

CHAPTER 3

MATERIALS AND METHODOLOGY

3.1 General

This chapter comprises the comprehensive research testing program and methodology with the aim to utilize industrial waste material-based geopolymer as an effective environmentally friendly binder to traditional binders for the stabilization of soft soil using a deep soil mixing technique. Various geotechnical as well as physiochemical tests are necessary to be conducted to assess the efficacy of geopolymer as a binder for the DSM technique. This chapter includes the details of the raw materials utilized, the procedure for preparing the geopolymer admixture, the techniques used to prepare the treated soil specimens, and the details of the experimental program with the types of equipment used.

Only a small number of full-scale field tests have been documented in the literature, and they are quite expensive. Reduced size physical model testing is the recommended method in light of this to the advancement of geotechnical understanding. The outcomes of the model tests are helpful for both validating analytical and numerical results and comprehending the behavior of the prototype ground.

Moving vehicles cause cyclic loading on embankments that support roads, highways, trains, and other transportation infrastructure. The repetitive vehicle movement produces cyclic stress and strain variation in the subgrade, which can lead to fatigue and deformation over time. A set of physical axisymmetric model tests have been conducted on single geopolymer stabilized soil columns (GPSCs) in very soft soil under static and cyclic loading conditions. Moreover, the performance of an

embankment supported on groups of end-bearing and floating geopolymer stabilized soil columns (GPSCs) in very soft clay under traffic loading conditions was investigated using small-scale physical model tests. This chapter includes the details of the test setup, model preparation, column installation, instrumentation, and loading procedure in the single and embankment model tests.

3.2 Test materials

3.2.1 Kaolin Clay

The soil used in this study was commercial kaolin clay, characterized as soft clay having low swelling and shrinkage capacity and low cation exchange capacity (3–15 meq/100 g of dry soil) (Ghosh and Bhattacharyya 2002). The grain size distribution of kaolin clay obtained using a hydrometer test is shown in Fig. 3.1. The physical properties of kaolin clay, presented in Table 3.1, were determined using several laboratory tests.

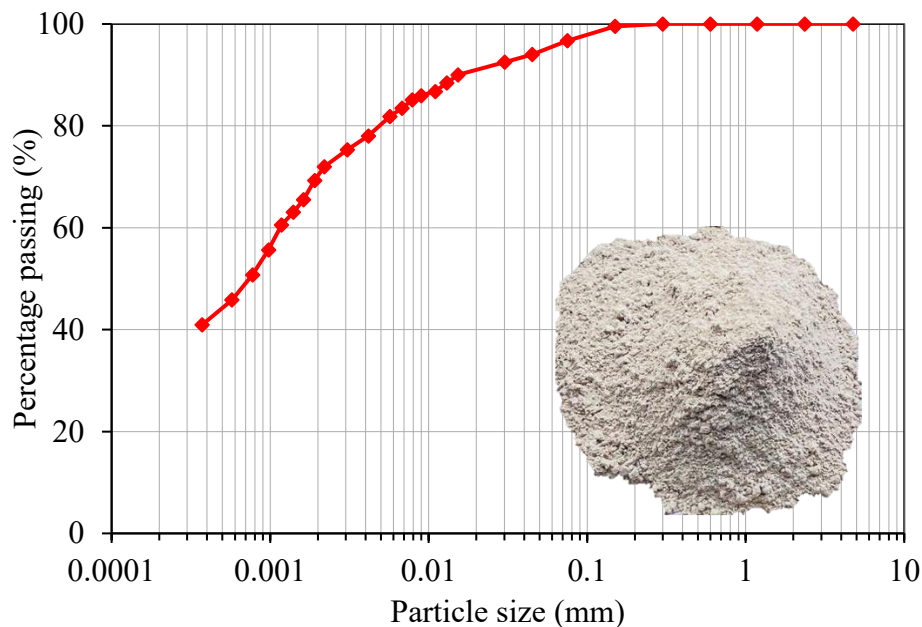
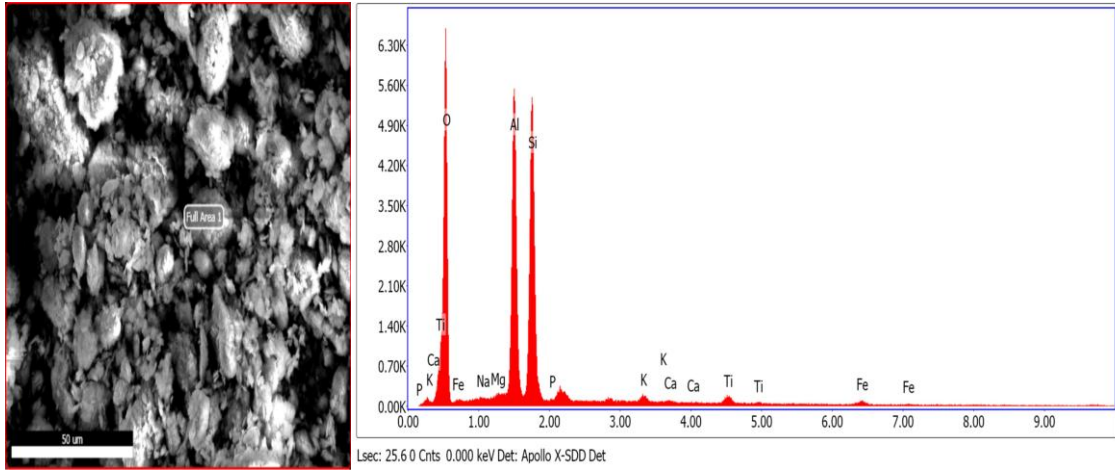


Fig. 3.1 Grain size distribution of kaolin clay.

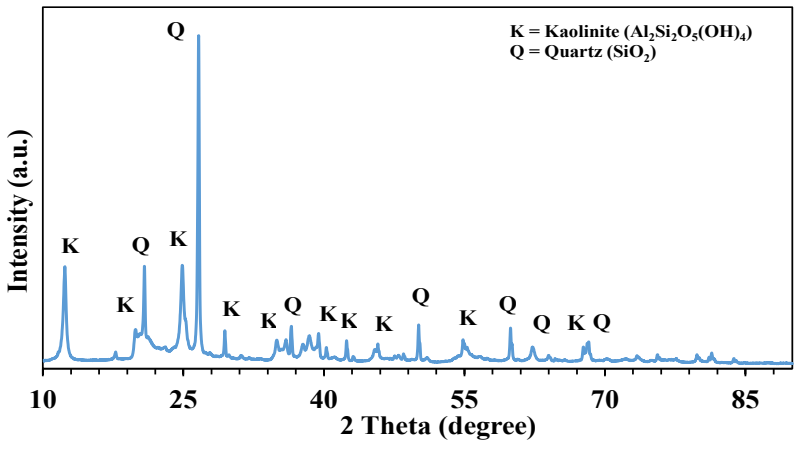
Table 3.1 Properties of kaolin clay.

| Parameters | Value |
|---|--------------|
| Liquid limit, w_l (%) | 41.2 |
| Plastic limit, w_p (%) | 22 |
| Plasticity index, I_p (%) | 19.2 |
| Specific gravity, G_s | 2.6 |
| Sand (%) | 3.3 |
| Silt (%) | 24.7 |
| Clay (%) | 72 |
| pH value | 7.51 |
| Maximum dry unit weight (kN/m^3) | 16.08 |
| Optimum moisture content (%) | 19.3 |
| Soil classification | CL |

The major elements of the kaolin clay included 22.27% Si and 19.54% Al obtained from SEM with an energy dispersive X-ray spectroscopy (EDS) test, as shown in Fig. 3.2(a). Fig. 3.2(b) shows that the mineral phases identified in kaolin clay using the XRD test include kaolinite and quartz.



(a)



(b)

Fig. 3.2 (a) SEM images with EDS test results of kaolin clay and (b) XRD pattern of kaolin clay.

3.2.2 Geopolymer Materials

Ground granulated blast furnace slag (S) and dolomite (D) used in the study were procured from Tata Steel Limited, Jamshedpur, Jharkhand, India. S is the by-product of the iron and steel-making process, obtained by quenching molten iron slag from a blast furnace in water or steam to produce a granular product that is then dried and ground into a fine powder. Dolomite stones are used as a flux in the steel-making process, and while crushing the dolomite stones, fine particles of about 20–40% by weight of

dolomite stones are produced. This dolomite powder is disposed of as waste material. The procured S and D had particle sizes passing through 4.75 mm. They were grounded and sieved through a 425 μm sieve to obtain a uniform powder to maximize the reactivity. Sieve analysis and hydrometer test were performed on the materials, and the grain size distribution is shown in Fig. 3.3. The engineering properties of GGBS and dolomite are listed in Table 3.2.

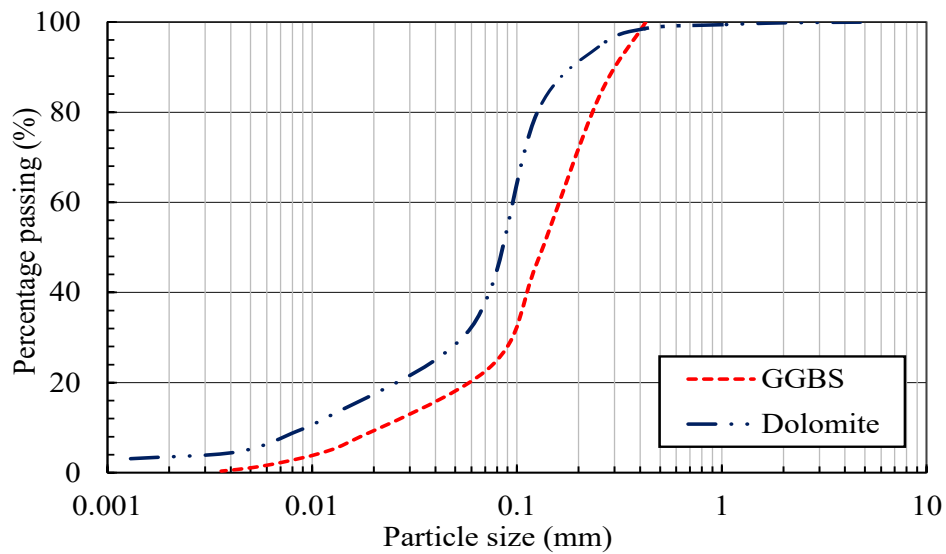


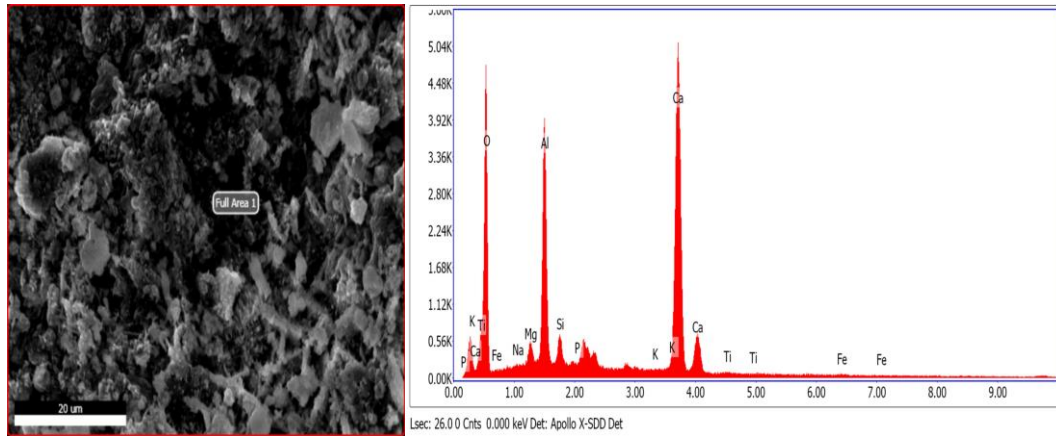
Fig. 3.3 Grain size distribution of GGBS and dolomite.

Fig. 3.4 presents the SEM with EDS results of the GGBS and D. The EDS results show that the major elements in the GGBS are 54.83% O, 31.43% Ca, and 10.13% Al, and in dolomite are 48.29% O, 27.12% Ca, 9.75% Mg, and 7.48% C. FTIR spectra of GGBS and dolomite in the range of 400–4000 cm^{-1} are shown in Fig. 3.5. The weak vibration corresponding to H-OH stretching is noticed at the band of 3460 cm^{-1} and 3408 cm^{-1} in GGBS and dolomite respectively (Ismail et al. 2014). The peak identified at 1445 cm^{-1} and 1442 cm^{-1} in the FTIR spectrum of GGBS and dolomite, respectively, corresponds to the symmetric stretching of the C-O bond of the carbonate (CO_3^{2-}) group (García Lodeiro et al. 2009). GGBS shows a pattern of asymmetric stretching vibration

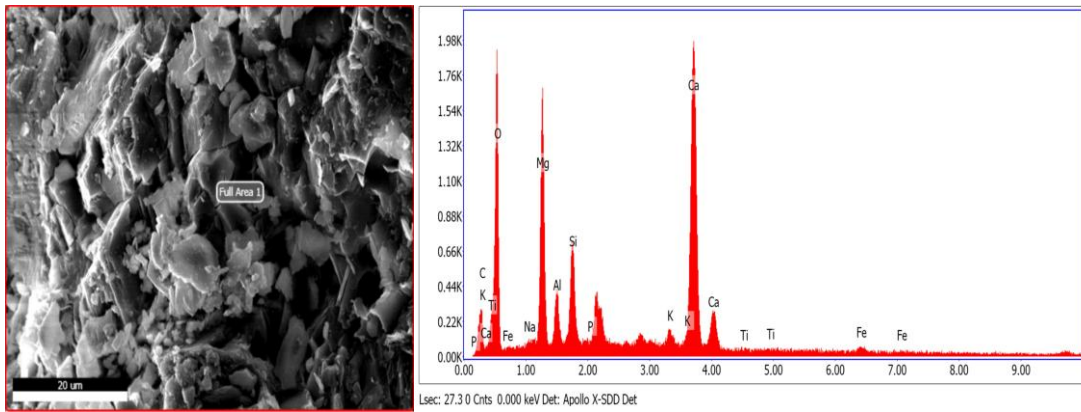
of Si-O-Si, Al-O, Si-OH, and O-Si-O between 800 and 1000 cm^{-1} with a peak at 875 cm^{-1} (Davidovits 2008). The stretching vibration mode observed at 712 cm^{-1} is attributed to the Si-O, indicating the presence of quartz, and the next peak at 516 in the FTIR spectrum of GGBS corresponds to Si-O-Si (García Lodeiro et al. 2009). The peaks observed at 2522 cm^{-1} , 1442 cm^{-1} , 882 cm^{-1} , and 728 cm^{-1} show the stretching bending, asymmetric stretching, out-of-plane bending, and in the plane bending modes of the C-O (carbonate group) in the FTIR spectrum of dolomite. The presence of FTIR absorption band at 3018 cm^{-1} , 2892 cm^{-1} , and 2624 cm^{-1} in the sample indicates the presence of dolomite (Ji et al. 2009).

Table 3.2 Engineering properties of GGBS and Dolomite.

| Geotechnical properties | | Unit | GGBS | Dolomite |
|--------------------------------|----------------------------------|-------------|-------------|-----------------|
| Specific gravity | | | 2.8 | 2.82 |
| Grain size distribution | Medium sand (0.425 mm to 2 mm) | % | - | 1.42 |
| | Fine sand (0.075 mm to 0.425 mm) | % | 76.28 | 57.38 |
| | Silt (0.002 mm to 0.075 mm) | % | 23.72 | 40.03 |
| | Clay (<0.002 mm) | % | - | 1.17 |
| | D_{10} | mm | 0.026 | 0.0087 |
| | D_{30} | mm | 0.088 | 0.05 |
| | D_{60} | mm | 0.17 | 0.1 |
| | c_u | | 6.54 | 2 |
| | c_c | | 1.75 | 2.87 |



(a)



(b)

Fig. 3.4 SEM images with EDS test results of (a) GGBS and (b) Dolomite.

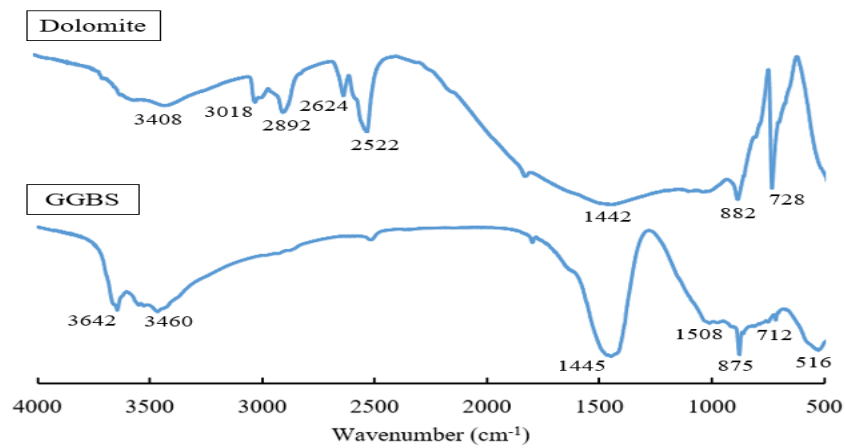


Fig. 3.5 FTIR spectrum of GGBS and dolomite.

3.2.3 Alkali Activators

The alkali activator used in the study was prepared with commercially available sodium hydroxide (NaOH) pellets and sodium silicate (Na_2SiO_3) solution, considered the most suitable liquid alkali activator in the geopolymerization process. NaOH solution of 8M concentration was prepared by dissolving 320 g NaOH pellets in distilled water to prepare 1 lt solution. As NaOH is a strong base, a higher concentration of NaOH was avoided, considering the previous recommendations for harmful health effects, safety, and economic factors (Nematollahi and Sanjayan 2014; Yaghoubi et al. 2018). Due to the occurrence of the exothermic reaction, the alkali activator solution was prepared 24 h before use. Na_2SiO_3 was obtained in solution form [SiO_2 (28.1%), Na_2O (14%), H_2O (57.9%)] with $\text{SiO}_2/\text{Na}_2\text{O}$ equal to 2.

3.2.4 Embankment Soil

Silty sand was used as a fill material for the construction of the embankment, as recommended in RDSO guidelines (RDSO 2020). The grain size distribution curve of silty sand is shown in Fig. 3.6. The properties of the embankment material determined from the laboratory test are presented in Table 3.3.

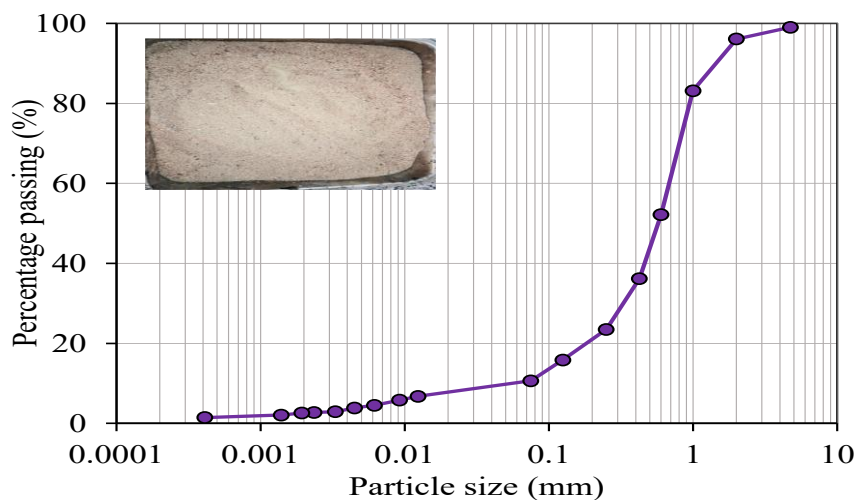


Fig. 3.6 Grain size distribution of embankment soil.

Table 3.3 Properties of embankment soil.

| Parameters | Value |
|---|-------|
| Specific gravity, G_s | 2.65 |
| Maximum dry unit weight (kN/m^3) | 18.22 |
| Optimum moisture content (%) | 8.45 |
| Angle of internal friction, φ (degrees) | 34 |

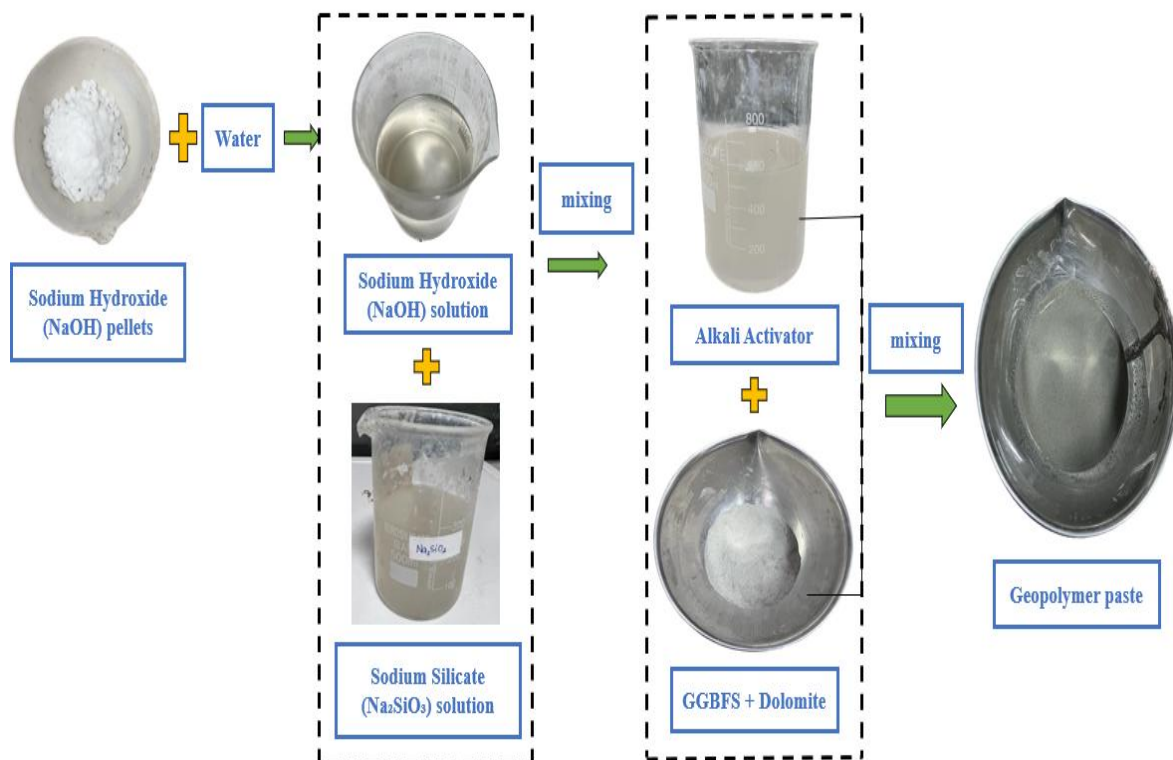
3.3 Specimen preparation

The process of preparing geopolymer paste and kaolin clay-geopolymer paste is shown in Fig. 3.7(a) and 3.7(b). First, the geopolymer paste is prepared by mixing liquid alkali activator (L) and precursor (P) for 2–3 min. Then, the required water is added to oven-dried kaolin clay to prepare clay paste with a certain moisture content. Finally, kaolin clay-geopolymer paste is prepared by mixing geopolymer paste with kaolin clay paste for an additional 5 min in a mixer. This paste is then poured into the cylindrical molds of a height: diameter ratio of 2:1 in 3 layers. The bottoms of the molds were sealed with thick, sticky plastic before pouring the paste. For each layer, tapping of molds was done 25 times to release the entrapped air from within the paste. The molds were sealed immediately and left to cure at room temperature for 24 h. Then, the samples were demolded from the molds and wrapped in the cling film until curing time (7, 14, and 28 days) in a control room with a temperature of 27 ± 2 °C.

This study adopted various combinations of the precursor materials and alkali activators under curing conditions to analyze the effect of different parameters in the geopolymerization process. In the DSM technique, up to 20–25% binder content was reported to be added with soil depending on the soil type (Abdullah et al. 2020; Ghadir

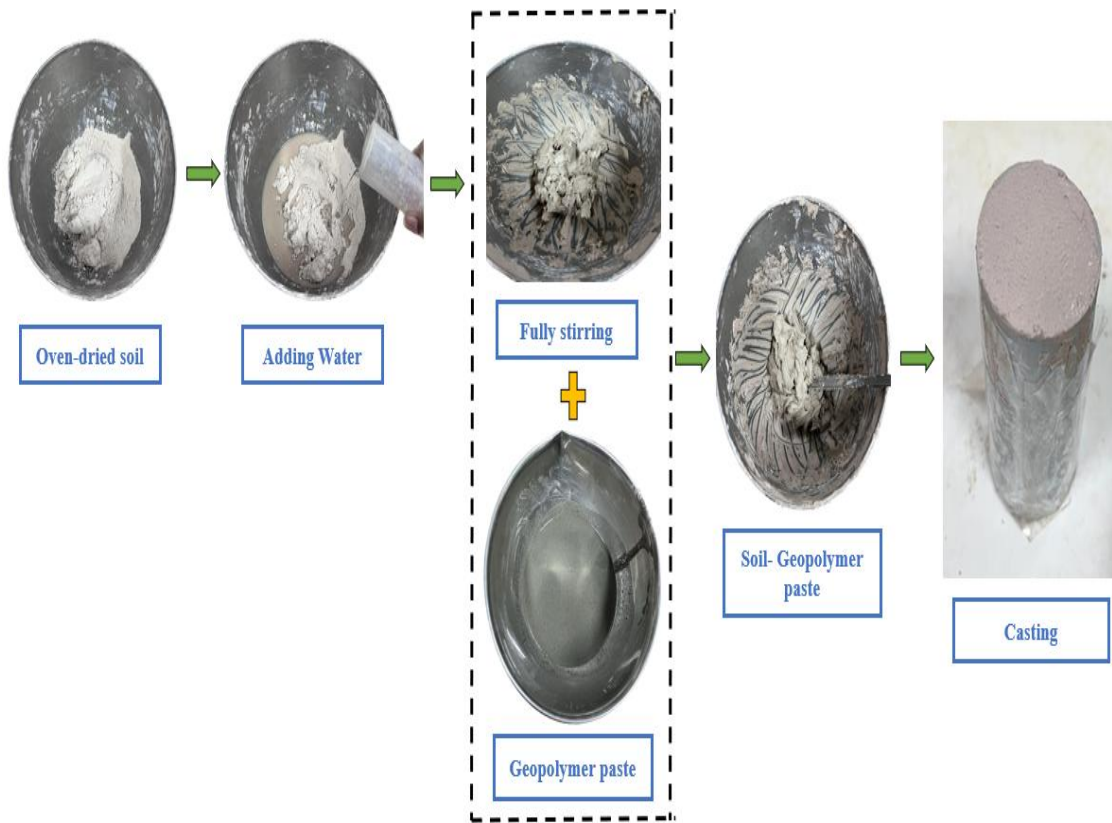
and Ranjbar 2018). Hence, to analyze the effect of precursor content on the stabilization of kaolin clay for the DSM technique, 5%, 10%, and 20% precursor content (by dry mass of clay) were considered in this study. Research has shown that the type of precursor significantly affects the mass ratio of NaOH: Na₂SiO₃ (Ayub and Khan 2023; Li et al. 2021; Zhou et al. 2021). Therefore, to investigate the effect of adding dolomite with GGBS for different ratios of alkali activators at different curing times, NaOH: Na₂SiO₃ (NH: NS) mass ratio (100:0, 75:25, 50:50, 25:75, and 0:100); GGBS: dolomite (S:D) ratio (20:0, 18:2, 16:4, and 14:6) considering precursor content of 20%; and curing time (7, 14, and 28 days) were considered. Furthermore, the effect of moisture content variation of kaolin clay and alkali activator/precursor (L/P) ratio was studied. Specimens stabilized with 5%, 10%, and 20% OPC were also prepared as control specimens for comparison with geopolymer-stabilized soil.

Preparation of geopolymer paste



(a)

Preparation of kaolin clay-geopolymer paste



(b)

Fig. 3.7 Preparation of geopolymer-stabilized clay sample: (a) Geopolymer paste preparation and (b) Clay-geopolymer sample preparation.

3.4 Experimental methods

All the experiments were conducted in the Department of Civil Engineering, IIT BHU, India. The experimental testing regime employed in this research includes unconfined compressive strength (UCS) test, pH test, static triaxial test, and cyclic triaxial test in order to assess the soil improvement and characterize the engineering properties of kaolin clay treated with geopolymer. Moreover, microstructural analysis was conducted using scanning electron microscopy (SEM) test and Fourier transform infrared spectroscopy (FTIR) test. Furthermore, a Toxicity Characteristic Leaching

Procedure (TCLP) test was performed on GGBS, dolomite, and optimum mix specimens to determine the leaching behavior of heavy metals. The following sections include information on sample preparation, testing program requirements, important testing machine characteristics, and testing method specifics.

3.4.1 Unconfined Compressive Strength (UCS) Test

A series of unconfined compressive strength tests were conducted on the geopolymer-stabilized kaolin clay as prepared in section 3.3 and cured for 7, 14, and 28 days. Several influencing parameters were analyzed to select the optimum mix proportions based on strength improvement.

To perform the UCS test, the samples were prepared in cylindrical molds of 38 mm in diameter and 76 mm in height, and the test was performed in accordance with (ASTM D2166M - 16 2016). Three identical samples of each mix for all curing periods were tested at a 1.2 mm/min strain rate. Accordingly, the average UCS value and standard deviation from the three tests were reported. As per the acceptance criterion, the UCS of each specimen could not deviate by more than 10% of the average UCS.

3.4.2 pH Test

A pH test was conducted on a minimum of three samples to determine the ion concentration in the sample after a required curing period using an electronic pH meter, as the minimum required pH for the occurrence of a pozzolanic reaction is 11 (Garcia-Lodeiro et al. 2015). The average of three values was reported as the pH value of the sample. The collected crushed samples passing through a 75 μm sieve from the UCS tests were used for the pH determination in accordance with (ASTM D4972-13 2007). The pH meter had a range of 0–14 and an accuracy of ± 0.01 and can accurately determine the pH value of the sample at any temperature in a range of 0°C – 80°C.

3.4.3 Static Triaxial Test

After evaluating the effect of different parameters from the UCS test, the optimum mix was selected to perform an isotropically consolidated undrained (CU) static triaxial test to analyze the undrained shear response of the treated kaolin clay. Along with stress-strain response, pore water pressure evaluation within the soil mass can also be studied in the CU triaxial test. The tests were conducted per (ASTM 2020) on the 28 days cured samples. The test was conducted under different confining pressures of 100, 200, and 300 kPa to simulate the expected field stresses for soil located at a depth of 5, 10, and 15 m from the ground surface (assuming the saturated unit weight of soil mass of 20 kN/m³). The specimens were firstly saturated by increasing the back pressure until a B value greater than 0.91 was achieved. A B-value of 0.91 indicates 99% saturation in cemented soil specimens (Abdullah et al. 2019b; Head and Epps 2015). The specimens were then isotropically consolidated at the effective confining pressure, and then shearing was done at the rate of 0.01667 mm/min.

3.4.4 Cyclic Triaxial Test

To carry out the cyclic triaxial test, the samples were prepared in cylindrical molds of 50 mm in diameter and 100 mm in height, and the test was performed in accordance with (ASTM D5311 2011) and (ASTM D3999 2011) standards. The samples were cured for 28 days as strength development is time-dependent for geopolymer-stabilized kaolin clay. The consolidated undrained cyclic triaxial tests were performed under stress-controlled conditions as the earthquake, wave loads, and varied vehicle loads are better simulated in stress-controlled conditions (Fig. 3.8).

Confining pressures of 50, 100, and 200 kPa were considered as these pressures are within the normal stress range for pavement subgrade (Lang et al. 2020; Mohtar et al. 2013)(Lang et al. 2020; El Mohtar et al. 2013). Cyclic Stress Ratio (CSR) of 0.5,

0.75, 1.0, and 2.0 corresponding to cyclic stress of 100, 200, 300, and 400 kPa, respectively, were taken considering the range of traffic load in the field (Leng et al. 2017). Furthermore, loading frequencies of 0.5, 1, 1.5, and 2 were selected considering the varied traffic loading conditions with different speeds (Min et al. 2022; Sun et al. 1988; Tang et al. 2016)

Cyclic loading of sinusoidal wave type was applied during the tests, and cyclic stress ratio (CSR) was calculated as follows (ASTM D5311 2011):

$$CSR = \frac{\Delta q_{cyc}}{2\sigma'_c} \quad (3.1)$$

Where Δq_{cyc} is the variable deviatoric stress and σ'_c is the effective confining stress.



(a)

(b)

Fig. 3.8 (a) Cyclic triaxial test apparatus and (b) Geopolymer stabilized specimen with triaxial cell assembly.

3.4.5 Microstructural Analysis

3.4.5.1 Scanning Electron Microscopy (SEM)

After performing the UCS test, representative crushed samples of geopolymer-treated kaolin clay at 28 days were dried at 60 °C for 24 h. Dried samples were then finely ground and passed through a 75 µm sieve for microstructure investigation using scanning electron microscope (SEM) imaging and energy dispersive X-ray spectroscopy (EDS) tests on the sample taken from the center. The EVO 18, Zeiss 4.0 high-resolution SEM detector was employed to conduct the microstructure analysis.

3.4.5.2 Fourier Transform Infrared Spectroscopy (FTIR)

The Fourier transform infrared spectroscopy spectra of the geopolymer stabilized clay were scanned using the Nicolet Is5 (Thermo Electron Scientific Instruments LLC) instrument. The KBr pellet method was used, in which 1 g of fine powder of the specimen was mixed with 100 mg of potassium bromide (KBr) powder. The mix was pressed for 5 min to form a pellet of 1 cm diameter and 1–2 mm thickness. All the spectra were obtained at wavenumber from 400 to 4000 cm^{-1} with 32 scans at a resolution of 4 cm^{-1} as the minerals have characteristic absorption bands in the midrange of the infrared wavenumbers.

3.4.6 Toxicity Characteristic Leaching Procedure (TCLP)

EPA 1311 (USEPA 1992) is one of the primary leaching tests used to determine the level of heavy metal pollution in stabilized soil as it is in direct contact with the subsurface water (Amini and Ghasemi 2019). The TCLP test was performed on GGBS, dolomite, and one selected geopolymer-stabilized specimen with the highest strength.

To reduce the size, specimens were passed through a 425 µm standard sieve. 10 g of that specimen was taken in the polypropylene container. After that, the sample was

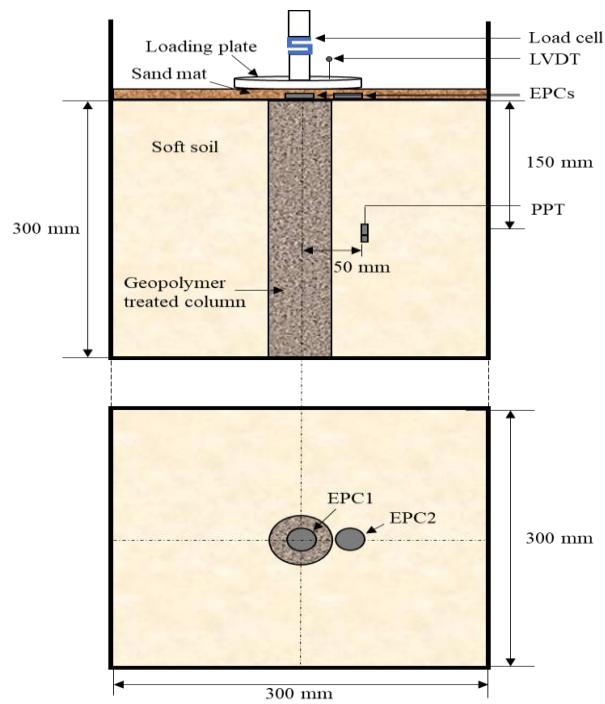
mixed with 200 mL of acetic acid solution with a pH of 2.9 (keeping the liquid/solid ratio of 20 mL/g). The solution was placed in the special machine and agitated at a temperature of 20 °C at 30 rpm velocity for 18 h. The solution was then filtrated through a 0.7 μm glass fiber filter. More acetic acid solution was added to the leachate of the solution so that the $\text{pH} < 2$. Finally, the solution obtained was analyzed for heavy metal leachate concentration by an inductively coupled plasma–atomic emission spectrometry (ICP-AES) machine.

3.5 Single geopolymer column foundation model tests

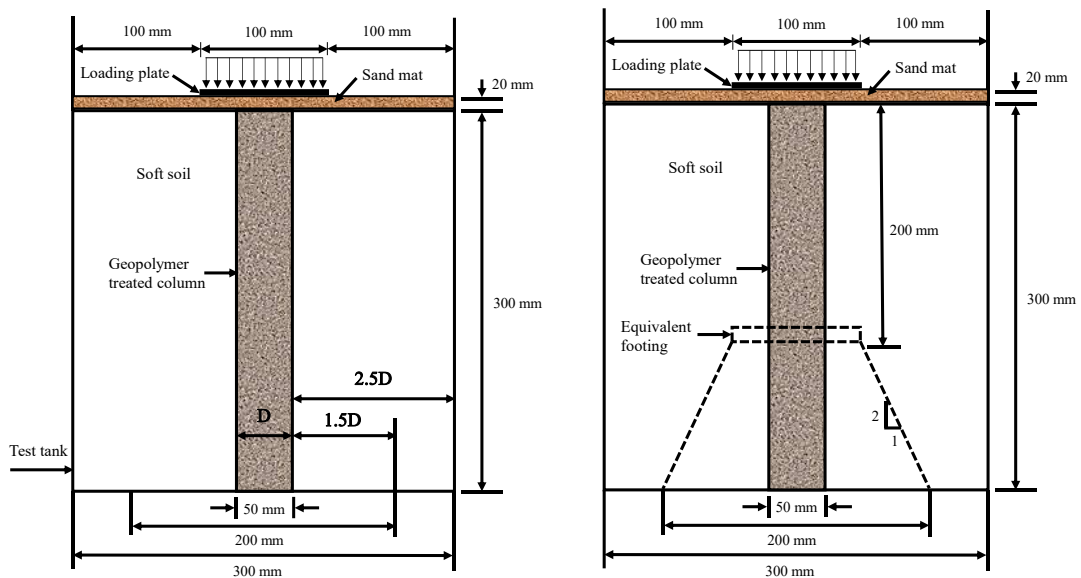
3.5.1 Test Setup

A set of small-scale laboratory model tests were performed in a square tank with inner sides of 300 mm, height of 500 mm, and thickness of 5 mm, as shown in Fig. 3.9(a). To prevent the tank from moving laterally while being loaded, the sides were sealed. The dimensions of the model test tank were considered based on necessary boundary conditions so that no interference would occur between the sides of the tank and the failure zone of the DSM column, as shown in Fig. 3.9(b). To achieve this, (Meyerhof and Sastry 1978) recommended that the failure zone extend over the radial distance of about $1.5d$ from the periphery of the column, where d is the diameter of the column. For the column with a maximum diameter of 60 mm, $1.5d$ is 90 mm from the periphery of the column. Moreover, (Ali et al. 2012; Shahu and Reddy 2011) recommended that the tank boundaries should be chosen such that the induced stresses should become insignificant ($<10\%$) at the tank boundaries. The selected dimensions of the tank ensure that the stresses generated at its borders are not greater than 10% of the loads applied to the footing. The stress distribution was calculated considering an equivalent footing placed at two-thirds depth from the top of the column, as presented

in Fig. 3.9(b). It can be seen that the 2V:1H stress distribution does not reach the tank boundaries, so the stresses at the boundaries become insignificant.



(a)



(b)

Fig. 3.9 (a) Model test setup along with the location of earth pressure sensors and pore pressure transducers and (b) Criteria for selection of tank size.

3.5.2 Similitude Law

In geotechnical engineering, the scaling rules must be appropriately applied in order to create a small-scale physical model in the loading chamber. Table 3.4 provides a summary of the necessary scaling factors for the mechanical and physical characteristics considered in this investigation for 1g physical modeling. The applied stress in the model testing is the same as that in the prototype (scaling factor for stress, $S_{\sigma} = 1$), and the geometry of the model was scaled by a factor of 25 ($S_f = 25$). This strategy was also used in several previous research involving 1g physical modeling tests (Aqoub et al. 2020; Van Eekelen 2015; Xu et al. 2024).

An experimental setup with a model scale of 1:25 was designed to study the behavior of soft soil stabilized with a DSM column. The typical diameter of the DSM column varies in the range of 0.5 to 2.1 m, with center-to-center spacing of 1 - 1.5 m and length of 5 and 30 m, and the area replacement ratio varies from 8 to 50% (Kitazume & Terashi 2013). This model represents a typical unit of GPSC-stabilized ground subjected to vertical loading. The column was installed in the center, having a diameter of 30 to 60 mm and an area replacement ratio of 9-36%, representing a diameter of 0.75-1.5 m in the field. Considering the column length of 7.5 m in the field, the height of the model was designed to be 300 mm, representing a linear scale factor of 25.

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Table 3.4 Similitude law for model tests against prototype.

| Variables | Dimension | Scale ratio |
|------------------|-------------------|--------------------|
| Length | m | 1: S_f |
| Area | m ² | 1: S_f^2 |
| Density | kg/m ³ | 1:1 |
| Stress | kPa | 1:1 |
| Force | kN | 1: S_f^2 |
| Strain | m/m | 1:1 |

3.5.3 Model Preparation and Column Installation

Deep soil mixing is an effective ground improvement technique to improve the strength of soft soils with undrained shear strength, c_{us} , of less than 25 kPa. Considering this, the model tests were performed on a soft soil bed with undrained shear strength of 5-5.5 kPa. Several trails of vane shear tests were conducted to determine the water content required to prepare a soft soil bed at targeted shear strength. Approximately $38\% \pm 1\%$ water content was added to oven-dried dry kaolin clay to achieve the target shear strength. A uniform soil paste was prepared using a mechanical mixer. In order to achieve uniform and consistent moisture equalization, the prepared paste was placed to rest for 24 h. It was covered with a damp jute bag to prevent moisture loss. Before filling the tank with soft soil, the inner sides were coated with silicon grease and covered with a polythene sheet to minimize side-wall friction. The paste was placed in 10 layers in the tank, with a thickness of 30 mm for each layer, thoroughly kneaded by hand to

remove any entrapped air voids. The tank was then covered with a moist cloth to avoid any moisture loss and left undisturbed for two days to regain the thixotropic strength.

The procedure to prepare and install DSM columns was considered from the previously reported research by (Bouassida and Porbaha 2005; Fang and Yin 2007; Horpibulsuk et al. 2012; Said et al. 2019). DSM columns were prepared with 20% geopolymer binder. First, the oven-dried kaolin clay was mixed with the required amount of water with the required undrained shear strength of 5 kPa. Then, the required geopolymer paste was prepared by mixing the liquid alkali activator and precursor for a few minutes. Finally, kaolin clay-geopolymer paste was prepared by mixing kaolin clay paste and geopolymer paste for 5 min. The paste was then poured in layers into the cylindrical PVC pipes of different diameters and lengths as per the required model test conditions, and each layer was tapped 25 times to remove the entrapped air voids. At the same time, the sample for UCS tests was also prepared in the cylindrical mold with a diameter of 38 mm and a height of 76 mm. After 24 h, the samples were extracted and wrapped into the cling film to avoid moisture loss and cured for 21 days.

To produce a model of DSM column-treated ground, the location of the column was marked on the clay bed. A 300 mm x 300 mm wooden sheet was used as a guiding system for the installation of the column. The wooden sheet with a hole of the required diameter at the center, as per the test conditions, was kept on the soil bed to ensure that the PVC pipe and the column were vertically inserted into the soil. A spirit level was used to ensure that the pipe penetrated vertically into the model ground. A thin open-ended PVC pipe of 1 mm thickness and an internal diameter equal to the target diameter column is slowly inserted vertically without disturbing the nearby soil up to the required depth. To minimize the effect of friction between soil and pipe, the inner and outer sides of the pipe were coated with oil. The soil inside the pipe was removed with the help of

an auger, and a cylindrical hole was created. The pipe was pulled out, and the cured geopolymer-treated column was inserted carefully into the hole. The complete process of soil bed formation and column installation is shown in Fig. 3.10. The column was cured inside the soil model for 7 days to obtain homogeneous contact between the soil and the column. Finally, a sand mat with a 20 mm thickness was placed on the top of the model ground to transfer the load from the footing to the DSM column and the surrounding soil.

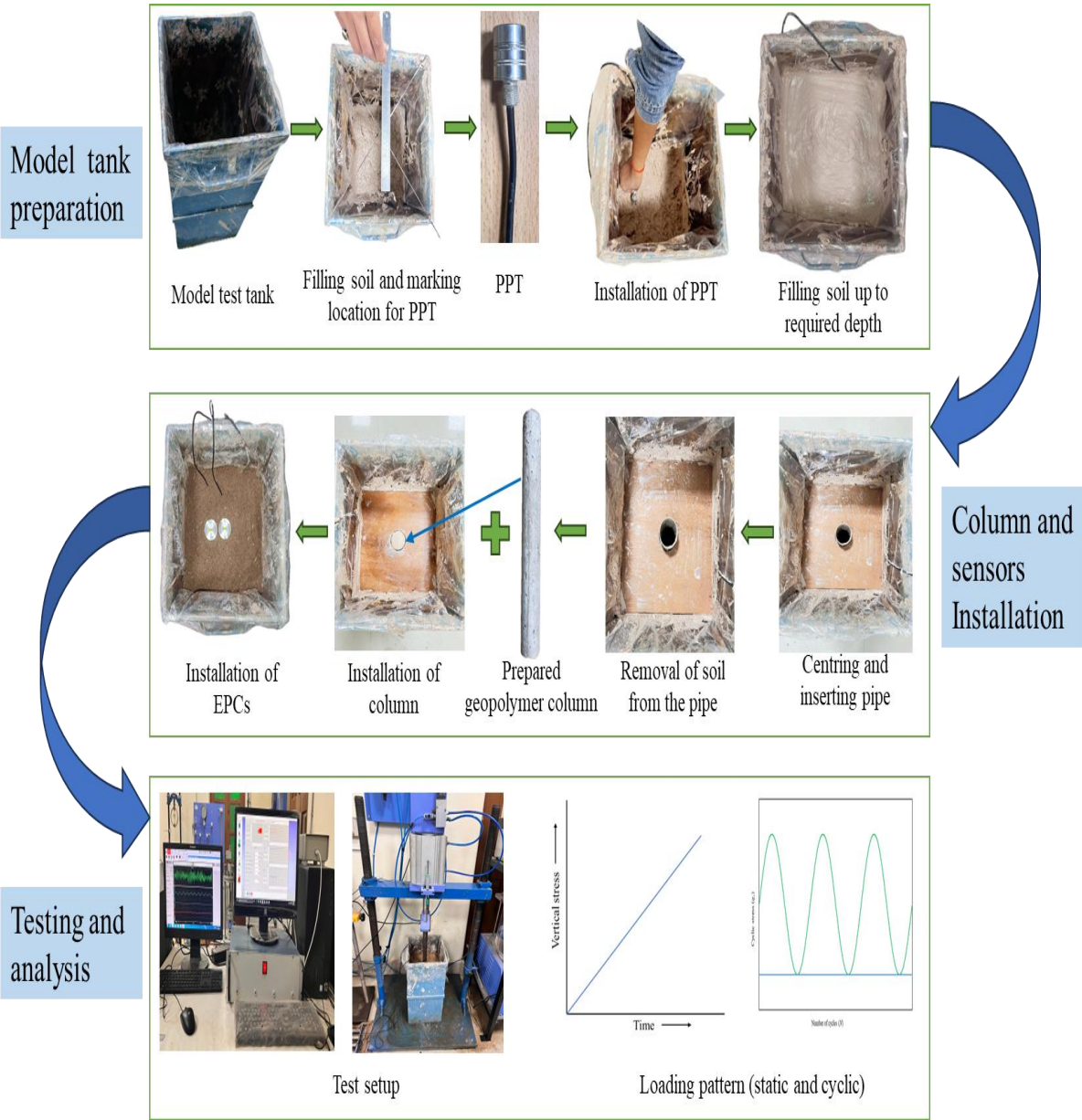


Fig. 3.10 Systematic diagram of the model test tank preparation and GPSC installation.

3.5.4 Instrumentation and Loading Procedure

In this model test, one small-sized pore-pressure transducer with a capacity of 50 kPa and two earth pressure cells with a capacity of 200 kPa and 1 MPa were installed in the tank. The pore pressure transducer was installed to detect the pore water pressure while making the soil bed. The earth pressure cells were installed after column installation. The 200 kPa earth pressure cell was installed above the soft soil bed, and a 1 MPa earth pressure cell was installed above the column to measure the load acting on the column and the surrounding soil. The exact location of these sensors is shown in Fig. 3.9(a). A DEWESOFT universal data gathering system was used to store and retrieve data from PPT and EPCs.

Static and cyclic loads were applied using a multifunctional, servo-controlled pneumatic loading data acquisition device with a 20 kN capacity. Using an LVDT connected to a data logger, the settlement of the footing was measured. The details of all the instruments used in the study are presented in Table 3.5. All equipment, including the load cell, LVDT, and sensors, were calibrated properly before each model test.

Table 3.5 Technical specifications of geotechnical instrumentation.

| Sensor name | Type | Measurement | Capacity |
|--------------------------------|---------------------|------------------------|----------|
| Load cell | S-type | Load | 20kN |
| LVDT | - | Settlement | ±50 mm |
| Earth pressure cell 1 (EP1) | KDE-200 kPa | Dynamic earth pressure | 200 kPa |
| Earth pressure cell 2 (EP2) | KDE-1 MPa | Dynamic earth pressure | 1MPa |
| Pore pressure transducer (PPT) | BPR-A-S, FSO-50 kPa | Porewater pressure | 50 kPa |

The model test setup, along with the pore pressure transducer, earth pressure cell, and data logger, is shown in Fig. 3.11.

3.5.5 Load Application under Static Condition

The model tests subjected to static loading were conducted in strain-controlled mode at a vertical settlement rate of 1 mm/min. The settlement rate was obtained using Eq. 3.2 to simulate the undrained condition during loading (Lehane et al. 2008). The normalized velocity (V) that defines the undrained condition at a particular settlement rate is given as

$$V = \frac{v_f B}{c_v} > 20 \quad (3.2)$$

where v_f , B , and c_v are the foundation velocity, the footing width, and the coefficient of consolidation. Taking the c_v value as 4.4 m²/year for the kaolin clay, the width of the footing as 100 mm, and the foundation velocity of 1 mm/min, the resulting normalized velocity to depict undrained condition is 22.7, which is higher than 20.

The footing was subjected to a maximum settlement of 50 mm due to the limitation of the maximum movement of the actuator's piston.



Fig. 3.11 Model test setup along with data loggers, earth pressure cells, and pore pressure transducer.

3.5.6 Load Application under Cyclic Condition

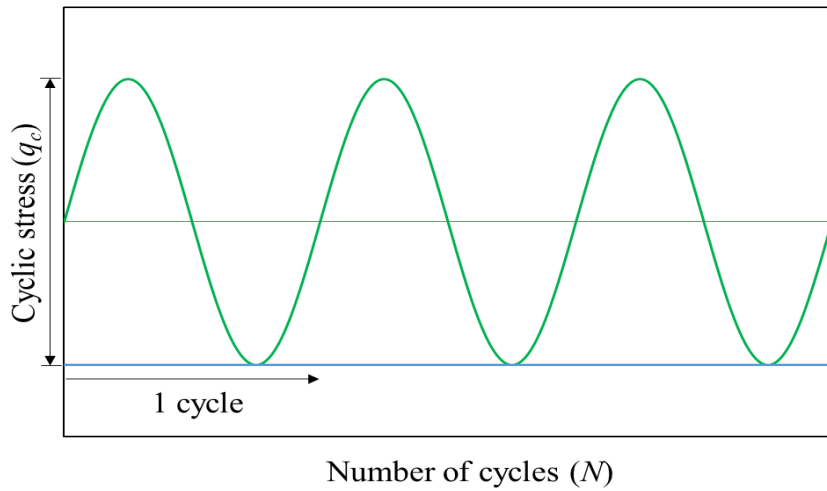
The cyclic loading tests were performed in a load-controlled mode in the one-way sinusoidal cyclic form at a frequency of 1 Hz for 10,000 cycles or until failure, whichever is reached earlier. The typical variation of the results obtained from the stress-controlled cyclic model test is shown in Fig. 3.12. Prior research has reported a slight impact of the frequencies in the 0.1 to 10 Hz range on the accumulated vertical strain pore water pressure (Liu and Xiao 2010; Toyota and Takada 2021). According to previous studies (Yoo and Abbas 2020; Zhang et al. 2020; Zhao et al. 2017), the loading frequency usually does not exceed over 6 Hz, and when the frequency is less than 4, the effect of cyclic loading frequency turns out to be small. Various researchers have used

a loading frequency of 1 Hz to simulate the traffic loading conditions of the field in laboratory tests (Chazallon et al. 2006; Gu et al. 2024; Lei et al. 2023; Wu et al. 2022; Yoo and Abbas 2020). Therefore, a frequency of 1 Hz has been used in the present study. (Lin et al. 2019) and (Kermani et al. 2019) reported that traffic loading magnitude ranges between 10.6–44.5 kPa and 12.5–24 kPa for different vehicles, from cars to fully loaded dump trucks on the roadway under low-volume traffic. (Van Eekelen and Bezuijen 2012) reported that the traffic loads on embankments up to 5 m height can exert pressure up to 101.9 kPa. Also, a normalized parameter, cyclic stress ratio (CSR), represents the application of cyclic loading amplitude on the subgrade and is defined as (Zhang et al. 2020).

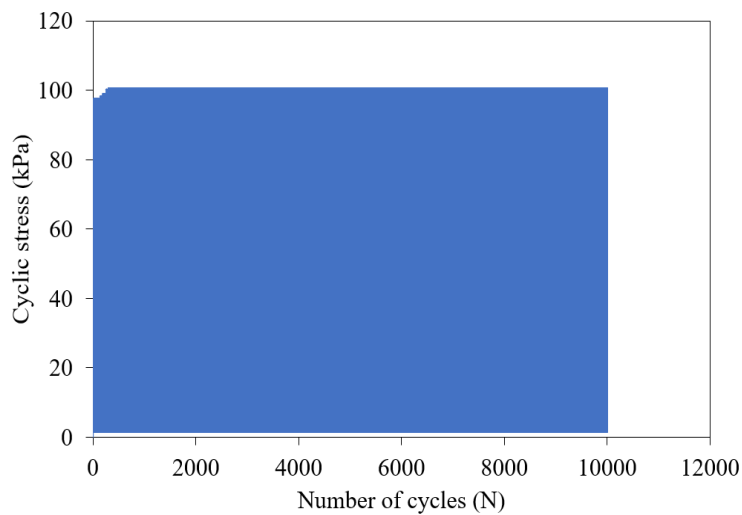
$$CSR = \frac{q_c}{q_s} \quad (3.3)$$

where q_c is the cyclic loading amplitude, and q_s is the ultimate bearing capacity of the column-improved soil under static loading. The ultimate bearing capacity of a GPSC-improved soil with an A_r of 9%, 16%, 25%, and 36% from static loading tests is determined as 62.4 kPa, 101.23 kPa, 201.3 kPa, and 248.9 kPa, respectively. Past research reported that under cyclic loading, the critical stress is about 50% of the failure stress under monotonic loading, i.e., $CSR = 0.5$ (Frost et al. 2004). Taking CSR of 0.5 for A_r of 25% gives a cyclic loading amplitude of 100.65 kPa. Considering the cyclic stresses induced under traffic and train loading, this study takes the cyclic load amplitudes of 100.65 kPa to study the effect of A_r under cyclic loading. Also, to analyze the effect of different cyclic loading amplitudes, experiments were performed at CSR of 0.2, 0.35, 0.5, 0.75, and 1 for A_r of 25% to classify the threshold amplitude level for GPSC-stabilized soils.

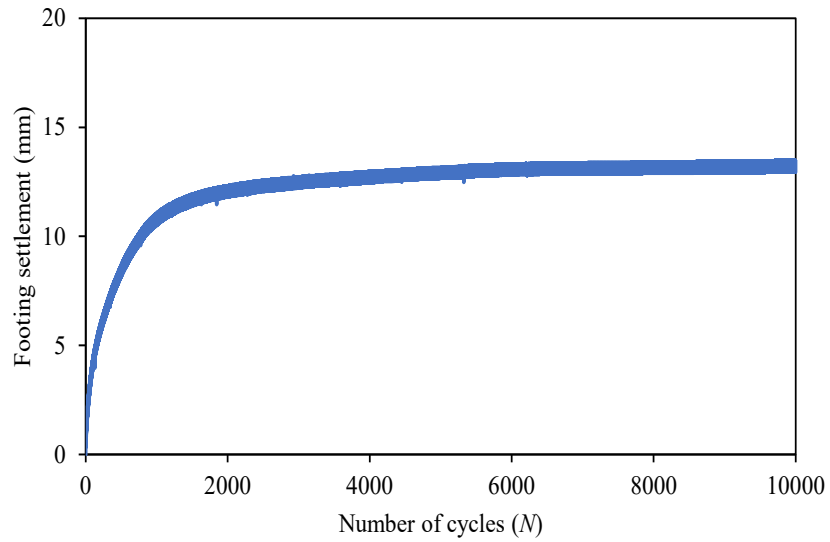
The cyclic loading frequency and magnitude of traffic loading (static and cyclic stress) applied in the model tests were the same as in the prototype. Similar considerations have been taken in the previous studies conducting model studies (Aqoub et al. 2020; Gupta et al. 2025; Li et al. 2023a; Pradeep and Kumar 2024; Shahu et al. 2023, 2024; Yoo and Abbas 2020; Zhang et al. 2020).



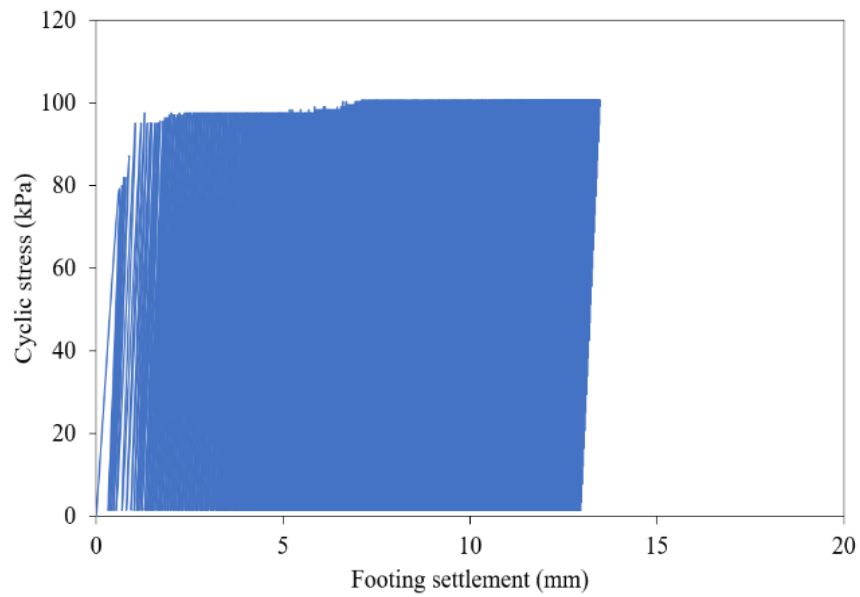
(a)



(b)



(c)



(d)

Fig. 3.12 (a) Variation of cyclic stress (q_c) pattern with number of cycles (N), (b) Typical variation of q_c with N from the stress-controlled cyclic model tests, (c) Variation of footing settlement with N , and (d) Variation of q_c with footing settlement.

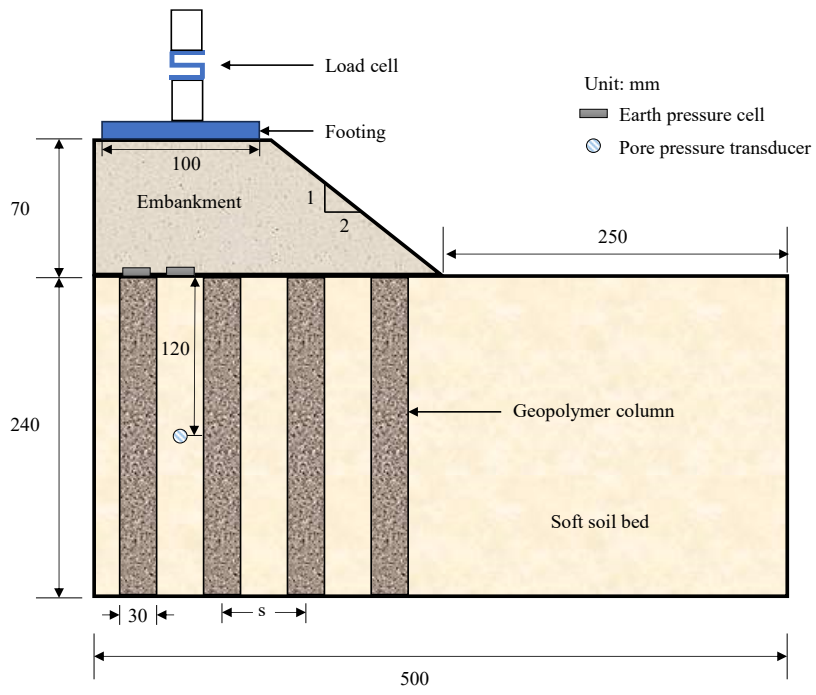
3.6 model tests on Embankment with geopolymer column foundation

3.6.1 Details of Model Tests

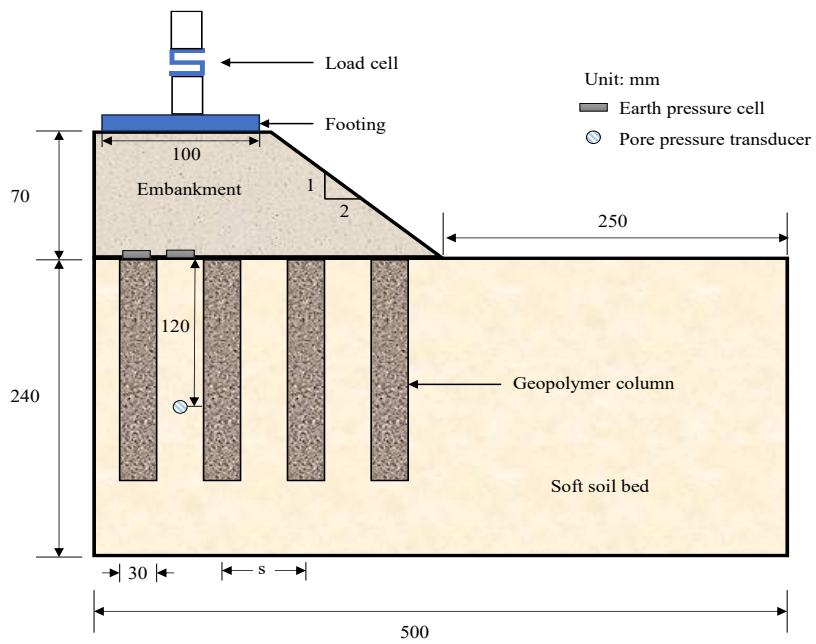
In this study, the physical model tests simulate an embankment over a composite foundation with geopolymer columns installed vertically at a uniform spacing in a horizontal clay layer in the square pattern and subjected to vertical loading. Fourteen experiments were conducted on the unimproved and improved soft soil models with various area replacement ratios (A_r) and l/h ratios (end bearing and floating columns) and subjected to static and cyclic loading.

3.6.2 Test Setup and Modeling Considerations

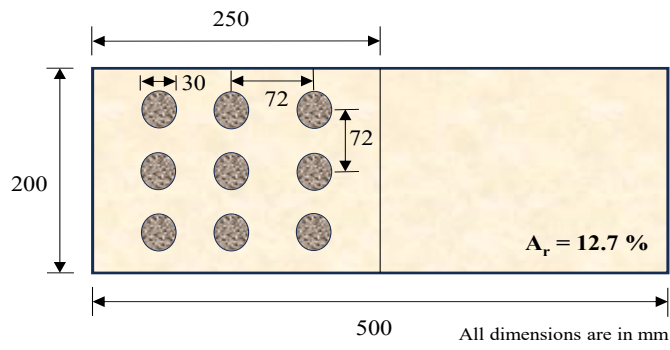
To analyze the behavior of soft clay improved with a group of geopolymer columns underneath an embankment, an experimental setup with a linear scale factor of 1/50 of the prototype was designed and fabricated. The model test tank used in the study has a length of 500 mm, a width of 200 mm, and a height of 400 mm, as shown in Fig. 3.13. The soft clay foundation bed was laid up to a depth of 240 mm in the tank, wherein the end-bearing and floating columns of 240 mm [Fig. 3.13(a)] and 180 mm [Fig. 3.13(b)] in length, respectively, were installed. The front side of the test tank was detachable to provide real-time monitoring of the failure mode of the improved ground in the model tank and made up of the same material as the rest of the tank.



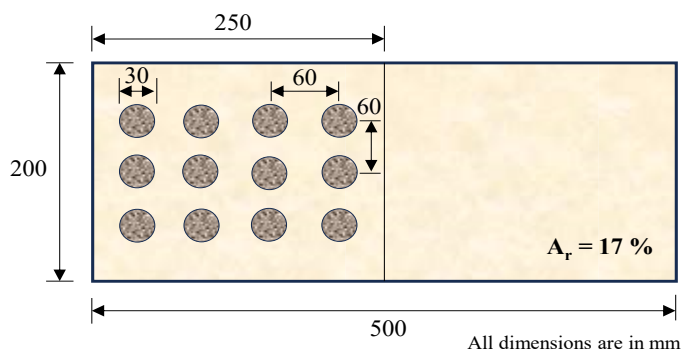
(a)



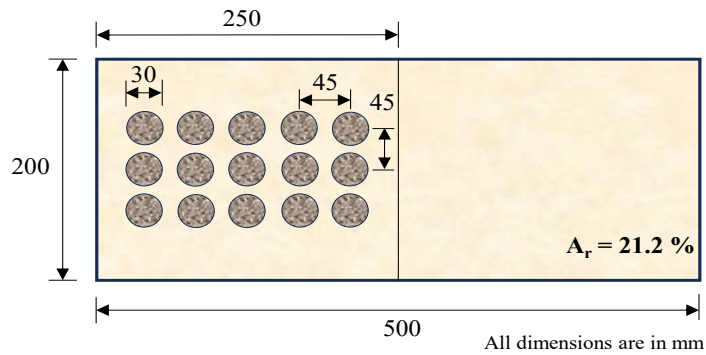
(b)



(c)



(d)



(e)

Fig. 3.13 Systematic view of the embankment model with instrumentation: (a) Cross-sectional view of the end-bearing GPSCs ($l/h = 1$), (b) Cross-sectional view of the floating GPSCs ($l/h = 0.75$), and (c-e) Plan view of GPSCs arrangement for $A_r = 12.7\%$, 17% , and 21.2% respectively.

Due to cross-sectional symmetry, the embankment on the soft clay was modeled in half (Chen et al. 2022; Fattah et al. 2017; Gu et al. 2024). The embankment had a height of 70 mm (3.5 m for prototype), a crest length of 110 mm (5.5 m for prototype), and a slope of 1V:2H. In the DSM technique, the diameter of the column varies from 0.5 to 2.1 m, with spacing usually between 1.0 and 1.5 m center-to-center and length between 10 and 30 m (Rashid et al. 2017; Kitazume & Terashi 2013). Assuming the diameter and length of the GPSCs as 1.5 m and 12 m, respectively, for the prototype, the diameter and height of the model were 30 mm and 240 mm. The other most important parameter in the DSM technique is the area replacement ratio (A_r), which is the ratio of the total area of columns to the total area of the improved ground by the columns. The area replacement ratio in the field varies from 10% to 30%. Hence, three area replacement ratios of 12.7%, 17%, and 21.2% were considered, and columns were installed in a square pattern for the analysis in this study, as shown in Fig. 3.13(c), 3.13(d), and 3.13(e).

3.6.3 Model Preparation and Group Column Installation

Deep soil mixing is a promising ground improvement approach for soft soils with an undrained shear strength (c_{us}) of less than 25 kPa. In light of this, soft clay beds with an undrained shear strength of 5-5.5 kPa were used for the model testing. Multiple trails of vane shear tests were carried out to ascertain the required water content to generate a soft soil bed at the desired shear strength. Dry kaolin clay was mixed with approximately $38 \pm 1\%$ water content to attain the desired shear strength. A mechanical mixer was used to make a homogeneous soil paste. The prepared paste was allowed to rest for a day to attain uniform and consistent moisture equalization. To avoid moisture loss, a damp jute bag was placed over it. To reduce side-wall friction, the inner sides of the tank were covered with a polythene sheet and silicon-greased before filling it with

soft soil. The paste was added to the tank in eight layers, each layer of 30 mm in thickness thoroughly kneaded by hand to eliminate any trapped air voids. In order to prevent moisture loss, the tank was covered with a moist jute bag and left undisturbed for two days to restore its thixotropic strength.

The DSM column installation and preparation process were adapted from previously published research by (Bouassida and Porbaha 2005; Fang and Yin 2007; Rashid et al. 2018; Said et al. 2019; Yin and Fang 2010). A set of pre-formed geopolymer-treated columns was prepared by mixing kaolin clay slurry (with the same moisture content as the bed) with 20% geopolymer binder derived from the dry kaolin clay mass. In order to achieve a required undrained shear strength of 5 kPa, the oven-dried kaolin clay was initially mixed with the required amount of water. The liquid alkali activator and precursors were combined for a few minutes to get the required geopolymer paste. Finally, kaolin clay paste and geopolymer paste were mixed for five minutes. After that, the paste was poured into cylindrical PVC pipes of 30 mm diameter and lengths of 240 mm and 180 mm for the model test. To release the trapped air spaces, each layer was tapped 25 times. Concurrently, a 38 mm diameter by 76 mm height cylindrical mold was used to produce the sample for UCS testing. The samples were dismantled after 24 hours, sealed in cling film to prevent moisture loss, and allowed to cure for 21 days at a controlled room temperature of 27 ± 1 °C.

To install the geopolymer-treated soil columns (GPSCs) inside the remolded soil bed in the model test tank, the locations of the columns were first marked on the clay bed. For this, three wooden guiding plates with different numbers of circular holes with a diameter of 30 mm with different spacing were used to get the corresponding area replacement ratio. The utilized wooden plate has 9, 12, and 15 holes, corresponding to area replacement ratios of 12.7%, 17%, and 21.2%, respectively. The wooden plate was

held in a horizontal position above the model ground, and a thin open-ended PVC pipe with an outer diameter of the targeted diameter of the GPSC was then slowly pushed into each hole of the model ground up to the targeted depth. A leveling bubble was used to ensure that the pipe penetrated vertically into the model ground. A thin layer of silicon grease was applied on the pipe's inner and outer surfaces so there was no friction between the soil and the pipe. An auger was used to remove the soil inside the pipe. After that, the pipes were taken out one by one carefully and GPSCs were inserted in each hole up to the required depth simultaneously. During this process, no heaving was observed in the model ground. Fig. 3.14. shows the systematic diagram of installing GPSCs inside the model ground.

The construction of the embankment fill was initiated after the installation of columns. Using a mixer, a predetermined weight of silty sand was combined with water to attain an optimum moisture content of 8.45% and a maximum dry unit weight of 18.22 kN/m³. The fill material was placed in five layers. Each layer was compacted using a tamping rod until the required embankment height was achieved. The model ground was left for seven days before the loading tests were conducted.

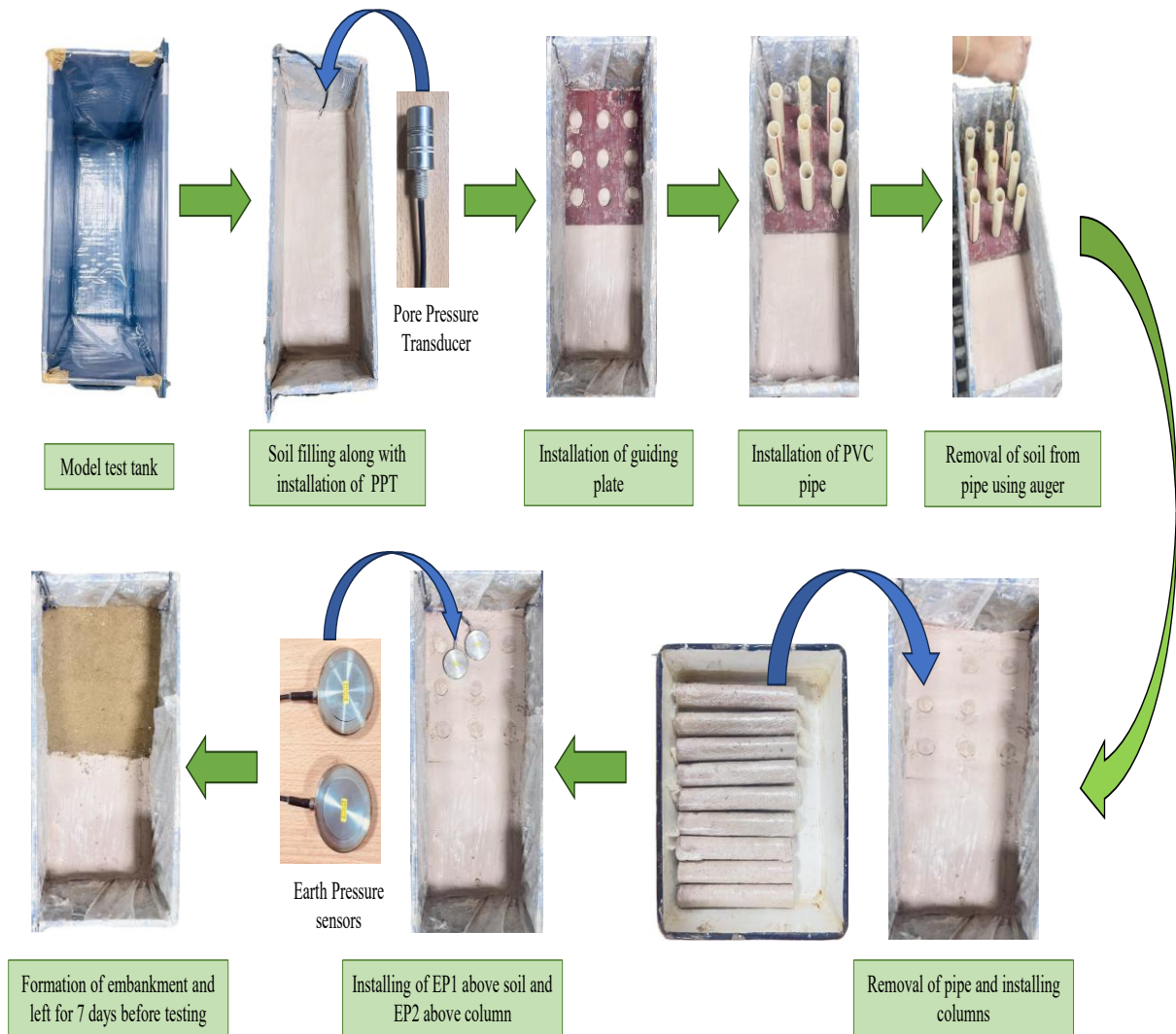


Fig. 3.14 Systematic diagram of GPSCs installation in embankment model.

3.6.4 Instrumentation and Loading Procedure

One miniature pore pressure transducer with a capacity of 50 kPa and two earth pressure cells with capacities of 200 kPa and 1 MPa were installed in the tank for the model tests. While preparing the soil bed, the pore pressure transducer was installed. After the installation of the columns, the earth pressure cells were placed below the crest of the embankment. To measure the load acting on the column and the surrounding soil, a 200 kPa earth pressure cell was placed above the soil bed, and a 1 MPa earth pressure cell was placed above the column. Fig. 3.13(a) and 3.13(b) display the exact position of these sensors. A multi-purpose, 20 kN capacity, servo-controlled pneumatic loading

data acquisition system was used to apply static and cyclic loading. Fig. 3.15 shows the model test setup, data logger, pore pressure sensor, and earth pressure sensor. The displacement of the footing was measured using LVDT connected to the data logger. The details of all the instruments used in the study are presented in Table 3.5.

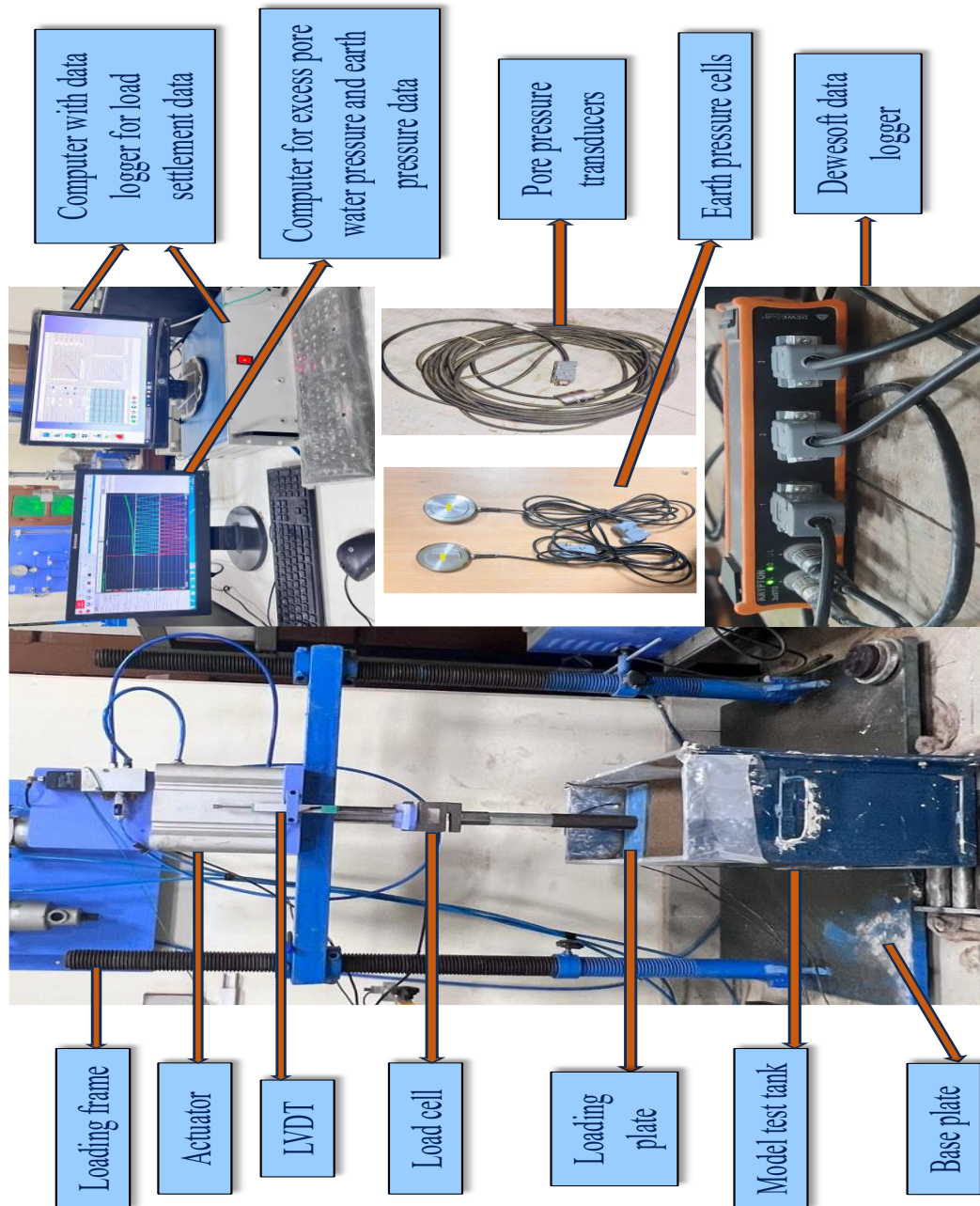
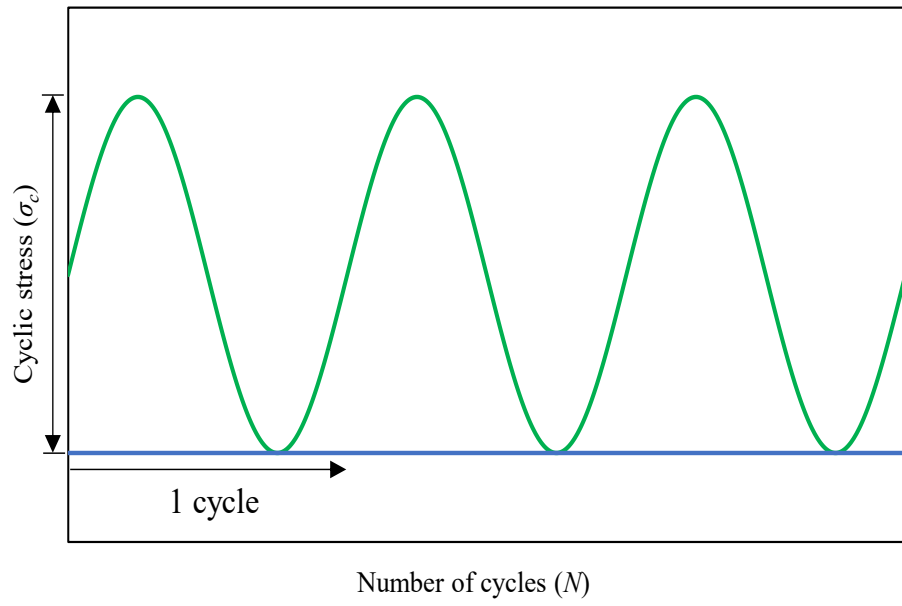


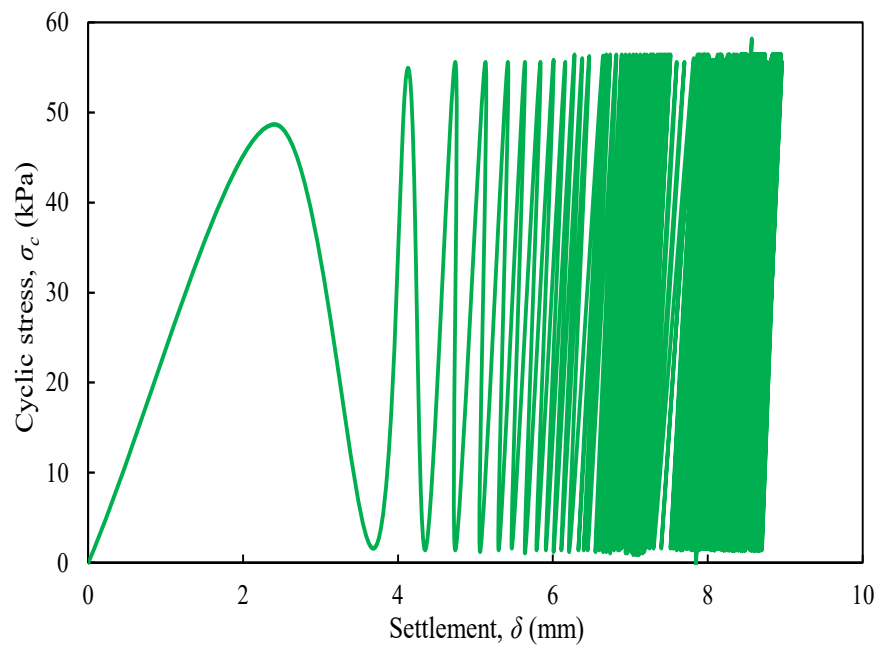
Fig. 3.15 Model test setup.

The static loading tests were carried out in strain-controlled mode at a vertical displacement rate of 1 mm/min to simulate the undrained condition during loading. The displacement rate was obtained using Eq. (3.2), proposed by (Lehane et al. 2008), to determine the normalized velocity (V) that is used to define the undrained condition at a particular displacement rate.

The cyclic loading tests were performed in a load-controlled mode in the one-way sinusoidal cyclic form at a frequency of 1 Hz for 10,000 cycles or until failure, whichever is reached earlier (Fig. 3.16(a)). Prior research has reported a slight impact of the frequencies in the 0.1 to 10 Hz range on the accumulated vertical strain and pore water pressure (Liu and Xiao 2010; Toyota and Takada 2021). (Lin et al. 2019) reported that traffic loading magnitude ranges between 10.6–44.5 kPa for different vehicles, from cars to fully loaded dump trucks on the roadway. (Kermani et al. 2019) reported that the load intensity varies between 12.5-24 kPa for low-volume traffic load. This study considered a slightly higher load intensity of 55 kPa to simulate the actual field loading condition. Fig. 3.16(b) shows the typical vertical stress-settlement curve for 10,000 cycles under load-controlled cyclic loading tests on the prepared models.



(a)



(b)

Fig. 3.16 (a) Cyclic loading pattern with number of cycles and (b) Typical variation of δ under cyclic stress (σ_c) of 55 kPa.

