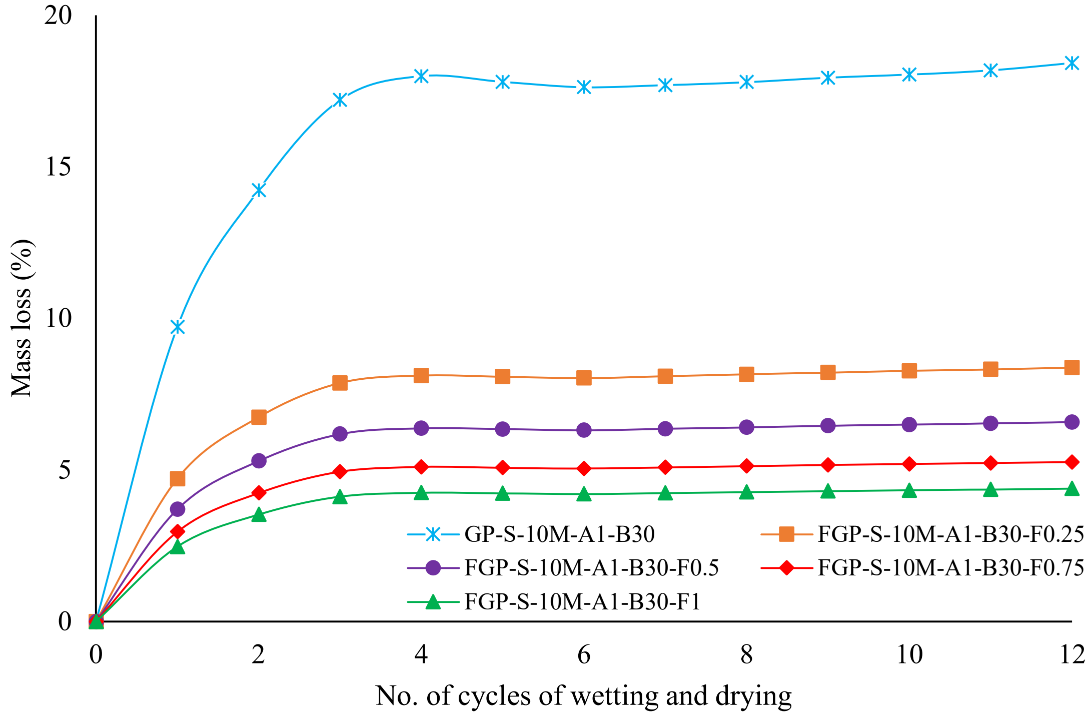



**Fig. 4.48. Mass loss of GP and FGP specimens with B = 30% and A/B = 1.0 subjected to w-d cycles**

#### 4.4.2.2. Volume Change

The volume of both unreinforced and fiber reinforced soil-geopolymer (GP and FGP) specimens subjected to 12 w–d cycles is presented in Figures 4.49 to 4.52. It can be observed that there is an increment in volume change of the specimens for each w-d cycle for all the specimens. For all the FGP specimens, with increase in binder content from 20% to 30% and decrease in A/B ratio from 1.0 to 0.75, there is a reduction in volume change after every cycle and the same was observed for GP specimens. However, the FGP specimens have shown lower change in volume (< 5%) as compared to the GP specimens (> 5%). This can be attributed to the inclusion of fibers that helped to control the propagation of desiccation cracks within the stabilized soil specimens during every w-d cycle. By bridging across the desiccation cracks, PP fibers reduce the crack width and prevent further crack growth, which can minimize volume change and maintain stability in the matrix. Among all the FGP mixes with different binder contents, A/B ratios and fiber dosages, the volume change is minimum for the FGP mixes with 1% fiber content having average volume change of 2.56%. The volume change is observed to be < 10% for all the mixes in the study and is found to be minimum (2.02%) for FGP mixes with binder content of 30%, A/B ratio of 0.75 and fiber dosage of 1% at the end of 12 w-d cycles. Thus, all the FGP mixes also have satisfied the durability criterion for volume change.

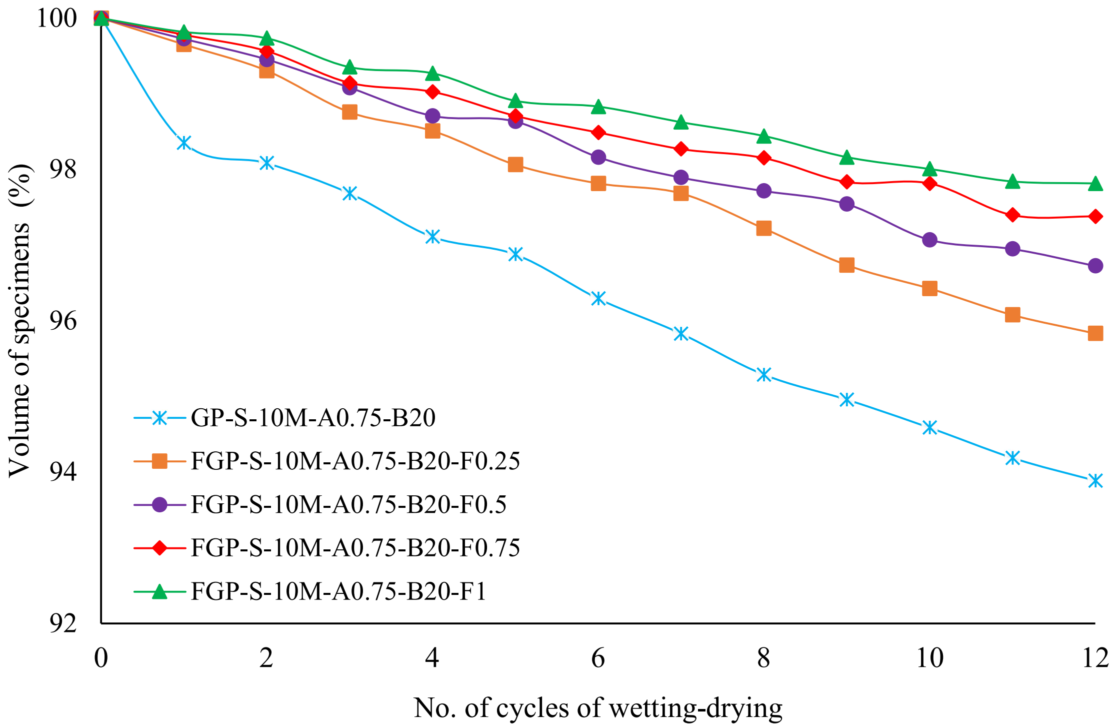



**Fig. 4.49. Volume of GP and FGP specimens with B = 20% and A/B = 0.75 subjected to w-d cycles**

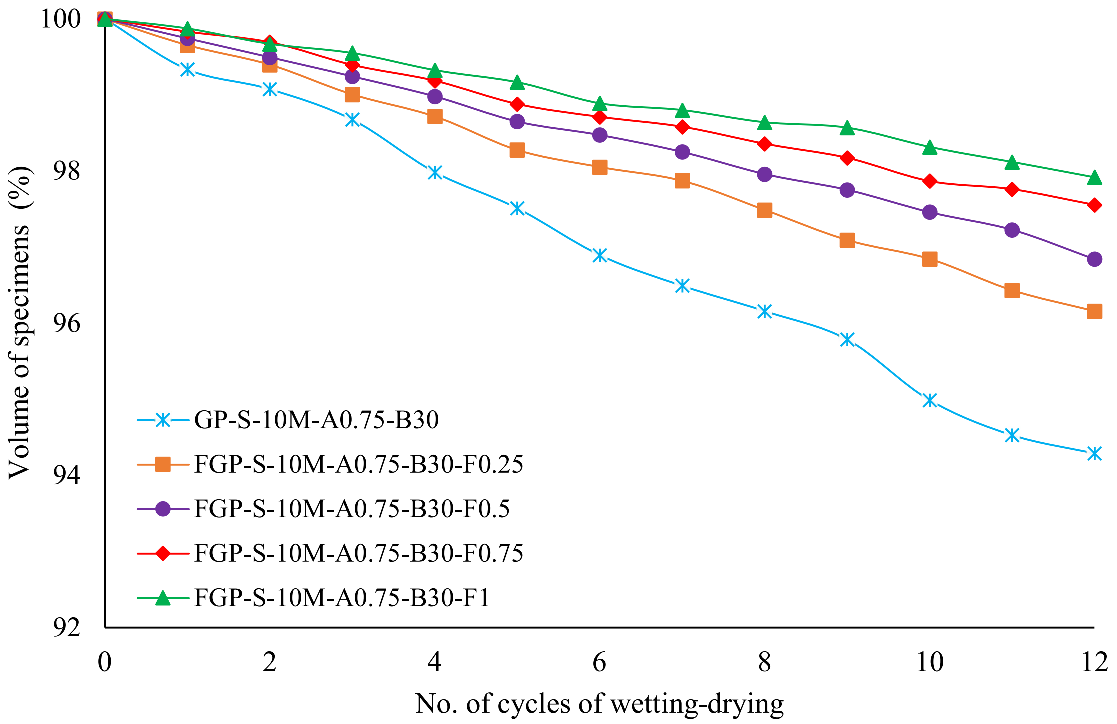



**Fig. 4.50. Volume of GP and FGP specimens with B = 30% and A/B = 0.75 subjected to w-d cycles**

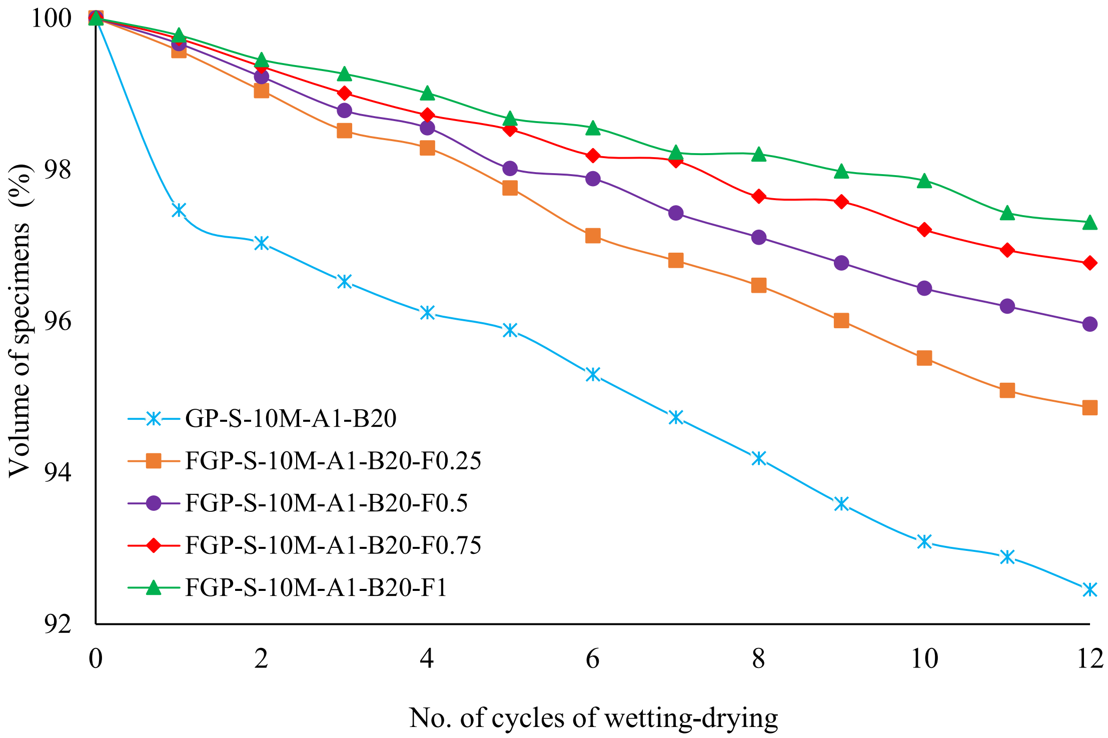



**Fig. 4.51. Volume of GP and FGP specimens with B = 20% and A/B = 1.0 subjected to w-d cycles**

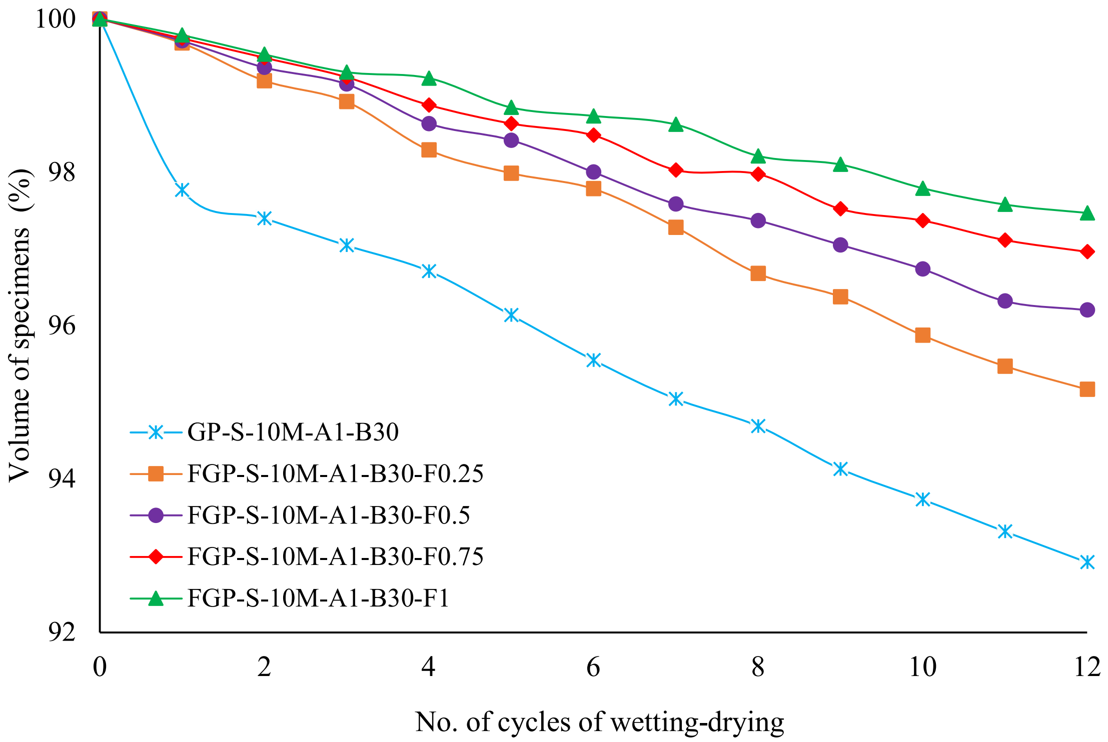



**Fig. 4.52. Volume of GP and FGP specimens with B = 30% and A/B = 1.0 subjected to w-d cycles**

#### 4.4.2.3. Residual Strength

The UCS of the GP and FGP specimens subjected to wetting and drying are tested after 1, 3, 6, 9 and 12 w-d cycles, and is termed as residual strength or residual UCS. Figures 4.53 to 4.56 show the residual UCS of both GP and FGP mix specimens at different number of w–d cycles, from which it can be observed that residual UCS of all the specimens decreased with increase in the number of w-d cycles up to the third cycle. After the 3$^{rd}$ w-d cycle, the loss in UCS was negligible and the residual UCS is observed to be almost constant from 6$^{th}$ to 12$^{th}$ cycle for all the mixes. This might be due to the formation of a strong geopolymeric network internally throughout the stabilized specimen and control of desiccation crack propagation by PP fibers which has shown resistance to wetting and drying. Thus, the FGP specimens have shown higher residual UCS as compared to the GP specimens. This can be attributed to the better bonding and interlocking between the soil particles, hardened geopolymer and PP fibers. The macro-cracks and surface deterioration are also observed to be minimum for FGP specimens with an increase in the fiber content. Though all the GP and FGP specimens satisfied the target UCS of 1.034 to 6.0 MPa, the FGP specimens with 30% binder content, A/B ratio of 0.75 and 1.0 have shown better residual UCS values (>3.0 MPa) for all fiber dosages at the end of 12 w-d cycles. Among all the FGP mixes, the mixes with 0.5% PP fiber dosage have exhibited higher residual UCS under the w-d cycles at any A/B ratio and binder content.

Thus, from the durability results of fiber reinforced soil-geopolymer (FGP), it is observed that all the FGP mixes have satisfied the durability requirements such as lower mass loss, volume change < 10% and residual UCS in the range of the target UCS (1.034 to 6 MPa) and hence are durable against wetting and drying. The FGP mix with fiber dosage of 1%, binder content of 30%, A/B ratio of 0.75 is observed to have performed better in durability against wetting and drying in terms of volume change, mass loss and residual UCS as compared to the other FGP and GP mixes.

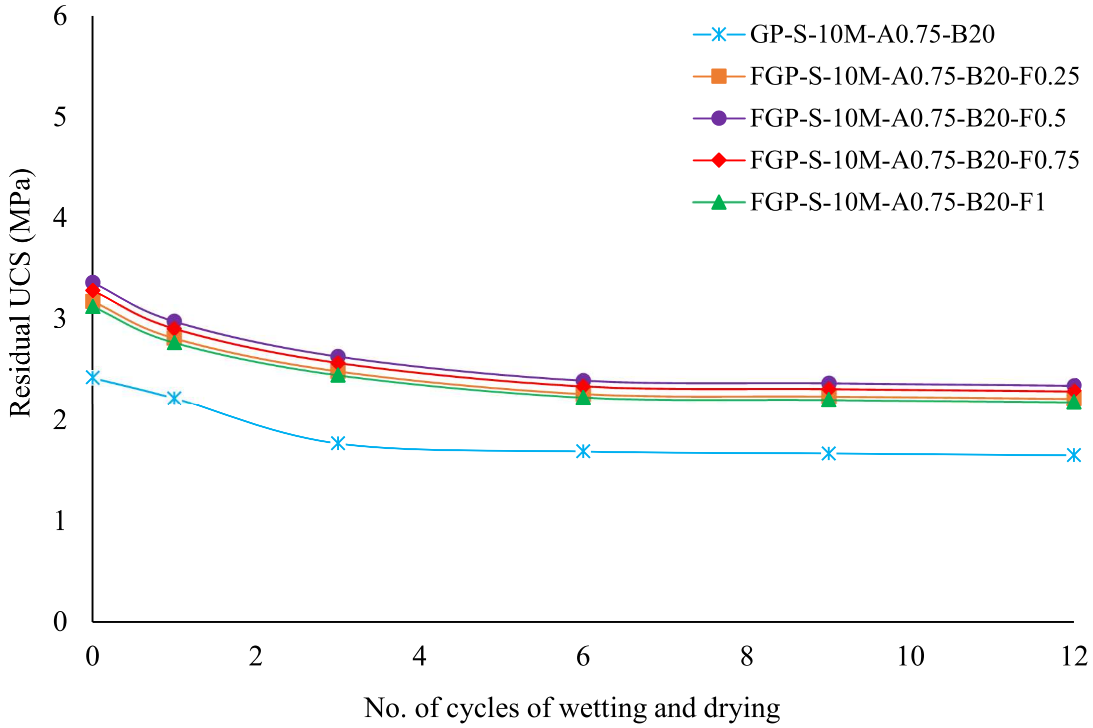



**Fig. 4.53. Residual UCS of GP and FGP specimens with B = 20% and A/B = 0.75 subjected to w-d cycles**

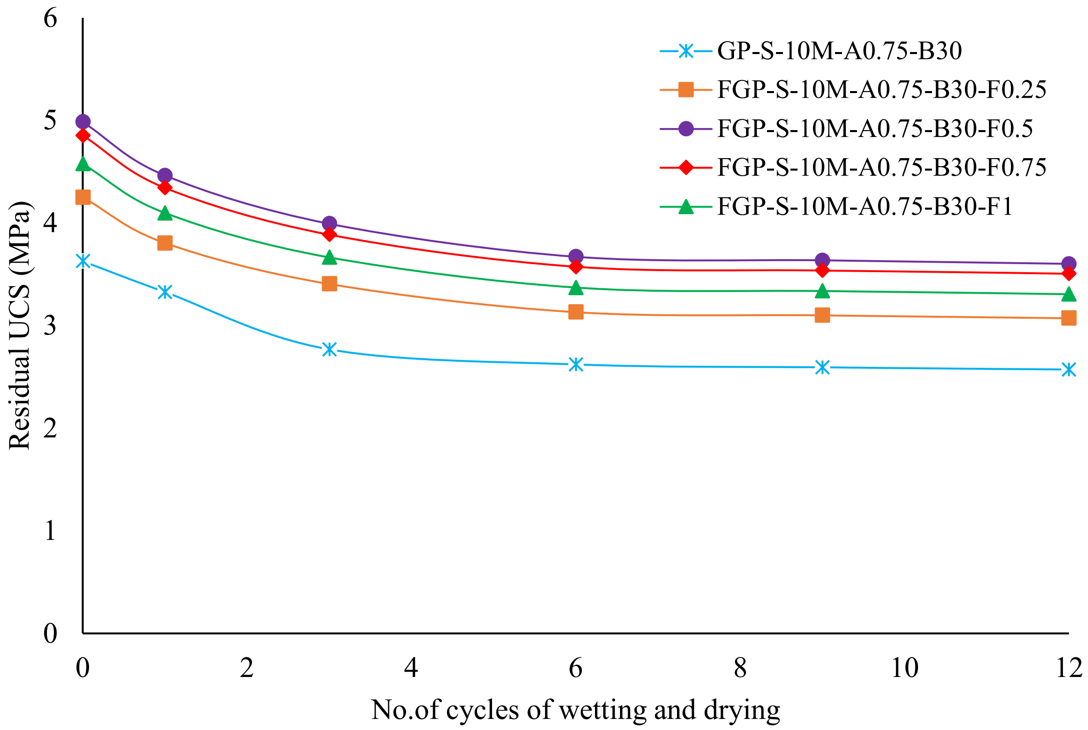



**Fig. 4.54. Residual UCS of GP and FGP specimens with B = 30% and A/B = 0.75 subjected to w-d cycles**

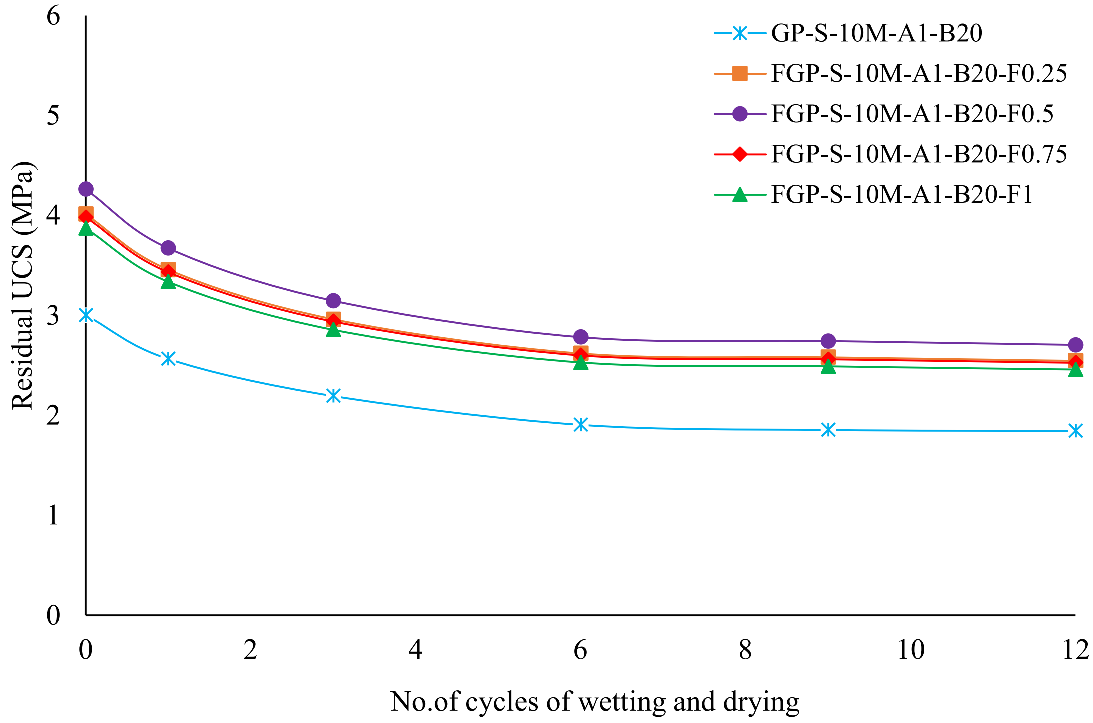



**Fig. 4.55. Residual UCS of fiber reinforced soil-geopolymer specimens subjected to w-d cycles at B = 20% and A/B = 1.0**

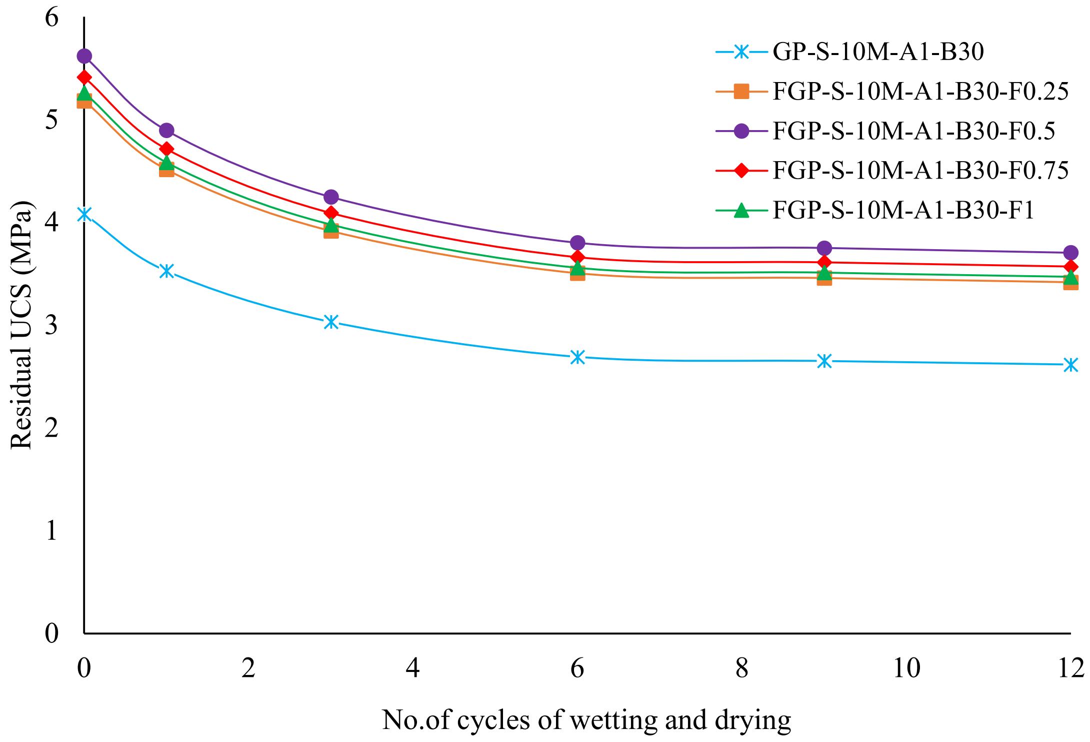



**Fig. 4.56. Residual strength of fiber reinforced soil-geopolymer specimens subjected to w-d cycles at B = 30% and A/B = 1.0**

## 4.5. Bearing capacity study on model composite soft clay bed

To understand the behaviour of GP and FGP columns when used as reinforcement to the model soft clay bed simulating the actual field condition of DSM columns in soft ground, model tank tests are performed to evaluate the bearing capacity of the GP and FGP column stabilized model soft clay bed under uniaxial loading. The effect of parameters such as binder type, binder content, fiber inclusion, number of columns (area ratio) and column type or column end condition on the ultimate bearing capacity of the improved soft clay bed is investigated.

The ultimate load is defined as the point at which the slope of the load-settlement curve first reaches zero or a steady, minimum value (Vesic, 1973). For presentation of results, the settlement of the footing plate is normalized by the width of the footing and the plots are drawn with applied vertical stress (kPa) against settlement/footing width. The ultimate bearing capacity of the model soft clay bed improved with end bearing and floating GP and FGP columns is evaluated and compared with that of the model soft clay bed improved with soil-cement (C) columns in this study. For convenient reading, model soft clay bed improved with end bearing and floating C, GP and FGP columns is mentioned as composite soft clay bed with C-EB & C-FL, GP-EB & GP-FL and FGP-EB & FGP-FL columns respectively.

### 4.5.1. Variables of the Phase-IV study

The laboratory test tank study is conducted on a model soft clay bed under uniaxial loading as described in Section 3.2.2 of Chapter–3 for soil moisture content of $0.75w_L$ ($w_L$ is liquid limit of the soil). The binder content for both binders, cement and geopolymer, varied as 20% and 30% by dry weight of soil, while keeping the A/B ratio and NaOH molarity at 0.75 and 10M respectively. The effect of fiber inclusion to the soil-geopolymer columns is studied at PP fiber content of 1% by dry weight of soil. Model test tank studies are conducted for composite soft clay bed with single column and group columns in 2x2 and 3x3 column arrangement and thus, the effect of number of columns under an area of 400 sq.cm. (equal to the area of footing plate) is studied with the help of improvement area ratio ($A_r$) which is defined as the ratio of the total area improved with columns to the total area under load (area of footing plate, i.e., 400 sq.cm.). The improvement area ratios for single, 2x2 group and 3x3 group columns are calculated as 1.23%, 4.91% and 11.05% respectively. The effect of column type or column end condition, that is, end bearing or floating is also evaluated. Thus, a total of

37 tests are performed with variables and their respective proportions as given in Table 4.6. The designations used for these model tests are presented in Table 4.7. More model test tank details including the preparation of the soft clay bed, column installation, and schematic diagrams of the scaled models and loading procedure are given in Chapter–3.

**Tabe 4.6. Variables of Phase-IV study**

| Parameters | Geopolymer | Cement |
|---|---:|---:|
| Materials | GGBS | OPC |
| Alkali or activator | NaOH | Water |
| Molarity of NaOH | 10M | - |
| A/B ratio or w/C ratio | 0.75 | 0.4 |
| Soil water content (%) | $0.75w_L$ | $0.75w_L$ |
| Binder content (%) | 20, 30 | 20, 30 |
| Fiber content (%) | 0, 1 | 0 |
| Type of Columns | End Bearing, Floating | End Bearing, Floating |
| Number of Columns | 1, 4 (2x2), 9 (3x3) | 1, 4 (2x2), 9 (3x3) |

**Tabe 4.7. Designations for model tests on column improved soft clay**

| Tank / column Designation | Type of Binder | Binder Content (%) | Fiber Content (%) | No. of Columns | Column end condition |
|---|---|---|---|---|---|
| Unimproved Soil | None | - | - | - | - |
| C-S-B20-Single-EB | Cement | 20% | - | 1 (Single) $A_r$=1.23% | End Bearing |
| C-S-B30-Single-EB | | 30% | - | | |
| GP-S-10M-A0.75-B20-Single-EB | Geopolymer | 20% | - | | |
| GP-S-10M-A0.75-B30-Single-EB | | 30% | - | | |
| FGP-S-10M-A0.75-B20-F1-Single-EB | | 20% | 1 | | |
| FGP-S-10M-A0.75-B30-F1-Single-EB | | 30% | 1 | | |
| C-S-B20-2x2Group-EB | Cement | 20% | - | 4 (2x2 Group) $A_r$=4.91% | |
| C-S-B30-2x2Group-EB | | 30% | - | | |

| | | | | | |
|---|---|---|---|---|---|
| GP-S-10M-A0.75-B20-2x2Group-EB | Geopolymer | 20% | - | | |
| GP-S-10M-A0.75-B30-2x2Group-EB | | 30% | - | | |
| FGP-S-10M-A0.75-B20-F1-2x2Group-EB | | 20% | 1 | | |
| FGP-S-10M-A0.75-B30-F1-2x2Group-EB | | 30% | 1 | | |
| C-S-B20-3x3Group-EB | Cement | 20% | - | 9 (3x3 Group) $A_r$=11.05% | |
| C-S-B30-3x3Group-EB | | 30% | - | | |
| GP-S-10M-A0.75-B20-3x3Group-EB | Geopolymer | 20% | - | | |
| GP-S-10M-A0.75-B30-3x3Group-EB | | 30% | - | | |
| FGP-S-10M-A0.75-B20-F1-3x3Group-EB | | 20% | 1 | | |
| FGP-S-10M-A0.75-B30-F1-3x3Group-EB | | 30% | 1 | | |
| C-S-B20-Single-FL | Cement | 20% | - | 1 (Single) $A_r$=1.23% | Floating |
| C-S-B30-Single-FL | | 30% | - | | |
| GP-S-10M-A0.75-B20-Single-FL | Geopolymer | 20% | - | | |
| GP-S-10M-A0.75-B30-Single-FL | | 30% | - | | |
| FGP-S-10M-A0.75-B20-F1-Single-FL | | 20% | 1 | | |
| FGP-S-10M-A0.75-B30-F1-Single-FL | | 30% | 1 | | |
| C-S-B20-2x2Group-FL | Cement | 20% | - | 4 (2x2 Group) $A_r$=4.91% | |
| C-S-B30-2x2Group-FL | | 30% | - | | |
| GP-S-10M-A0.75-B20-2x2Group-FL | Geopolymer | 20% | - | | |
| GP-S-10M-A0.75-B30-2x2Group-FL | | 30% | - | | |
| FGP-S-10M-A0.75-B20-F1-2x2Group-FL | | 20% | 1 | | |
| FGP-S-10M-A0.75-B30-F1-2x2Group-FL | | 30% | 1 | | |

| | | | | | |
|---|---|---|---|---|---|
| C-S-B20-3x3Group-FL | Cement | 20% | - | 9 (3x3 Group) $A_r$=11.05% | |
| C-S-B30-3x3Group-FL | | 30% | - | | |
| GP-S-10M-A0.75-B20-3x3Group-FL | Geopolymer | 20% | - | | |
| GP-S-10M-A0.75-B30-3x3Group-FL | | 30% | - | | |
| FGP-S-10M-A0.75-B20-F1-3x3Group-FL | | 20% | 1 | | |
| FGP-S-10M-A0.75-B30-F1-3x3Group-FL | | 30% | 1 | | |

### 4.5.2. End bearing column condition

The model load test results, depicted in Figures 4.57 to 4.59, illustrate the relationship between the applied vertical stress and the normalized settlement of the footing. These tests are conducted on composite soft clay beds with single, 2x2 group and 3x3 group C-EB, GP-EB, and FGP-EB columns. The plots also present the curve of the control load test conducted on an unimproved soft clay bed for comparison. The maximum applied vertical stress in all the plots directly indicates the ultimate bearing capacity of the composite soft clay bed. From these figures, it can be observed that even a single column could improve the values of ultimate stress on soft clay bed significantly.

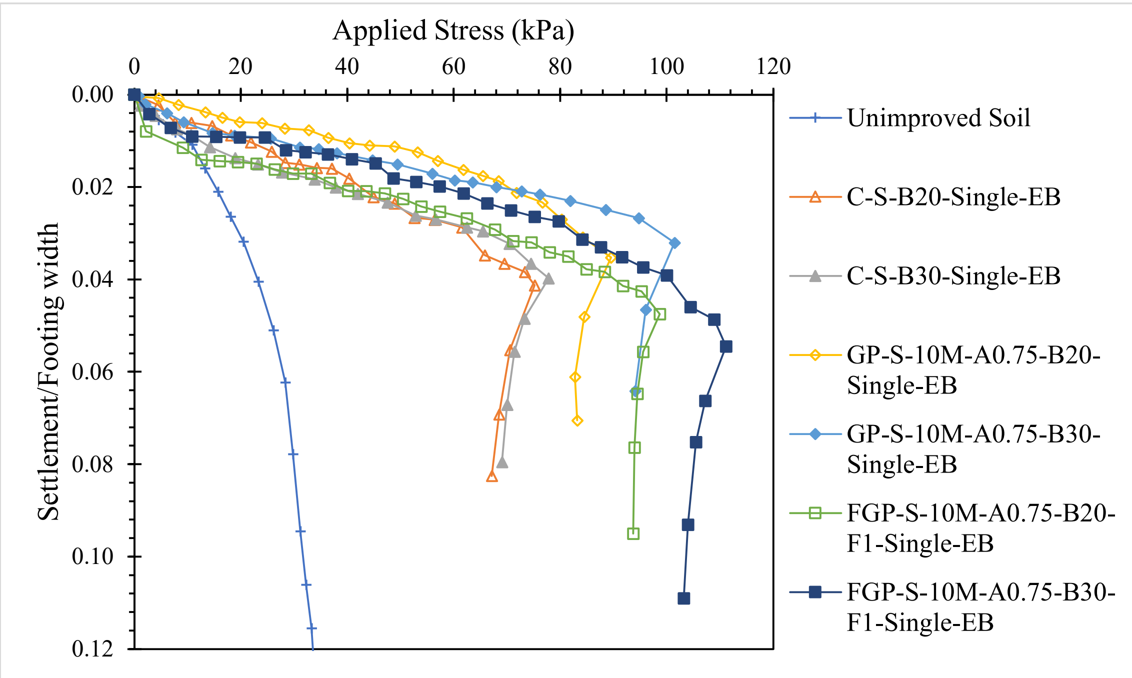



**Fig. 4.57. Applied Pressure vs Settlement/Footing width curves for soft clay bed with Single column – End bearing condition**

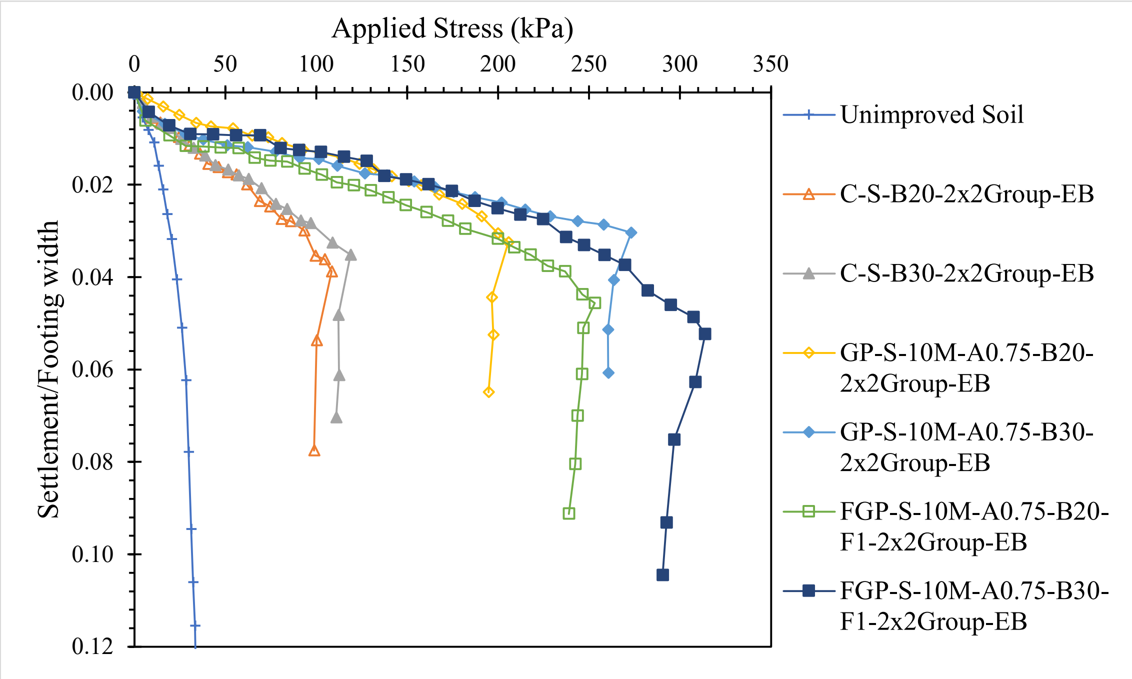



**Fig. 4.58. Applied Pressure vs Settlement/Footing width curves for soft clay bed with 2x2 group columns – End bearing condition**

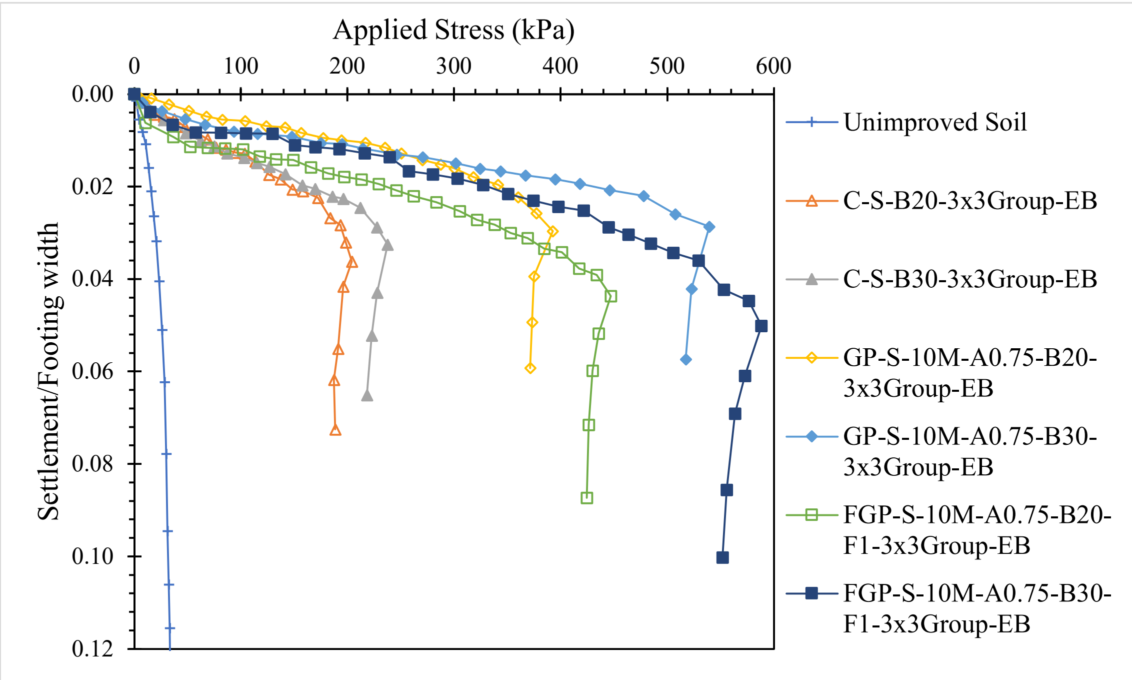



**Fig. 4.59. Applied Pressure vs Settlement/Footing width curve for 3x3 group columns – End bearing condition**

The figures also demonstrate that, for identical improvement area ratios ($A_r$), the composite model ground with FGP-EB columns exhibited higher ultimate failure stress, followed by the GP-EB columns, and lastly the C-EB columns. It is also observed that composite soft ground with GP-EB columns exhibited higher stiffness (indicating sudden brittle failure at ultimate load) than that with FGP-EB columns and C-EB columns at any $A_r$ and binder content. The increased stiffness or rigidity and resistance to failure of the ground are attributed to the presence of EB columns that generate higher levels of stress. The FGP-EB columns displayed lower stiffness than GP-EB columns indicating increased displacement under load. This can be attributed to the PP fiber mechanism in the FGP columns. Although the fiber reinforcement in the soil-geopolymer columns improved their ductility, the composite ground with FGP-EB columns displayed a sudden drop in the curve at ultimate failure stress. This may be due to the buckling or bending of the FGP columns under increased load.

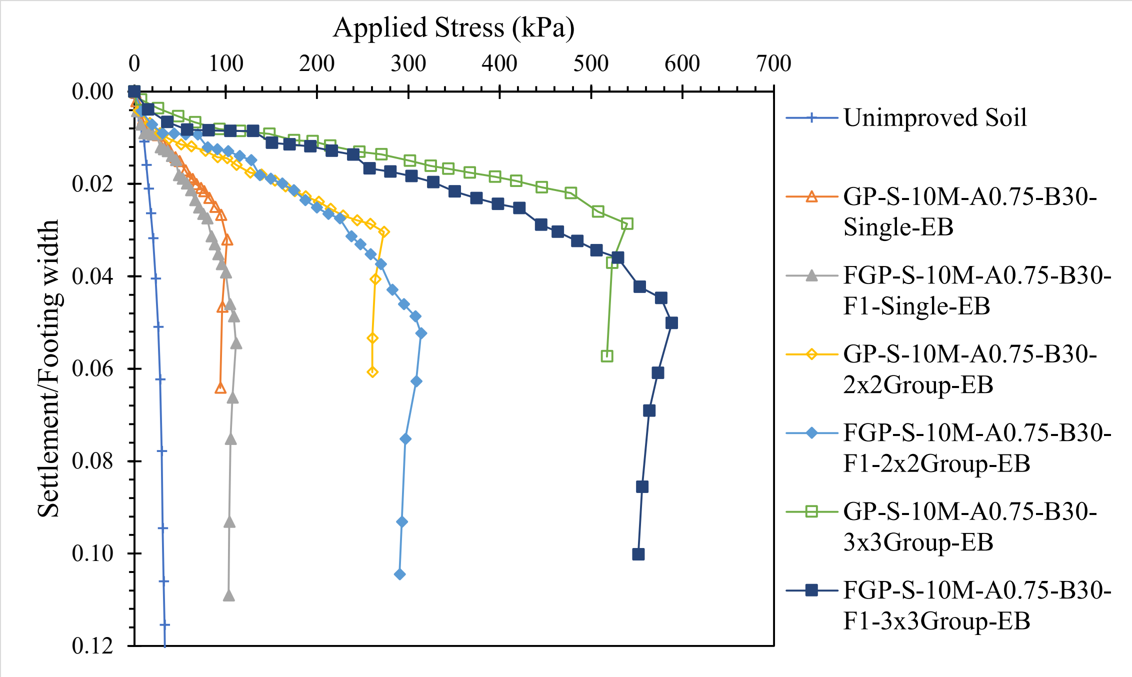



**Fig. 4.60. Applied Pressure vs Settlement/Footing width of the soft clay bed with single, 2x2 and 3x3 group columns – Effect of Area ratio – End Bearing condition**

Figure 4.60 shows a comparison of applied stress versus normalized settlement curves for composite ground with GP-EB and FGP-EB columns for different improvement area ratios ($A_r$). With an increase in $A_r$ from 1.23% to 11.05%, the ultimate failure stress is increased drastically indicating a much greater bearing capacity of the composite ground. The soft clay bed displays a stress-settlement curve that closely resembles the curve observed in typical shear failure, which is indicated by a clear peak failure stress. As the $A_r$ increases, both the stiffness and failure stress of the composite soft clay bed improve as depicted in Figure 4.60. This is because the composite soft bed contains a greater amount of hardened soil-cement or soil-geopolymer material with increase in $A_r$. However, the increase in stiffness with an increase in $A_r$ is comparatively higher in the case of composite ground with GP-EB columns compared to that with FGP-EB columns.

Figure 4.61 illustrates the variation of experimental ultimate bearing capacity of composite soft clay bed with improvement area ratio, binder content and binder type in end bearing column condition. It can be noted that the composite soft ground with FGP-EB columns arranged in 3x3 group pattern ($A_r$ = 11.05%) at binder content of 30% exhibited higher ultimate bearing capacity (588.21 kPa), followed by the composite soft ground with GP-EB columns (539.27 kPa) having similar variables.

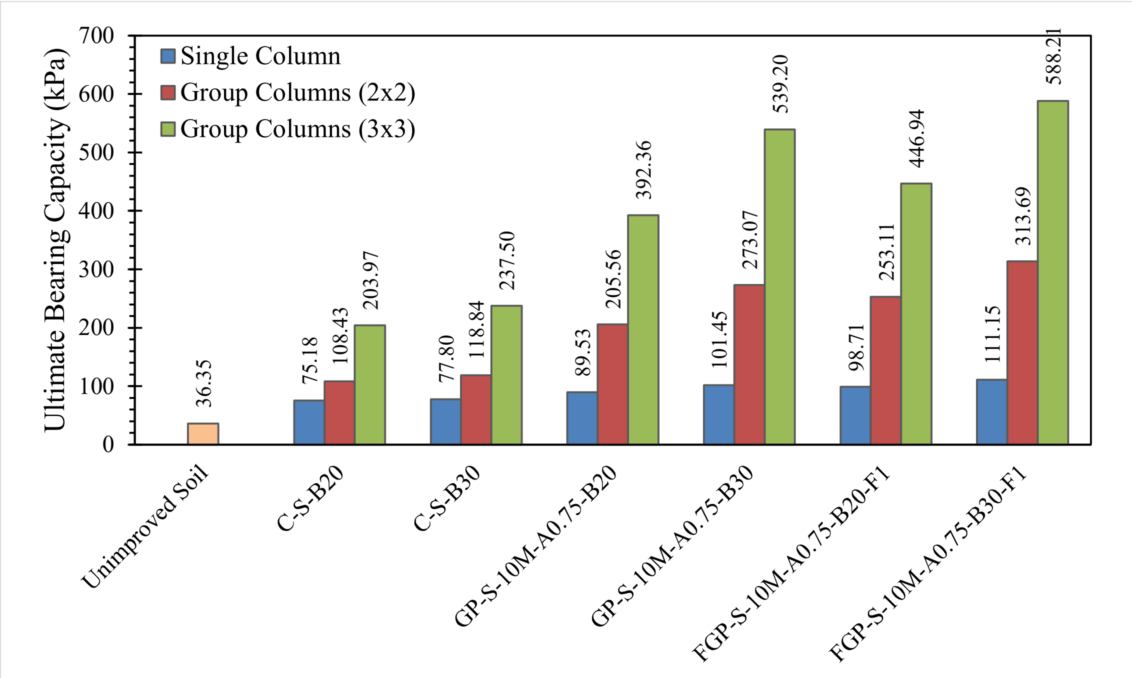



**Fig. 4.61. Variation of bearing capacity of the composite soft clay bed with C, GP and FGP columns – End bearing condition**

The bearing capacity values of the composite soft clay bed improved with C-EB, GP-EB and FGP-EB columns are calculated from the equations (as given below) given by the previous researchers for ground with soil-cement columns, using undrained shear strength of the soft clay ($c_{us}$) and undrained shear strength of the columns ($c_{uc}$) obtained from the UCS values of the respective mixes in this study. The $c_{us}$ of the unimproved model soft clay bed is taken as 20 kPa.

$q_u = c_{uc} \cdot \alpha + (1 - \alpha) \cdot c_{us}$ [Weighted method, Terzaghi (1943) and Vesic (1973)]

$q_u = 0.7\, q_{uc} \cdot \alpha + \lambda\,(1 - \alpha) \cdot c_{us}$ [Broms' method, Broms (2004)]

The estimated ultimate bearing capacity values thus obtained from equations given by previous researchers are plotted and compared with the experimental ultimate bearing capacity values from the present study as shown in Figure 4.62. It is observed that the experimental values from the model study are close to the estimated values and presented a similar trend. Thus, the present laboratory scale model test tank study could closely assess the ultimate bearing capacity of the composite soft ground with soil-cement, soil-geopolymer and fiber reinforced soil-geopolymer columns in end bearing condition with different improvement area ratios, binder types and binder contents.

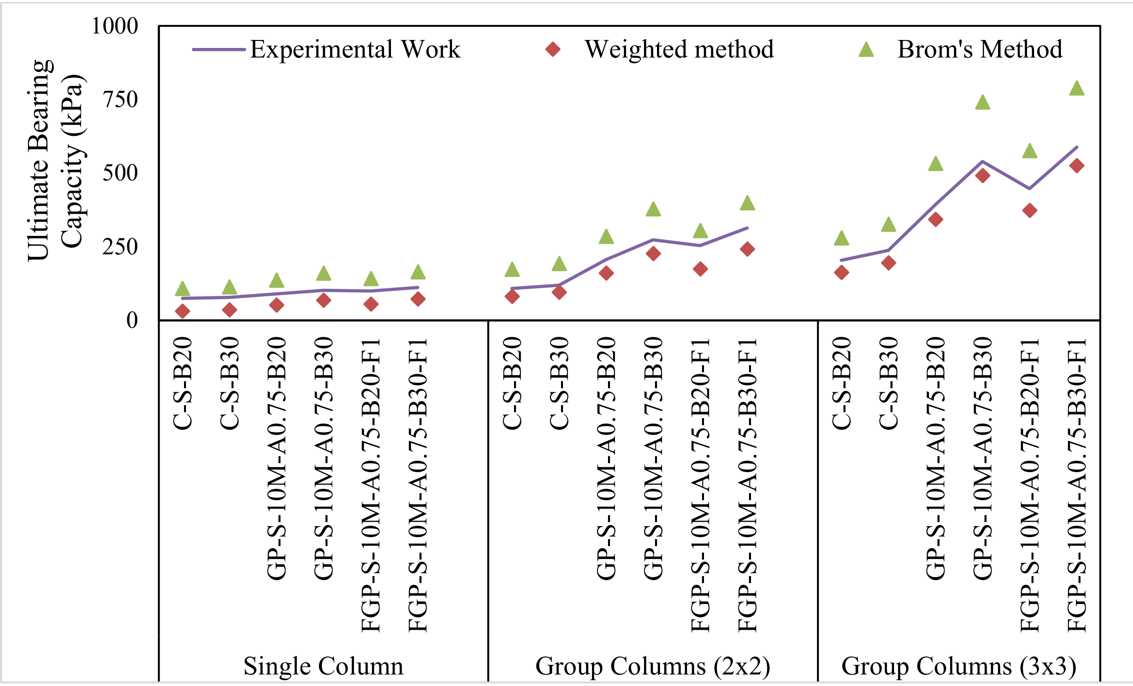



**Fig. 4.62. Comparison of experimental bearing capacity values with the ones obtained from equations given by previous researchers – End bearing column condition**

### 4.5.3. Floating columns condition

The results obtained from model test tank studies as depicted in Figures 4.63 to 4.65, illustrate the relationship between the applied vertical stress and the normalized settlement of the footing. These tests are conducted on composite soft clay bed with single, 2x2 group and 3x3 group C-FL, GP-FL, and FGP-FL columns. The plots also include the curve for the control load test conducted on an unimproved soft clay bed for comparison. The ultimate bearing capacity is obtained as the ratio of ultimate load to area of the footing. The ultimate load is defined as the point at which the slope of the load-settlement curve first reaches zero or a steady minimum value (Vesic, 1973). Figures 4.63 to 4.65 display a stress–settlement curve which resembles the one observed in punching or block shear failure, as mentioned by Vesic in 1973. This indicates that the composite soft ground with floating columns had undergone a punching or block shear failure in the present study.

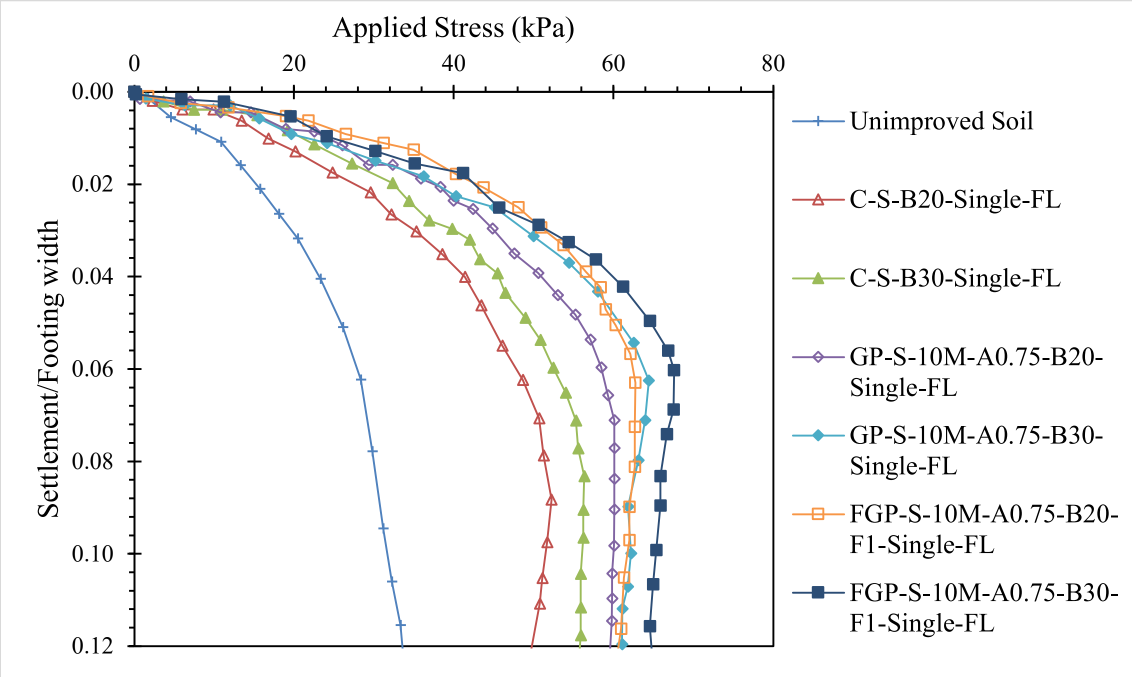



**Fig. 4.63. Applied Pressure vs Settlement/Footing width curve for soft clay bed with single column – Floating condition**

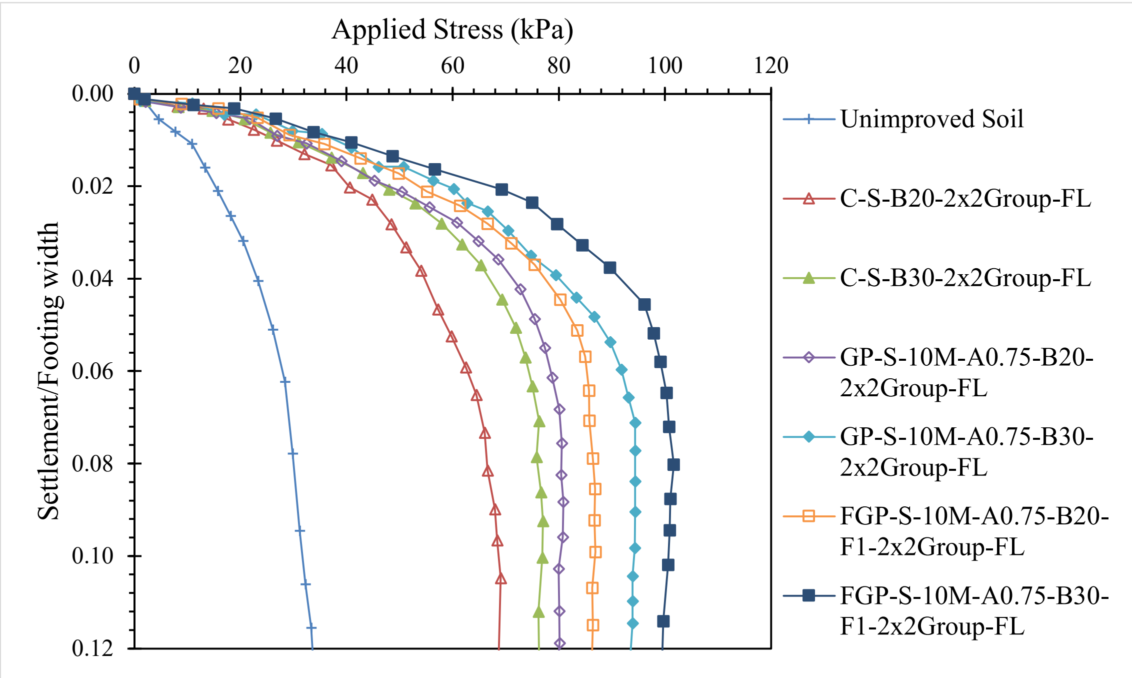



**Fig. 4.64. Applied Pressure vs Settlement/Footing width curve for soft clay bed with 2x2 group columns – Floating condition**

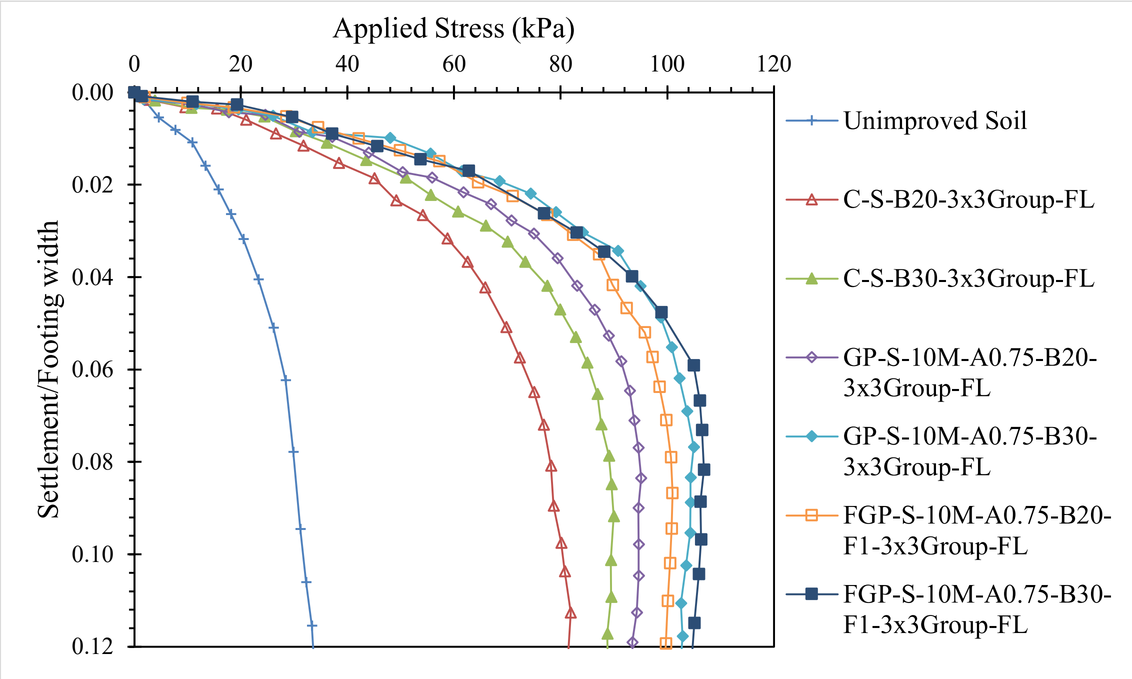



**Fig. 4.65. Applied Pressure vs Settlement/Footing width curves for soft clay bed with 3x3 group columns – Floating condition**

In floating column condition, though the ultimate stress values from the curves seem to have improved the soft clay bed with GP-FL and FGP-FL columns, this enhancement is not significant. This seems to be almost similar for composite ground with any type of columns in floating column condition. In general, the use of floating columns to improve large deposits of soft clay results in increased stiffness and failure stress of the ground as compared to the unimproved soft clay ground. This is possibly due to the load transfer from the footing to deeper depths of the soft soil in case of floating columns condition, where the clay has a higher overburden stress and hence a higher undrained shear strength. Thus, improvement of soft clay ground with floating columns leads to improved load distribution, thereby enhancing its performance. The stabilized soil columns act like shear pins below the footings. However, in the present study, as the tests are performed in small scale test tanks, the effect of overburden on the soil under the columns is negligible and hence its undrained shear strength is same as that in the top layers. Thus, the resistance to load offered by the soil under the columns is minimum thereby causing block failure and minimal improvement in the bearing capacity of the composite clay bed. This may also be attributed to the higher $c_{uc}/c_{us}$ values reported in the present study. The $c_{uc}$ varied from 1683 to 2618 kPa (high strength columns) and thus $c_{uc}/c_{us}$ values varied from 84.15 to 131, $c_{us}$ being 20kPa throughout the study. Rashid et al. (2015b) and Dehghanbabadaki et al. (2016) reported local shear failure of the improved ground with lower $c_{uc}/c_{us}$ values (5.5 and 9 respectively). Rashid et al. (2018) reported local shear failure of the improved ground along with bending of columns with $c_{uc}/c_{us}$ = 19. Mohanty et al. (2021) reported punching shear failure of the improved ground with slight outward displacement of the columns for $c_{uc}/c_{us}$ values ranging from 48 to 55. Thus, the higher $c_{uc}/c_{us}$ values in the present study represent punching or block shear failure of the improved ground, and the higher column strength and fiber reinforcement seems to have minimal effect on the improvement of bearing capacity of the small-scale composite soft clay bed in floating columns condition.

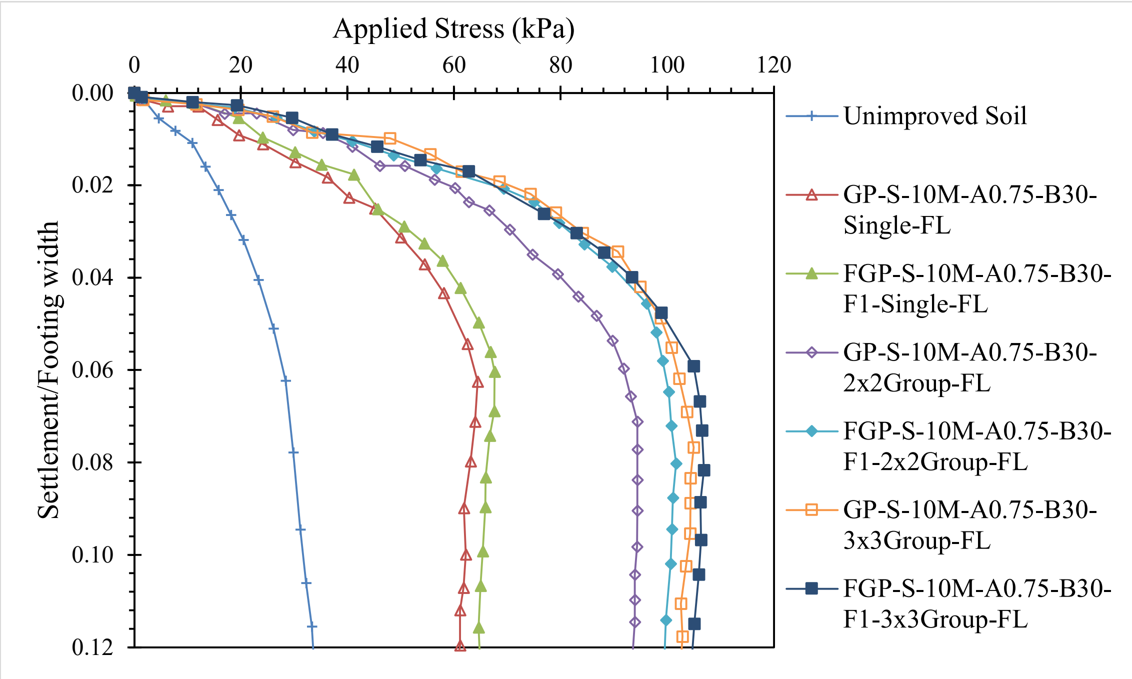



**Fig. 4.66. Applied Pressure vs Settlement/Footing width curves for soft clay bed with single, 2x2 and 3x3 group columns – Effect of Area ratio – Floating condition**

Figure 4.66 represents the effect of $A_r$ on the failure stress and settlement of the composite soft clay bed with floating columns. As the $A_r$ increases, both the stiffness (reduced strains) and failure stress of the ground improve as depicted in Figure 4.66. This may be because the composite ground contains a greater amount of stabilized material with an increase in the $A_r$ from 1.23% to 11.05%.

Figure 4.67 illustrates the variation of experimental values of the ultimate bearing capacity of composite soft clay bed with improvement area ratio, binder content and binder type in floating column condition. It can be noted that the composite soft clay bed with FGP-FL columns arranged in 3x3 group pattern ($A_r$ = 11.05%) at binder content of 30% exhibited higher ultimate bearing capacity (106.85 kPa), followed by the composite soft clay bed with GP-FL columns (104.94 kPa) having similar variables.

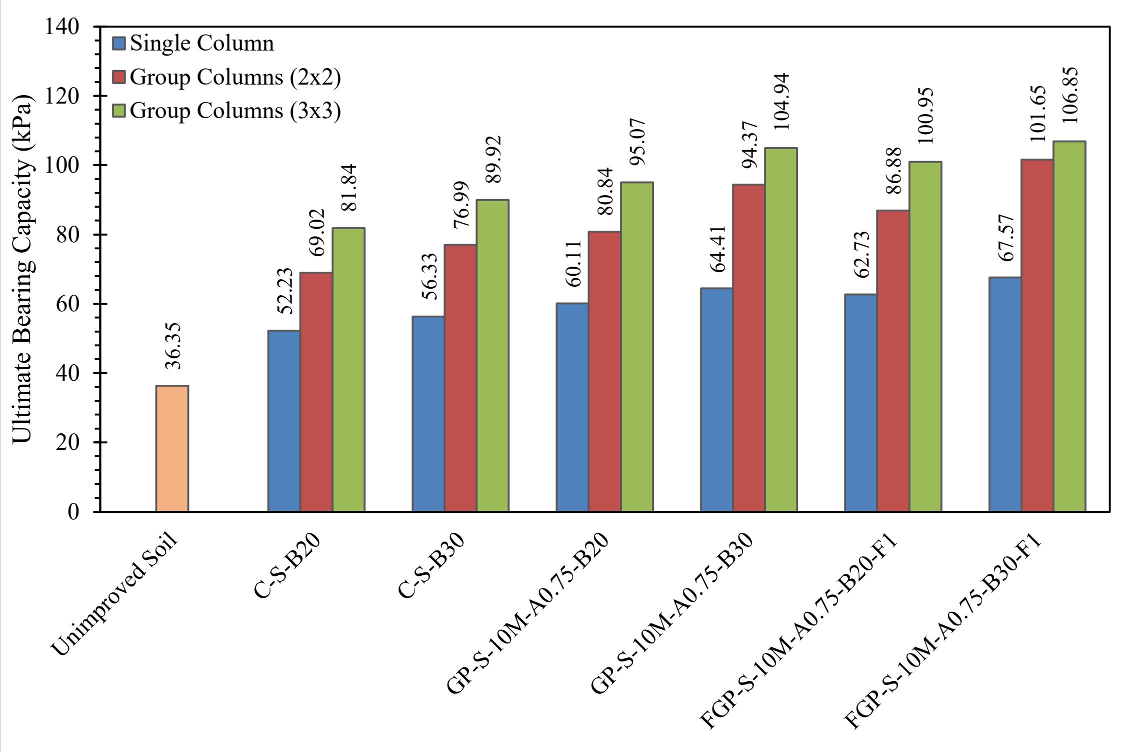



**Fig. 4.67. Ultimate bearing capacity of soft clay bed with C, GP and FGP columns – Floating condition**

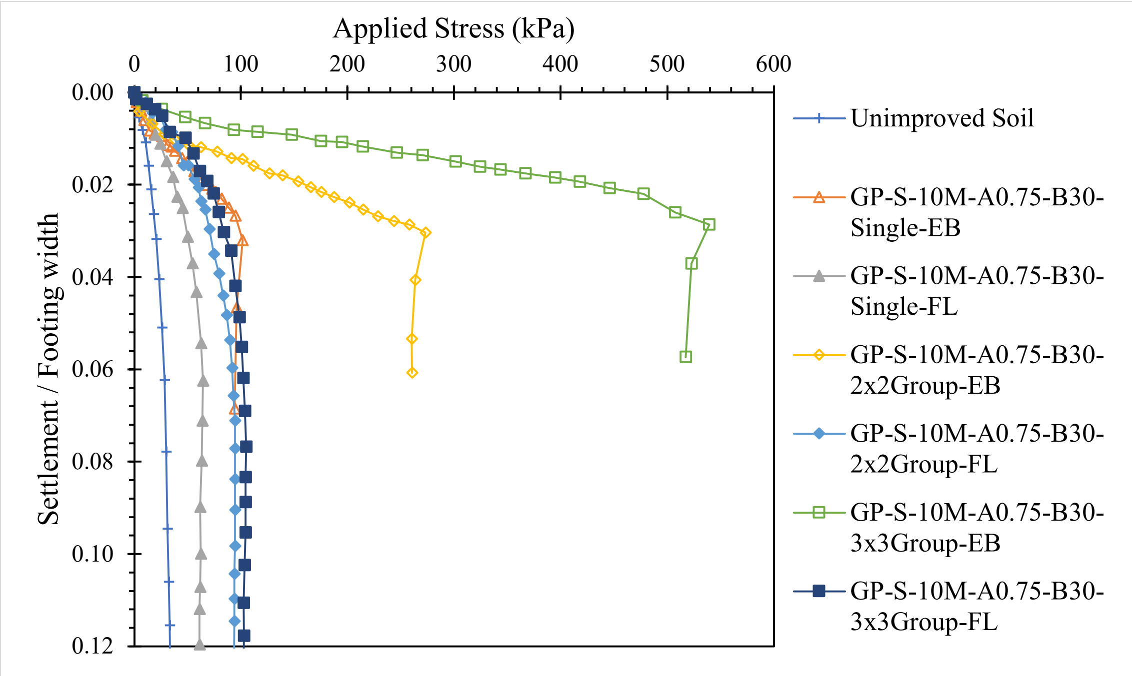



**Fig. 4.68. Applied Pressure vs Settlement/Footing width curves for soft clay bed with GP columns – End bearing and floating column condition**

The applied stress versus normalized settlement curves for composite soft clay bed with GP columns in both end bearing and floating column conditions are presented in Figure 4.68 for comparison. It demonstrates that the composite ground with end bearing columns exhibits the highest level of stiffness and ultimate failure stress as compared to that with floating columns. This can be attributed to the increased rigidity and resistance to failure of the stabilized soil columns within the soft clay bed under the end bearing condition. However, as the behaviour of composite ground with floating columns is affected by the $c_{uc}/c_{us}$ values and as the change in $c_{us}$ values with depth differ in laboratory and field conditions, further studies on this aspect are required and the bearing capacity values obtained in floating column condition may be considered only for the comparison between the use of soil-cement columns, soil-geopolymer columns and fiber reinforced soil-geopolymer columns.

# Chapter – 5
# Conclusions and Recommendations for future research

## 5.1. Conclusions

The efficacy of ground granulated blast furnace slag (GGBS) when used as binder activated with Sodium Hydroxide ($NaOH$) at various contents and concentrations reinforced with polypropylene (PP) fibers for stabilization of soft clay using DSM method was studied. The effect of several variables on the strength and durability of the synthesized geopolymer binder was investigated through standard experimental laboratory tests. The ultimate bearing capacity of the model soft clay ground stabilized with unreinforced and fiber reinforced soil-geopolymer columns geopolymer was also investigated using model tank tests. This study indicates that GGBS and $NaOH$ based geopolymers, enhanced the strength, stiffness and durability characteristics of the treated soft clay significantly. Slag-geopolymer stabilization increased the compressive and flexural strength of soft clay notably. The strength and durability performance of geopolymer stabilized soft clay satisfied the minimum target requirements of DSM technique under various conditions such as variation of soil water content in field and w-d cycles. Also, as a new sustainable binder, GGBS and $NaOH$ based geopolymer was more effective than cement, for stabilization of soft clay in terms of mechanical and environmental properties. Being produced from stockpiled industrial by-products, the GGBS based geopolymers have a lower carbon footprint compared to the conventional binders, thus are a practical and sustainable solution for future ground treatment projects using DSM on soft soil deposits. The following are the general conclusions from the present study.

1. The study aimed to find an optimal design of geopolymer mix that satisfied the minimum requirements set for deep mixed soil-binder columns. The geopolymer mixes with various combinations of GGBS and NaOH were added to the soft clay in order to investigate their effect on its engineering characteristics, varying the GGBS content, water content, molarity of NaOH, A/B ratio and curing time. The soft clay treated with varying dosages of cement was used for comparison. The variations in strength improvement were assessed using unconfined compression strength (UCS) tests and flexural strength tests, that were substantiated with brief microstructural analysis using scanning electron microscopy (SEM) and energy-dispersive X-ray spectroscopy (EDS) tests. The results showed that,

under unconfined compression, the specimens treated with geopolymer showed higher UCS compared to cement-treated specimens of the same dosage, and this may be due to the combined effect of pozzolanic and geopolymeric reactions of geopolymer. Also, binder dosage greater than or equal to 20%, A/B ratio greater than or equal to 0.75 and NaOH molarity of 8M to 10M were required to meet the specified target strength requirements for DSM applications. With increase in initial soil moisture content (higher than liquid limit), the strength of the treated specimens under unconfined compression and flexure is reduced and hence, increased binder dosage helps to meet the DSM requirements for high water content soils.

2. The effect of polypropylene fiber reinforcement on the engineering properties of the geopolymer treated soft clay was also studied at varying fiber contents. The fiber reinforcement has improved the strength of geopolymer treated soft clay mixes also reducing its stiffness making them less brittle. This may be attributed to the homogeneous distribution of fibers in the stabilized soil matrix that arrested crack formation by bridging cracks and enhancing the strength of the composite material. At the micro-level, fibers interact with the soil-geopolymer matrix through mechanical interlocking and chemical bonding that distribute stress more evenly, inhibit crack initiation and propagation, and improve the ductility of the material making the mix specimens more resistant to cracking under various loading conditions. The strength was observed to be maximum at 0.5% fiber content. However, the fiber reinforced soil-geopolymer specimens exhibited maximum reduction in stiffness at a fiber content of 1%.
3. Strong correlations between UCS with curing time, stiffness and flexural strength were obtained. Also, correlations for UCS and flexural strength were obtained between unreinforced and fiber reinforced soil-geopolymer specimens which are valid for fiber contents upto 1%. These could be helpful for engineers and researchers in prediction of strength development, designs and numerical modelling.
4. The durability characteristics like mass loss, volume change and residual UCS of unreinforced and fiber reinforced soil-geopolymer specimens subjected to twelve wetting-drying (w-d) cycles after 28 days curing period were evaluated for the selected mixes. The results indicated that all the specimens survived throughout the twelve wetting-drying cycles and have satisfied the durability requirements of lower mass loss, volume change < 10% and residual UCS in the range of the target UCS (1.034 to 6 MPa) and hence are

durable against wetting and drying. The soil-geopolymer mix with 30% binder content and A/B of 0.75 has shown better durability performance compared to the other mixes. With fiber reinforcement, the durability performance was better for the above mix with 1% polypropylene fiber content. Thus, the geopolymer treated soil mixes with binder content of 30% and A/B ratio of 0.75 reinforced with 1% polypropylene (PP) fibers by dry weight of soil could satisfy the strength and durability requirements and thus found to be the optimum mix combination for deep soil mixing applications for soils with liquid limit of around 68% and plasticity index of around 45%.

5. Model tank tests were performed to evaluate the bearing capacity of the unreinforced and fiber reinforced soil-geopolymer column stabilized model soft clay bed under uniaxial loading. The effect of parameters such as binder type, binder content, fiber inclusion, number of columns (area ratio) and column type or column end condition on the ultimate bearing capacity of the improved soft clay bed was investigated. The composite ground with end bearing columns exhibited the highest level of stiffness and ultimate failure stress as compared to that with floating columns. This can be attributed to the increased rigidity and resistance to failure of the stabilized soil columns within the soft clay bed under the end bearing condition. In end bearing columns condition, the composite model soft clay bed with fiber reinforced soil-geopolymer columns has shown improved load-deformation behaviour and maximum ultimate bearing capacity as compared to that with unreinforced soil-geopolymer columns irrespective of the area ratio and binder content. In floating columns condition, the composite model soft clay bed with columns of any binder and any area ratio has shown a block failure pattern. Hence, the effect of high column strength and incorporation of fibers has insignificant effect on the bearing capacity of the composite model soft clay bed with floating columns of high strength.
6. As per the cost comparison done in the present work as per standard rates of the materials in the market, to attain a particular UCS, the Geopolymer stabilization with GGBS and $NaOH$ has proven to be more economical than the Cement stabilization.

## 5.2. Recommendations for future research

Although several studies have been done on the inclusion of soil-cement columns in soft soil, there is still some gap to gain a better understanding of the applicability of the technique. There is a lot of scope for further research. Some of the probable research areas are listed below.

1. The mineralogical and microstructural studies can further be carried out in detail for geopolymer treated soft clay along with triaxial tests, long term durability and leachate studies.
2. The behaviour of geopolymer treated soft clay reinforced with other low modulus and natural fibers can be explored.
3. The effect of column length to diameter ratio and column spacings on the bearing capacity of the soil-binder column reinforced soft clay bed can be further studied in detail.
4. Life Cycle Assessment and cost-benefit analysis for field application can be studied.

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# List of Publications

## International Journals

1. **Bhavita Chowdary,** V. Ramanamurty and Rakesh J. Pillai, *"Efficacy of slag based geopolymer binder reinforced with polypropylene fibers in the stabilization of soft clays"* Transportation Infrastructure Geotechnology (2022). (**Scopus and ESCI Indexed)** https://doi.org/10.1007/s40515-022-00256-0

2. **Bhavita Chowdary,** V. Ramanamurty and Rakesh J. Pillai, *"Experimental evaluation of strength and durability characteristics of geopolymer stabilised soft soil for deep mixing applications"* Innovative Infrastructure Solutions 6, no. 1 (2021): 1-10. (**Scopus Indexed)** https://doi.org/10.1007/s41062-020-00407-7

3. **Bhavita Chowdary,** V. Ramanamurty and Rakesh J. Pillai, *"Fiber reinforced geopolymer treated soft clay – An innovative and sustainable alternative for soil stabilization"* Materials Today: Proceedings 32 (2020): 777-781. (**Scopus Indexed)** https://doi.org/10.1016/j.matpr.2020.03.574

4. **Bhavita Chowdary,** V. Ramanamurty and Rakesh J. Pillai, *"Bearing capacity of composite soft ground with fiber reinforced soil-geopolymer columns – a model study"* Journal of Materials in Civil Engineering (**SCI Indexed) (Submitted)**

## Book Chapter

1. **Bhavita Chowdary,** V. Ramanamurty and Rakesh J. Pillai, *"Geopolymer stabilization of soft clays – an emerging technique"* Proceedings of the Indian Geotechnical Conference 2019, Lecture Notes in Civil Engineering 136, Springer Nature Singapore (2021) pp. 451 – 458 (**Scopus Indexed)** https://doi.org/10.1007/978-981-33-6444-8_40

## International Conference

1. **Bhavita Chowdary,** V. Ramanamurty and Rakesh J. Pillai, "*Fiber reinforced geopolymer treated soft clay – An innovative and sustainable alternative for soil stabilization*". 3rd International Conference on Innovative Technologies for Clean and Sustainable Development (ITCSD2020), 19–21 February 2020, NITTTR Chandigarh (Presented and published).

## National Conference

1. **Bhavita Chowdary,** V. Ramanamurty and Rakesh J. Pillai, "*Geopolymer stabilization of soft clays – an emerging technique*", Indian Geotechnical Conference on Geotechnics for Infrastructure and Development and Urbanisation (IGC 2019, GeoINDUS), 19–21 December 2019, NIT Surat, Gujarat (Presented and published).